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GROUND CONTROL IN MINING

ROCK STABILITY

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Pillar Collapse at Welgedacht Colliery, South Africa: A Case Study

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ABSTRACT

During the process of pillar extraction at the Welgedacht Colliery, an unexpected pillar collapse resulted in the entrapment of equipment. The area was visited soon after the event to evaluate the stability of the area for the purposes of extracting the equipment. It was found that the area had stabilised sufficiently to allow work without undue risk to the safety of workers. The equipment was reclaimed successfully. A subsequent investigation into the reasons for the collapse was conducted. It entailed detailed underground and surface subsidence observations and numerical modeling. It was found that none of the perceived negative factors that were present at the time could result in pillar loads that were sufficient to cause the collapse. It was concluded that the local pillar strengths were less than the nationally used value and that they were possibly further reduced by aging.

INTRODUCTION

Welgedacht Colliery is situated in the northern region of the Kwa-Zulu Natal province of South Africa. It is one of South Africa's oldest operating mines, production having started shortly after the turn of the century. It produces a high quality bituminous coal for the export market.

Mining was done both underground and from the surface where the coal was less than 30 m deep. The underground mining methods included drill and blast and continuous miner development, pillar extraction and pillar splitting. The mine changed ownership a number of times and with each new owner came a different mining philosophy. As a consequence there were intermittent periods when pillar extraction was done and not done; this is reflected in the pillar sizes on the mine. Some are suitable for secondary extraction and some are too small.

While four coal seams are present in the area, only two - the Gus and the Alfred - have been mined. The interburden was from 2 to 5 metres thick and of varying quality, consisting of shale and

sandstone. The surface topography was steeply undulating, shown in Figure 1, and the coal seams outcropped on the sides of hills. The mining heights ranged from 2,5 to 4 m - in general, the lower (Gus) seam was thinner than the upper (Alfred) seam.



Figure 1 General view of the area where the mine is situated.

The immediate roof of the Alfred seam was a competent sandstone layer of about 7 m thick, see Figure 2. The remainder of the overburden consisted of alternating layers of shale and sandstone.

The mine recently ceased underground operations and for the last few years mining was concentrated on the last remnant blocks of coal. The age of the mine, the differences in pillar sizes, the rapidly changing mining depth and the double seam nature of the mine all contributed to frequent section moves and sometimes a need to mine small areas between blocks of older mining. This situation was further complicated by a number of sealed fire areas and areas where pillars had collapsed in the past. In colloquial South African mining language this is referred to as mining cockroach holes.

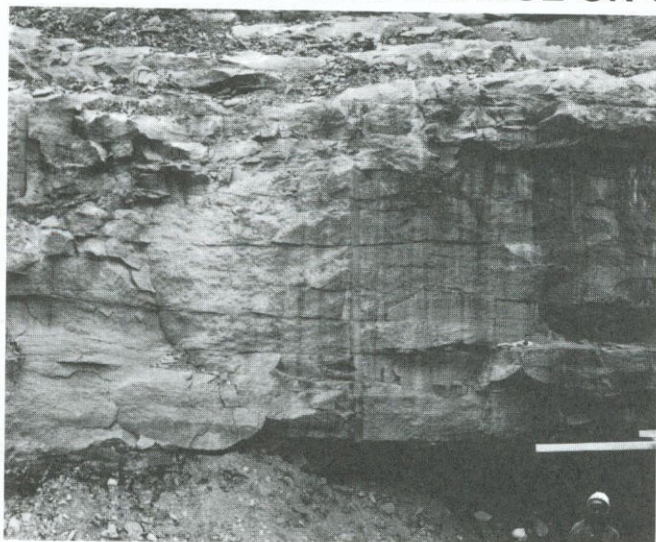


Figure 2 Photograph taken at the entrance to the mine adit, showing the competent sandstone layer in the immediate roof.

On 29 December, 1998 pillars collapsed in a pillar extraction section on the Alfred seam. The collapse was estimated to have involved some 70 pillars over an area of just more than 2 ha. No injuries were sustained, but a number of machines were trapped inside the collapsed area. This included a roofbolter, two shuttle cars, a loading chute and switch gear. The section was inspected to advise the mine on the possibility of entering the collapsed area to extract the equipment. Assistance was also rendered during the extraction process.

GENERAL DESCRIPTION OF THE COLLAPSED AREA

Pillar extraction was in progress at the time of the collapse. There were two pillar sizes - the first three lines were 18 m square pillars and the rest of the panel was to have been 12 m square pillars. Road widths were nominally 6 m. The collapse occurred when the fourth pillar in the last line of the large pillars was being extracted, see Figure 3. The depth below surface at the centre of the collapsed area was just over 70 m. There was an old gob immediately west of the collapsed area and another one about 75 m to the south. Towards the north-east pillars had been destroyed by a fire some years ago and there was another fire area further to the west. Both of the fire areas were sealed off.

The Gus seam had been mined underneath. The extent of mining on the Gus seam was less than on the Alfred, see Figure 4. The collapse occurred on the edge of an abutment on the Gus seam, but no pillars had been extracted on the Gus seam in the vicinity.

EYEWITNESS ACCOUNTS OF THE EVENT

According to workers in the section on the day of the collapse, the pillars started cracking and spalling early during the morning shift. This was not confined to the immediate vicinity where pillar extraction was in progress, as is normal during that type of operation, but occurred over a wide area. The occasional distant

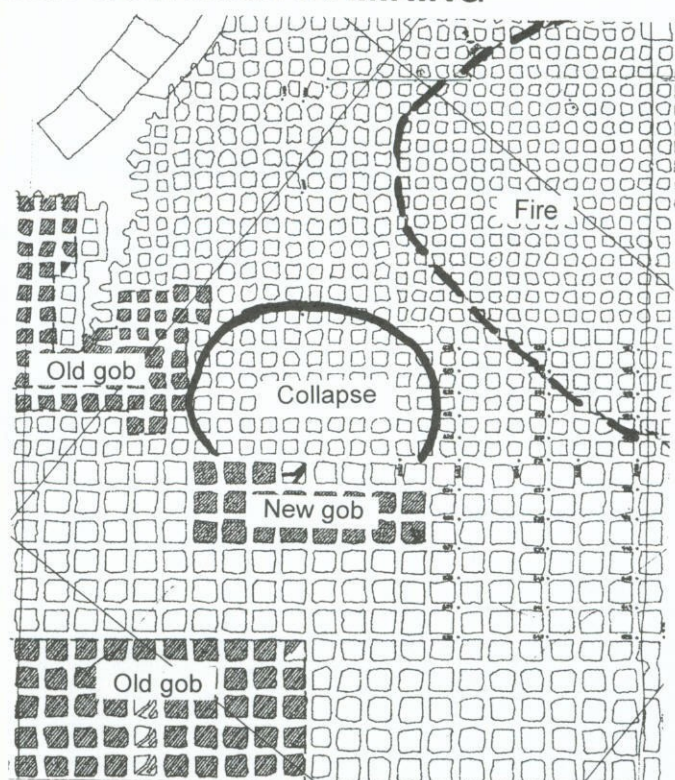


Figure 3 Plan of the Alfred workings, indicating the collapsed zone.

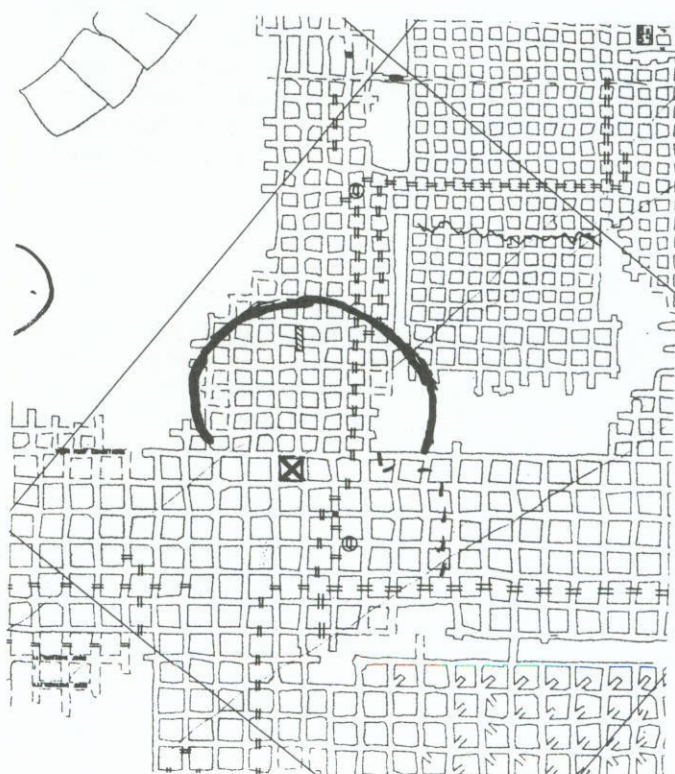


Figure 4 Plan of the Gus workings underneath the collapsed Alfred workings. The collapsed area and the position of the pillar that was being mined at the time of the collapse are superimposed on the plan.

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thumping noises associated with overburden failure were also heard. After about two hours the intensity of this rock activity had increased to the extent where the section was evacuated.

Once on surface, discussion ensued in the team and they considered their retreat to have been premature. The continuous miner operator and an assistant then went back into the section to drive the continuous miner back to a safe position. Once this was accomplished, they did not go back into the mine because the rock noise continued to intensify.

Later on that same afternoon a senior mine official inspected the area and found that the pillars had crushed (the free height was halved) and that the noise and spalling had abated.

The section was directly in line with the adit entrance to the mine, less than 300 m away. At no stage a wind blast or anything else was detected that could be linked to sudden or violent failure. This was in contrast to a similar experience on the same mine a week before this failure, when a long panel, where split pillars were holding up, failed with sufficient violence to dislodge the explosion proof flame door on the main fan chamber of the mine.

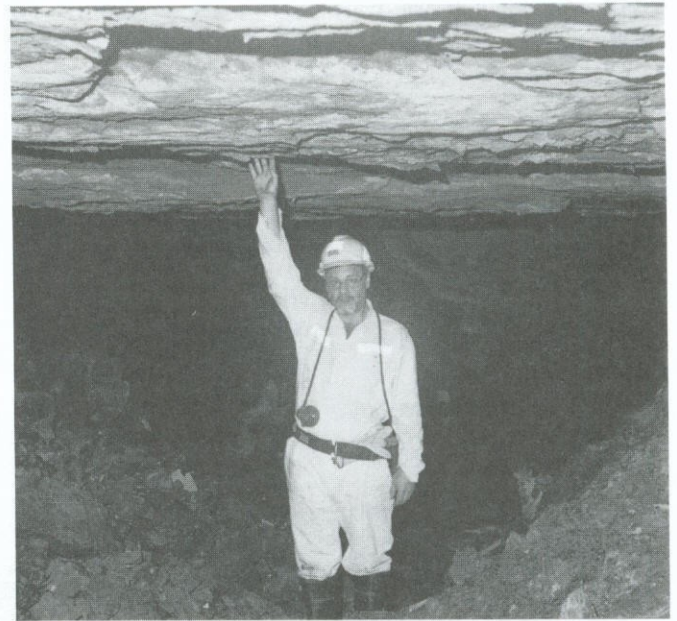


Figure 6 After the collapse, the free height was reduced to about 2 m.

UNDERGROUND OBSERVATIONS

The pillar collapse area was easily discernable: the pillar conditions changed from undamaged to totally crushed out in the space of three lines of pillars, a distance of about 50 m. In the undamaged area, the free height was about 4 m and in the crushed area, less than 2 m. This is shown in Figures 5 and 6.



Figure 5 The workings in the area where the pillars did not collapse was about 4 m high.

The pillar sides were covered in finely broken coal of about 10 to 20 cm diameter. The angle of repose of the broken material was about 45° . In the areas where no loading out for access was done, the free height was about 1,0 to 1,5 m and the span at roof height was in the range of 2 to 3 m. In general, the roof was in fair condition with isolated falls of approximately 50 cm to 1 m high. This is shown in Figure 7.



Figure 7 In general the roof was in good condition, although some falls of 0,5 to 1 m had occurred.

On closer inspection, the roof was seen to be severely cracked with fresh fractures about 1 cm or more wide. These wide fractures were spaced at about 2 to 3 m with finer cracks in between. An example is illustrated in Figure 8.

Once the finely broken rubble against the pillar sides was removed, the interior coal was seen to be broken into larger blocks, measuring 50 to 100 cm. The roof contact was distinct and apparently cohesionless, but can be described as rough, with an estimated coefficient of friction about 1,2 to 1,5. The sandstone roof was wavy with wave amplitudes of about 10 cm

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and wavelength of approximately 1 m. This is shown in Figure 9.



Figure 8 Warning wedges placed into the cracks.



Figure 9 Illustration of the larger blocks into which the inner portions of the pillars had failed and the wavy nature of the roof.

One shuttle car - shown in Figure 10 - was trapped by a roof fall. The other one was free, but behind the trapped one. The roofbolter was in good condition but also behind the trapped car. The switchgear was severely damaged and the loading chute likewise not considered worth rescuing. The two shuttle cars and the roofbolter were the items to be extracted. The first operation would be the most difficult one. If the shuttle car could be removed, it would be easier to reclaim the other equipment.

Apart from the odd distant, muffled thumping noise (sometimes accompanied by a very light tremor) the area was quiet. There were no sharp cracking noises associated with high loads on



Figure 10 The shuttle car that was trapped by a roof fall, blocking the road to the other equipment.

coal pillars. The overall impression was that the pillars had crushed out - whatever could fail had failed - and that the roof as a whole had come to rest on heaps of coal rubble. No further problem of the nature of a general collapse was foreseen.

However, there was some concern for the stability of the immediate roof. The plan to extract the trapped car was to fix a chain to it and then to extract it with a "He-man" - also called a "tooth-extractor" on other mines. Shown in Figure 11, this consists of horizontally mounted hydraulic cylinders that are used to pull the trapped equipment. It is fixed in position by two thrust cylinders inclined slightly forward off the vertical. The concern was that the thrust cylinders could disturb the already broken roof, causing more roof falls.

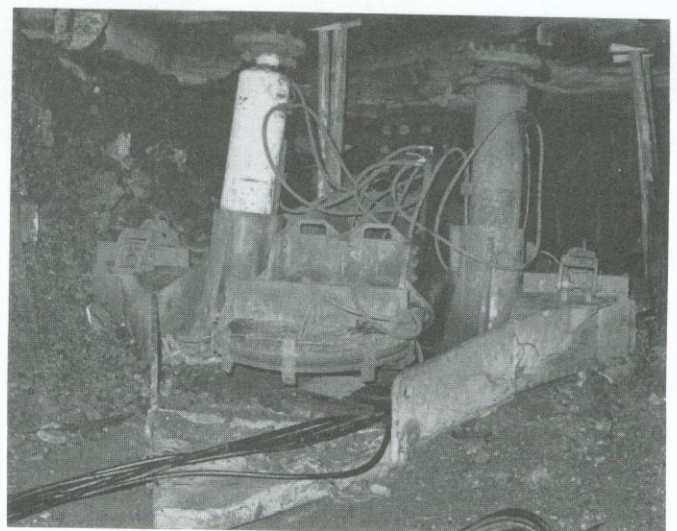


Figure 11 The "He-man" that was used to extract the shuttle car.

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PRECAUTIONS DURING THE EXTRACTION PROCESS

As no further major instability was foreseen, it was recommended to proceed with the reclamation process. A number of precautions were to be implemented. The mine is relatively remote and there was some urgency to extract the equipment, because the condition of the fire seal walls isolating the two fire areas in the vicinity were unknown. The emphasis was thus on using whatever was available on the mine rather than to order and wait for more sophisticated equipment. The following were the most important precautions:

- Only persons involved in the extraction process were to be allowed in the area.
- Minimum number of people to be involved.
- Guards to be placed at the last unfailed pillar line.
- Warning wedges to be inserted in the roof cracks.
- Stooping convergence warning devices to be used.
- Minimum coal to be removed for passage.
- Mine poles and timber packs to be installed.
- "He-man" to be operated remotely.
- Nobody to be within 20 m of the "He-man" when in operation.
- People in the extraction area to spread out linearly.
- Additional regular checks on gas sensing instruments to be done.

The equipment was reclaimed without any major problems. The only mishap occurred when an artisan slipped on spilt oil on top of the "He-man" power pack and fell off, bruising his arm in the process.

POSSIBLE CAUSES OF THE PILLAR COLLAPSE

On cursory examination, there were several potential causes of the collapse:

- There was a change in pillar size at the position where the mining was done at the time of the collapse.
- There was an abutment on the lower seam at the position of the mining.
- There were gobs on three sides of the mining: two pillar extraction areas and the fire area.
- Although the mining depth at the position of the collapse was not excessive (about 70 m), the mining area was overlain by a valley, the surface sloping upward on both the eastern and western sides.

It was believed that if not one of the above factors were in itself sufficient to cause a collapse, then the combination of those factors would be. It was attempted to quantify the contribution of each of those factors to the eventual collapse. This was done by a succession of LAMODEL analyses, each time removing one or more of the potential causes in the model.

Some form of adjustment of the LAMODEL results was required. The pillar loads obtained with the model were to be used to replace the tributary area loads in a safety factor formula. The formula was empirically derived using the

tributary area load assumption. The pillar strength that was so derived is a function of the load assumption: if the assumed loads were greater than the real loads, then the derived pillar strength will also be greater than the real strength.

For the safety factor calculation to be valid, either the pillar strength or the loads have to be adjusted. In a case like this one, it is simpler to adjust the loads. This was done by using LAMODEL to determine the pillar loads in a situation where there were no external disturbances like nearby gobs, other seam mining, etc. That load was then compared to the load obtained with the tributary area assumption and the ratio of tributary area load to LAMODEL load was applied to the other LAMODEL loads obtained with the more comprehensive models. In this way, LAMODEL was thus essentially used to quantify the contributory effects of the other mining in the vicinity. This can not be done by simply applying the tributary area theory.

In order to design a realistic model, it was important to know whether the surrounding gobs and the fire area had resulted in full overburden failure or not. This was determined indirectly by observations of subsidence on the surface, by visual inspection of cracks. The crack positions were fixed by means of a GPS instrument. Although the accuracy of the instrument at the time of the investigation was limited to 15 to 20 m, this was considered sufficient for the purpose: it was only important to determine whether defined areas had goafed or not. The exact positions of the cracks were less important.

A variety of older and very fresh cracks were found, as shown in the examples in Figures 12 and 13. The cracks were superimposed on the Alfred seam plan, shown in Figure 14. From this, the following were deduced:

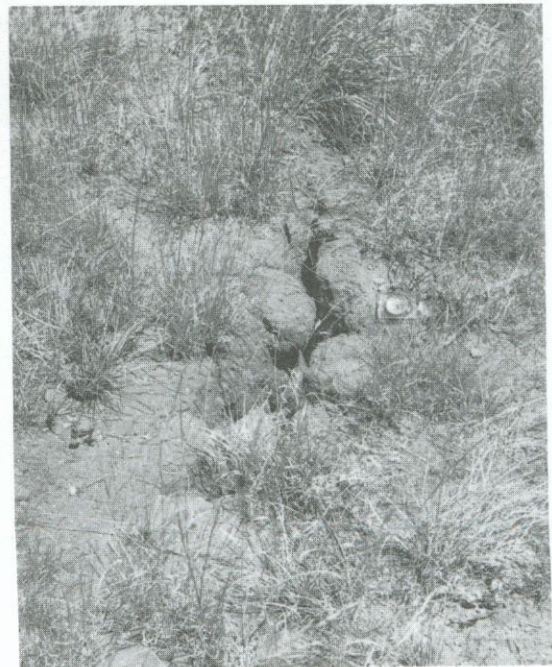


Figure 12 Example of one of the older cracks that were found, indicative of subsidence that occurred more than a year before the collapse.

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Figure 13 Part of the fresh crack that was found, associated with the Alfred seam collapse.

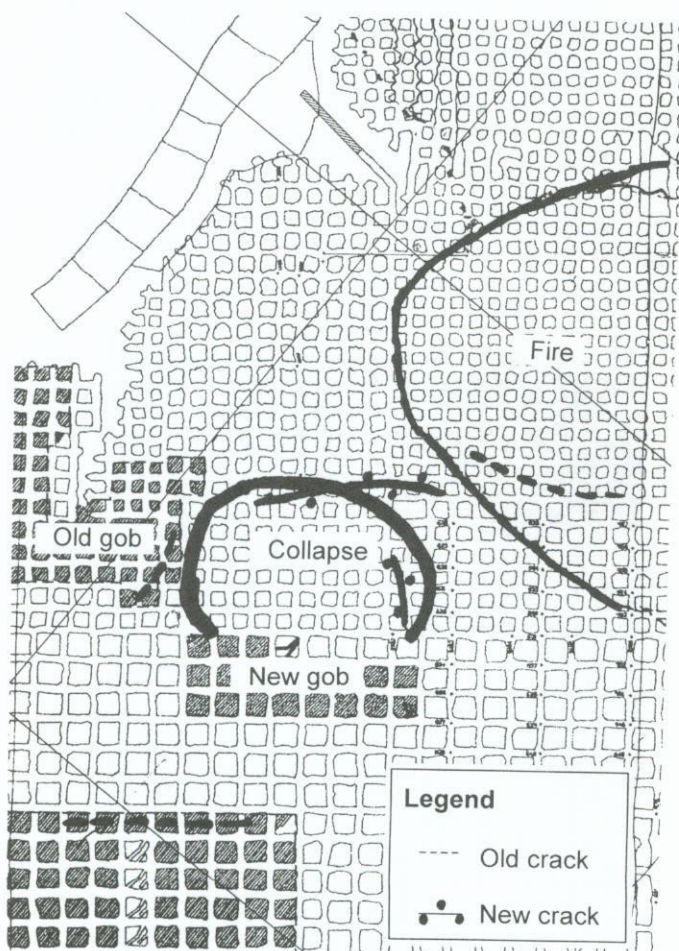


Figure 14 Approximate positions of the old and young cracks transferred to the mine plan of the Alfred seam.

- The fire area and the old pillar extraction areas had goafed fully – thus minimal load transfer from those areas would have occurred.
- The extent of the current collapse was wider than had been believed, indicated by the length of the fresh crack on the northern boundary of the collapse.
- The presence of strong sandstone layers in the roof transferring stress from the adjoining goafed and burnt areas to the failed pillars could be discounted, as they must have failed to result in the observed subsidence.

According to the LAMODEL analyses, the loading contributions of each of the postulated contributory factors were relatively small and not sufficient to have caused the collapse, even when added together. The safety factors of the pillars, using the proposed new strength factor for South African coal (van der Merwe, 1999) were greater than 2.5 for all the modeled conditions. The safety factors for the different situations are summarised in Table 1. Figure 15 is an example of a part of the LAMODEL output, showing the load contours on the pillars next to the gob line.

Table 1 Summary of safety factors for the different situations.

Case	Situation	Load (MPa)	Strength (MPa)	S.F.
1	Hand calculation: no second seam, no topography, no boundary effects	4,16	10,72	2,58
2	Single seam, gobs present, no topography	4,27	10,72	2,51
3	Single seam, gobs and topography present	4,27	10,72	2,51
4	Double seam with gobs and topography	4,24	10,72	2,53

In this case, no clear reason for the collapse could be found in a stress analysis. Not one of the postulated contributing factors resulted in loads that were sufficiently high to cause the collapse. The loads on the bottom (Gus) seam pillars were also analysed, but there the safety factors were even greater, due to the lower mining height resulting in increased strength of the pillars. This result was confirmed by mine personnel who inspected the Gus seam pillars after the collapse; they reported that the pillars had scaled, but not failed.

The fact remains that the pillars on the Alfred seam did indeed collapse and if the reason could not be found in the pillar loads, it had to be in the pillar strengths. The pillars were developed more than 20 years ago and were seen to be heavily jointed. It is thus not unreasonable to assume that the strength calculation, which is based on all pillars in South Africa that have failed, is not valid for Welgedacht. It was previously suggested to reduce the safety factors for the pillars in a different part of the mine by a jointing based reduction factor, Wevell (1994).

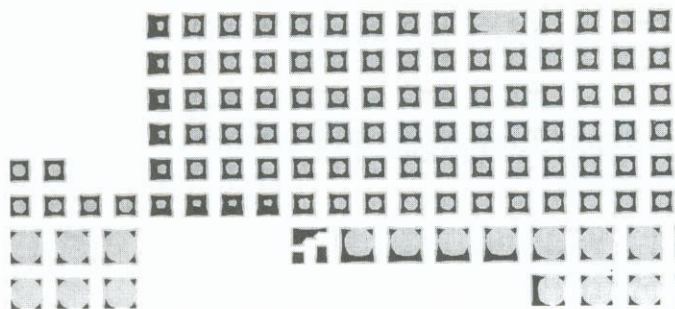


Figure 15. Part of the LAMODEL output, showing stress contours on the pillars adjoining the gob line.

There is one other area of the country, the Vaal Basin, where the pillars had previously been shown to be weaker by van der Merwe (1993). The equivalent of the proposed new safety factor for the Vaal Basin is:

$$\text{Strength} = 2,3 \frac{w^{0,82}}{h^{0,76}} \quad [1]$$

where w is the pillar width and h is the pillar height.

Applying equation [1] to the Welgedacht pillars will decrease the safety factors to around 1,4. While this is still high, it is approaching that range of safety factors where relatively minor deviations have a significant effect on the increase in failure probability. It is also not unreasonable to assume that the time effect decreased the pillar strengths even further. The pillars were beyond the predicted minimum life spans, using the prediction method of van der Merwe (1998).

CONCLUSIONS AND DISCUSSION

Even though the pillars had been destroyed, their convergence being in the region of 50%, the area was still accessible. The coal rubble had sufficient strength to support the 70 m of overburden without undergoing further displacement. This adds a further data point to the post failure load/displacement curve of in situ coal: at a load of 11 MPa the strain is 0,5 for pillars with a width : height ratio of 3,0.

A number of circumstances that could in general be considered as detrimental to safe pillar extraction existed simultaneously at the Welgedacht collapse. However, these could not have created a stress environment to result in the collapse. Even the bridging effect of the strong sandstone layers did not contribute to the failure.

The remaining half of the stability equation is the pillar strength. If excessive loads could not be found, then the pillar strengths had to be less than generally believed to result in the failure.

This case study highlights one of the important controversies in South African rock engineering, where differentiation of coal pillar strength is not the general practice. It also alludes to the other important issue, namely the effect of time on coal pillar strength. Research funds for this could thus far not be obtained

on a national scale. As mines get older and the coal reserves less, there will be attempts to extract pillars that were developed a long time ago and as of yet there is no generally applicable method with which the strength decay can be quantified.

The width to height ratio of the failed pillars was about 3,0. This places them on the borderline of potential for violent failure according to Chase et al (1994). Using the van der Merwe (1999a) Violence Index yields a value of 1,15 which is also on the borderline of a dangerous situation according to that classification. Both of these norms appear to be borne out by the eyewitness descriptions of the event: it was quick but not violent.

ACKNOWLEDGEMENT

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Causes of Massive Directional Roof Falls in Room and Pillar Mines - Two Case Studies

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ABSTRACT

In recent years many roof falls have been conveniently attributed to the adverse existence of a high horizontal stress. The normal practice of not conducting a follow-up study in a roof fall investigation usually leads to a wrong conclusion. This paper describes two cases of massive oriented roof falls in two separate room and pillar mines. The investigation in each case involved the analysis of past history and current conditions from which remedial measures were recommended and implemented. A follow-up period of up to three years was performed to evaluate the effectiveness of the recommended remedial measures. In both cases, high horizontal stress was proven positively not the cause, contrary to the original conviction of mine management. Rather, a weak immediate roof was the basic cause. Under a weak immediate roof, a proper roof bolting plan and adequate entry width and intersection span must be strictly observed.

INTRODUCTION

In recent years, many roof control problems have been attributed to the existence of a high horizontal stress; the common sense is that if an entry is perpendicular to the direction of a high horizontal stress, roof control problems such as cutter roofs and roof falls are liable to occur. Conversely, if roof falls occur preferentially in one direction in a section or sections of a mine, it is normally concluded that these roof falls must be due to the effects of a high horizontal stress oriented in a direction perpendicular to it. In such cases the most frequently recommended measure is to change the direction of the entry development. For longwall mining this could be a difficult task because longwall gateroad development is designed normally 2-3 years ahead of retreat mining. But for room and pillar mining changes can be made at any time in most cases.

Another interesting fact in the investigation of roof control problems is that normally researchers or consultants are called to investigate the causes of a roof control problem either

immediately or sometimes after it has occurred. The typical procedures in the investigation of a ground control problem are that the investigators study the site employing various ground control techniques including monitoring by instrumentation; Based on these data some causes are identified and remedial measures recommended to correct the problems. Most studies normally stop there and a follow-up study to confirm the conclusions/recommendations is for various reasons seldom performed!

This paper describes two case studies of massive roof falls in two small mines employing the room and pillar mining method in West Virginia including a follow-up period of 2-3 years after the study. The advantages of investigating a roof control problem in a small mine is that due to its fast rate of advance and simple management structure, any recommendations can be implemented and the results obtained quickly. Conversely it was due to these same reasons, a sophisticated roof control monitoring system can not be economically and effectively implemented in time to obtain the necessary data.

MINE A

Mining and Geological Conditions

Mine A is a drift mine and located in southern West Virginia (Fig. 1). The surface topography is rugged with high relief of valleys and mountain ridges. The seam is 54-60 in thick and varies from 160 to 435 ft deep. The immediate roof is thinly laminated black shale 1-2 ft thick overlain by gray shale, also laminated, of varying thickness. Above the gray shale is a soapstone followed by a rider coal. The black shale and gray shale are easily weathered once exposed to the air and fall off in small pieces.

Mains were developed by a seven-entry system in which chain pillars were 40 x 40 ft and the entry was 18 or 20 ft wide depending on location. The 4-ft long fully grouted resin bolts



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(3/4-in rebar in 1-in hole) in 4 x 4 ft pattern were used with the side bolt 3-4 ft from the ribs. During development, a panel of rooms were developed to the left. Room pillars were 20 x 20 ft and rooms were 20 ft wide. The length of panels varies with topography.

When the initial study was performed, the 1st SW Mains were developed in the SW direction and the second mains (1st SE Mains) were in the SE direction. There were no roof falls in the 1st SW Mains. But cutters and roof falls started as soon as the 1st SE Mains development began.

Roof Falls

A roof fall inspection was performed in late 1993. Entries and crosscuts were in excellent condition immediately after development, but roof cutter began to develop 10-12 crosscuts outby the face. The #1 Roof Fall (Fig. 2) started in the room to the left of the 1st entry in Xcut #40 in the 1st SE Mains. Note the entry number is assigned sequentially from the extreme left, when facing inby, to the extreme right hand side. The cutter initiated on the outby side of Xcut #40 between entry #1 and #2 and finally developed into roof falls. Four-point cribs were erected at the 2nd entry intersection but the cutter continued until it hit the pillar between entry #2 and #3. It stopped for a week and then propagated again following the pillar rib first inby and then along the outby side rib of the Xcut between entry #2 and #3. The process repeated itself until it reached entry #7 where a roof fall initiated at the intersection of entry #7 and Xcut #40. It propagated outward to Xcut #39 where eight 4-point cribs in three rows (3-2-3 staggered pattern) did not stop it. In fact it overrode the cribs and went outby to Xcut #38 where 8-ft Double-Lok bolts and 9-point cribs were used to stop it. The roof fall in the Xcut between entry #6 and #7 was stopped by a row of cribs side by side after the cribs were squeezed from 56 to 42 in high. Note the Xcut between entry #3 and #4 was 14 - 18 ft wide.

The #2 Roof Fall began in and propagated along Xcut #36. The intersection between entry #5 and Xcut #36 had been cleaned up exposing the extent of roof fall on both sides. Coal was 5 ft high and the roof fall was 9 ft high to the rider coal. Cribs were squeezed to 33-42 in high.

In the #3 Roof Fall, the rocks fell in whole mass nearly vertically on both ribs of the Xcut #43. At the intersection between entry #6 and Xcut #43, the coal height was 64 in and the roof fall was 126 in high to the rider coal. Roof in Xcut #43 between entry #1 and #2 overhung in the outby side. At the intersection of entry #1 and Xcut #43, a cutter occurred at the outby side because cribs were erected at the center. At the intersection between entry #2 and Xcut #43, the roof fell at the entry center because cribs were installed on both sides. The roof fall was stopped in entry #1 outby Xcut #43 by heavy cribbing. Roof falls in the rooms were small and discrete and occurred at the entry center between cribs. Two rows of bolts were completely exposed but still hanging in the roof.

In entry 1 just inby Xcut #37, a roof fall was stopped by using 8-ft Double-Lok bolts with an 8 x 8 in plate and 6 ft resin bolts with a 6 x 16 in plate.

Inby Xcut #43 a cutter in the initial stage was visible in all the outby side ribs of the Xcuts and on both sides of entry #5. Therefore there was an overwhelming trend of cutters on the outby side rib of the Xcut. On the other hand cutters in the initial stage were observed in many entries on the left side when facing inby.

There were trends that uneven rib steps of larger than 20 in stopped a cutter propagation. In entries or Xcuts, the right side ribs were generally stepped while the left side rib were smoother.

In the rooms of the inby sections where 4 ft mechanical bolts were installed, cutter did not occur as often as in the entries.

In all cases, pillar ribs were intact and showed no signs of pressure. Only in one Xcut (Xcut #66 between entry #3 and #4) was the outby side floor broken and tilted up - a sign of pillar punching.

In the 1st SW Mains the immediate roof of black shale had fallen off between roof bolts as a result of weathering by the intake air in the previous two years. In some summer days the foggy air filled the entries all the way to and stayed at the face. But there were no major roof falls.

Analysis of Causes for Massive Roof Falls

The immediate roof is 9-11 ft high up to the rider coal and is weak because it consists of the laminated black and gray shale and soapstone with some containing highly fossilized planes and nodules. It was also highly susceptible to weathering by the wet-dry cycle of the ventilation air. Immediately after undermining and before subjecting to air weathering, it was sufficiently strong to self support and enable a normal cutting cycle to be made by the continuous miner. After bolting the roof bolting pattern was such that it strengthened the roof much more at the entry center than along and near the ribs, because the side bolts were normally 3-4 ft from the ribs. After subjecting to some period of air weathering, cutters began to occur at and near the ribs. There was evidence that cutters initiated but did not propagate in the areas where supplemental roof bolts were installed 1-1.5 ft from the ribs.

Since most roof falls and cutters extended in the SW direction in the 1st SE Mains and no roof falls occurred in the 1st SW Mains, it seems there exists a high horizontal stress in the SE direction even though these two mains were located next to an abandoned mine to the right.

There was no correlation between surface topography and roof fall locations (Fig. 3).

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Recommendations

Based on the analysis of the causes for roof falls, the following recommendations were made:

1. The current roof bolting system is insufficient. The roof bolting density should be increased to 3 x 4 ft pattern. Alternatively other roof bolting system with pretension may be preferable.
2. Change the direction of mains development toward the SW direction.

Follow-up Study

The 1st SE Mains was abandoned immediately after the investigation and the 2nd SE Mains were then developed at 20 degrees clockwise, followed by the 3rd SE Mains at 37 degrees clockwise from the 1st SE Mains (Fig. 1).

In order to determine the most suitable type of roof bolting system, a roof bolt test area was established when the 2nd SE Mains was first developed. The following four types of roof bolting systems were installed in sequence and each type for a four-block zone, all were installed in a 4 ft by 4 ft pattern;

1. Mechanical bolts, 5/8-in by 4-ft long,
2. HyTec bolts (a mechanical bolt with expansion shell, 1-1/4-in OD tube and plate), 5/8-in by 4-ft long, installed with 250 ft-lb torque.
3. Fully grouted tensioned rebar bolts, 3/4-in by 4-ft long installed with 200 ft-lb torque.
4. Fully grouted resin bolts, 5/8-in by 4-ft long.

(Note all 5/8-in bolts were grade 75 and 3/4-in bolts were grade 60)

There were no major roof falls in these four test zones except a few minor cutters here and there. Therefore mine management decided to return to the original roof bolting system (i.e. #4 in the above-mentioned test types) for the remaining inby section of the 2nd and 3rd SE Mains.

Mining proceeded normally and continuously inby without any roof fall incidents. It stopped at the location shown in Figure 1 for a two-week summer vacation in 1995. When the mine returned to work, the #8 Roof Fall began to work at the outby end and propagated inby at approximately one block per hour for 14 blocks. Note it turned to the 3rd SE Mains for two blocks and had to be stopped by heavy cribbing and 6-8 ft long bolts. The #9 Roof Fall on the other hand began at the face and worked outby for six blocks and had to be stopped by similar method. These two roof falls were 13-14 ft high exposing similar stratigraphic sequence as those in the 1st SE Mains.

Mine management decided to abandon the mine at this point due to high roof support costs.

Summary

Obviously it was not a high horizontal stress but the weak immediate roof. This type of roof bolt requires a special roof bolting plan especially at the intersections.

MINE B

Mining and Geological Conditions

The mine workings were developed under a mountain ridge with a maximum depth of approximately 280 ft. The mining height was approximately 90 in which consisted of 79 in of coal overlain by interbedded thin layers of sandy shales or sandstone and dark shale (i.e. stack rock) of 3-6 ft thick. The sandy shale was thinly laminated as a result of fossils and mica flakes aligned to form planes of weakness. There were also many slickensides in the roof strata.

The seven-entry Mains (Fig. 4) were developed in the North-South direction with square chain pillars, 55-60 ft wide and the entries were 18 ft wide.

Roof Falls

At the time of roof fall inspection in 1996, mining was being conducted in a new section near the portal area (N in Fig. 4). The roof and pillar ribs were excellent. There had been no roof falls in this new section.

A total of 18 roof falls in the old sections were inspected. All roof falls had similar features: 4-6 ft high and flat on the top with vertical or near vertical edges. There were two types of roof falls in terms of edge location; one with edges cutting off along one or both sides of the entry ribs and the other with both edges along the side row of bolts on both sides of the entry. They all initiated and covered one or more entry/crosscut intersections (both 3-way and 4-way). Many roof falls fell directly above the bolt anchorage horizons. But there were also roof falls that fell either above or below the anchorage horizon exposing the bolts, some of which were still hanging and others were broken.

Several types of roof bolts, all of which were 5/8-in in diameter installed on a 4 x 4 ft pattern, had been used in different sections: 4-ft long fully grouted resin bolts; 4- and 6-ft long fully grouted resin bolts installed alternatively both across the entry and along the entry direction; 5-ft long Double-Lok high tension bolts with occasional 12-ft long ones. Roof falls occurred in all areas regardless of the type of bolt used.

A great majority of the roof falls that extended more than one pillar block were oriented in the North-South direction although some in the outby area were connected by roof falls in the East-West direction to form a stair-step type of roof fall.

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All roof falls occurred one/two weeks after mining. In most cases the roof fell in a whole piece exposing the thinly laminated stack rock. Yet in some roof falls, the rock fragments broke along laminations/bedding planes and assumed a shape of thin plates. The floor was intact although in one small area along the belt entry, it cracked up around the edges of several pillars.

A great majority of the exposed resin bolts in the roof fall areas showed glove fingering effect, i.e. the resin cartridge film remained intact. They were very slippery and had little gripping effect. Many exposed resin bolts did not have resin annulus in the lower 1 - 2 ft.

In the inby side, all entries were 18 ft wide with the diagonal span in the 4-way intersections ranging from 28 to 31 ft. All pillar ribs were intact in all roof fall areas with no sign of roof pressure.

Roof Rocks and Coal Property Tests

Coal/roof rock samples were obtained for rock mechanics property tests. The results are:

Coal

uniaxial Compressive Strength (average of 17 tests)	4,773 psi
Young's Modulus	360,000 psi
Poisson's Ratio	0.18
Brazilian tensile strength (average of 8 tests)	308 psi
Bulk density	67 lbs/cu.ft.

Roof

Brazilian tensile strength (perpendicular to laminations)	1,148 psi
Direct shear strength (perpendicular to laminations at zero normal pressure)	5,318 psi
Direct shear strength (parallel to laminations at zero normal pressure)	617 psi
Bulk density	145 lbs/cu.ft.

Overall, the coal strength is very high and the roof rock is also strong, but the shear strength along the laminations is weak. Therefore, it can be concluded that the coal pillars did not contribute to the roof falls.

Roof Bolt Anchorage Capacity Tests

Five roof bolt pullout tests were performed in one small area at #3 entry 50 ft outby the face. Figures 5 and 6 show the applied load v. displacement curves for conventional tension anchor and fully-grouted bolts, respectively. With the exception of bolt #3 which shows anchorage slip at 6 - 7 tons load, all bolts yielded. Therefore anchorage capacity of these two types of bolts exceed 9 and 15 tons, respectively. Note Bolt #1 through #3 are conventional and Bolts #4 and #5 are the fully grouted bolts.

From the test results the following conclusions can be made:

1. The roof strata have a strong anchorage capacity being larger than the bolt yield strength.
2. For conventional bolts, the larger the installed torque (or tension) the less bolt movement regardless of the bolt length.

Analysis of Causes of Massive Roof Falls

Roof falls in Mine B were due to a combination of weak roof strata and resin bolt installation problems. The immediate roof is a typical stack rock which is very weak. The exposed bolts in the roof fall areas showed that the resin bolting system employed was not totally effective due to numerous cases of glove fingering and bare bolts in the bottom 1-2 ft.

Although the estimated interburden thickness between the rider and the mined seam was 6-7 ft, none of the roof fall cavities reached the bottom horizon of the rider seam. Similarly the abandoned mine workings in the overlying seam approximately 100 ft above could not be responsible for the massive roof falls, because roof falls have occurred both directly under, adjacent, or far away from the abandoned mine workings.

Since the mine was developed within a shallow mountain ridge, a high horizontal stress field was not likely. This was corroborated by the fact that rooms developed in the NW direction in Area U (Fig. 4) produced a very bad top.

Recommendations

1. Resin bolt installation procedures should be reviewed thoroughly to eliminate the glove fingering effect and make sure the resin annulus forms along the full length of the bolt.
2. Since all roof falls initiated at the entry/crosscut intersections, the intersection roof span must be kept to the minimum and/or a special roof bolting plan should be employed for intersections.

Follow-up Study

Mine management decided to continue to produce coal by driving a five-entry system parallel but immediately adjacent to the old workings (Fig. 7). They established a couple of test areas, each of which covered three blocks of coal pillars. In this area the 4-ft and 6-ft alternate roof bolting pattern was used in all of the five entries. But the intersections in one entry were supplemented with a 7-ft HyTec bolts at the center of each of the 4x4 ft grids. The results showed no difference in roof behavior. Consequently mine management decided not to use any special bolting plan for the intersections. However measures were taken to make sure that the entry/crosscut intersections were properly driven without excessive spans and the roof bolt installation procedures recommended by the manufacturers strictly followed.

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CONCLUSION

In order to see if a high horizontal stress field exists, a test section was also established (X in Fig. 7) in which all entries were driven in the NE direction and the entry width was reduced from 18 to 16 ft. Roof conditions did not improve. In fact the roof was so bad that when development had reached the extreme end of the section, the section had to be abandoned quickly in order not to lose any equipment, because the entry height had been reduced considerably.

Up until December 1998, the mine officially incurred 96 roof falls. The actual number of roof falls was far more because roof falls in all nonessential entries had not been counted. In other words roof falls occurred continuously over time. Therefore, the weak stack rock roof is the major culprit for which an alternative bolting plan must be used to endure the time factor.

Summary

The weak immediate roof (i.e. stack rock) under an oversized intersection span was the cause of roof falls found in Mine B, not the high horizontal stress..

These two case studies demonstrated that a ground control problem investigation, with or without sophisticated instrumentation techniques and computer modeling, must consider the past history and include a follow-up study to insure successful remedial measures are found. Merely analyzing the existing condition from which remedial measures are derived is absolutely insufficient !

These two cases also demonstrated that the popularly employed 4 ft by 4 ft bolting plan is inadequate for the weak laminated roof strata, especially at the entry /crosscut intersections.

Finally these two cases demonstrated that roof bolting design for the weak laminated roof strata is still very much an art and desperately needs more research to improve it.

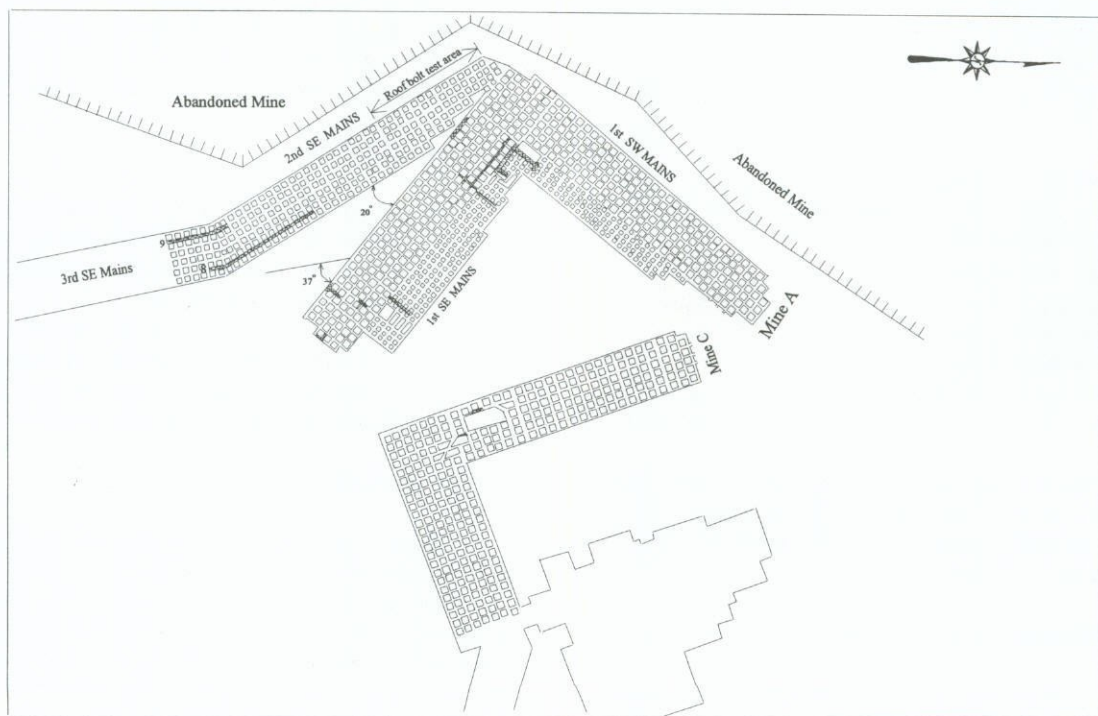


Fig.1 Layout of Mine A

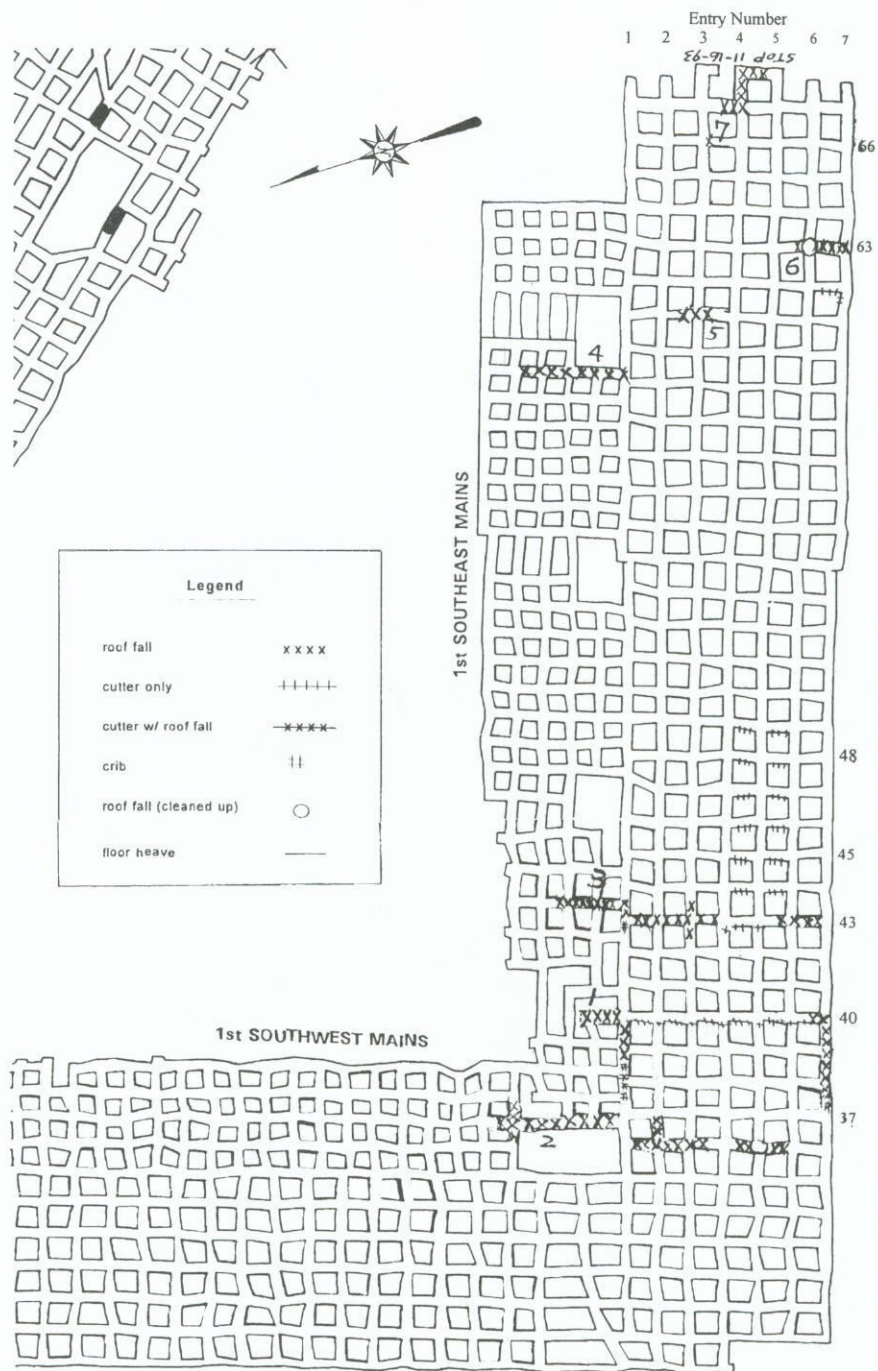


Fig.2 Cutters and roof falls investigated in the 1st Southeast Mains
(The numerical numbers indicate the sequence of occurrence of roof falls)



Fig.3 Surface topograph over the mined area, Mine A

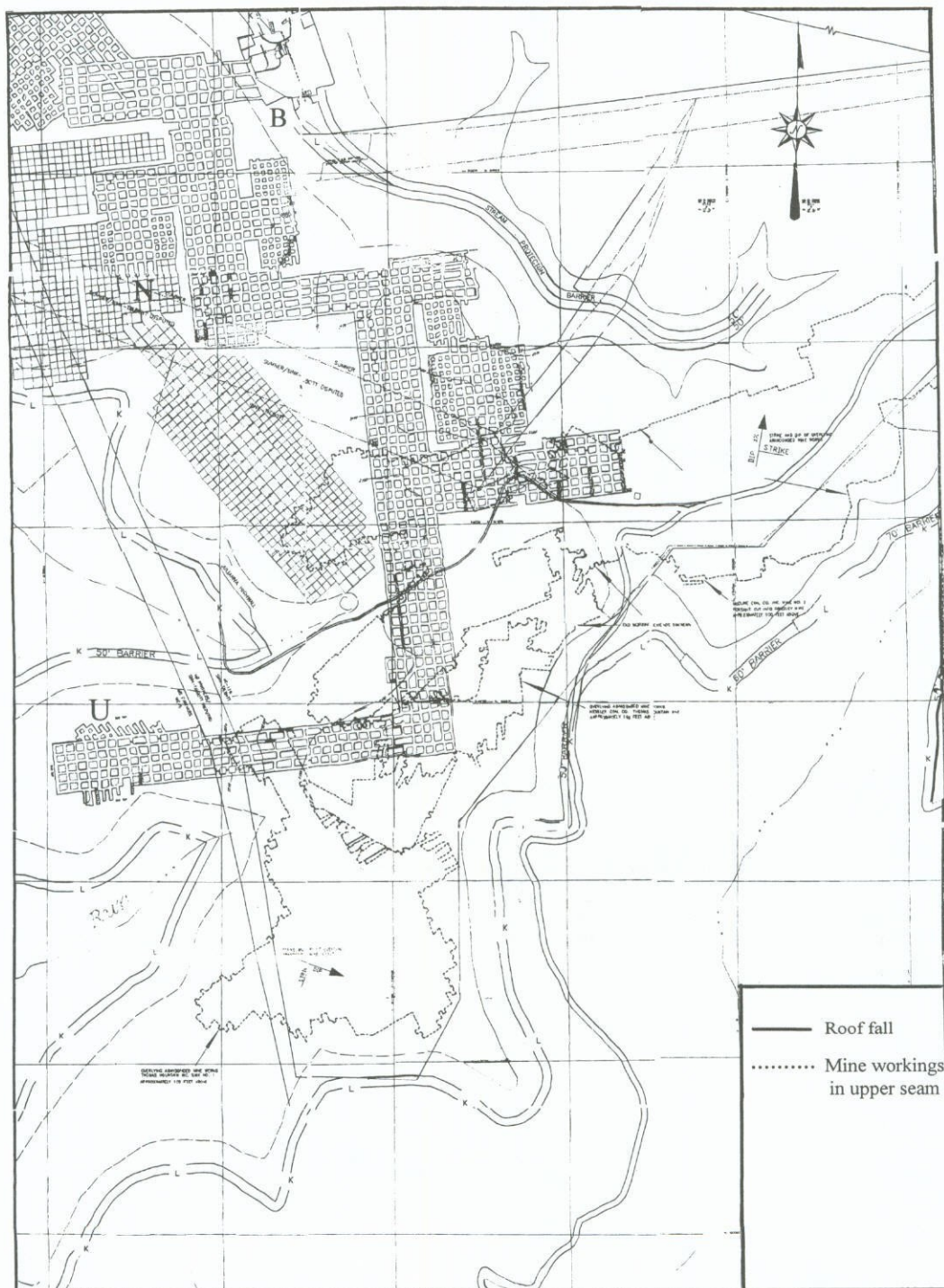


Fig.4 Mine B Layout in 1996

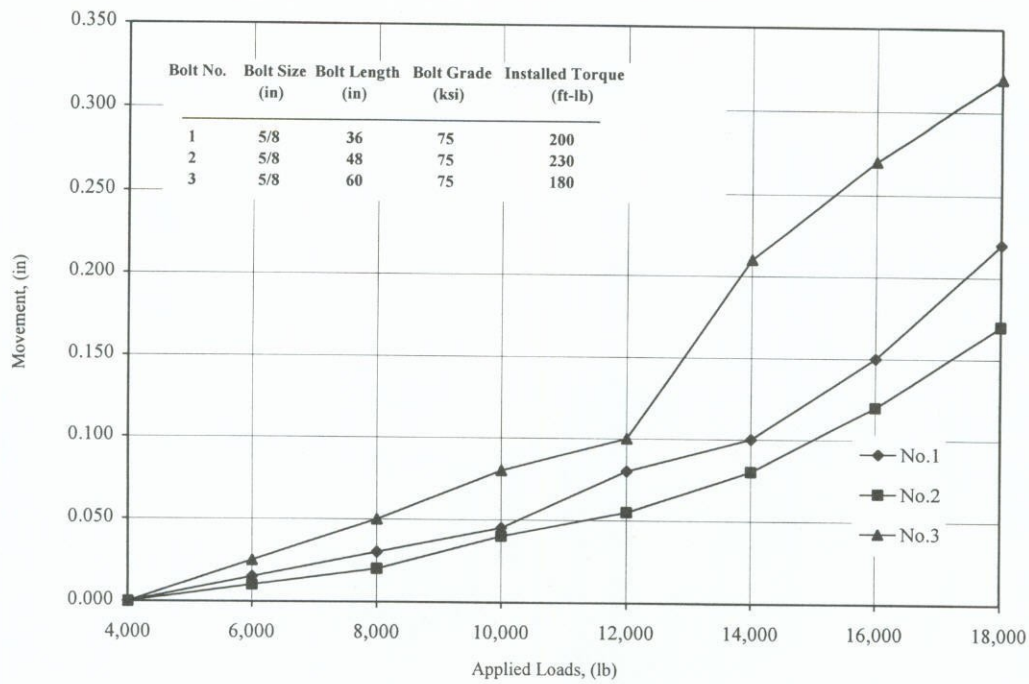


Figure 5 Pullout Tests Results for Conventional (mechanical-anchor) Bolts, Mine B

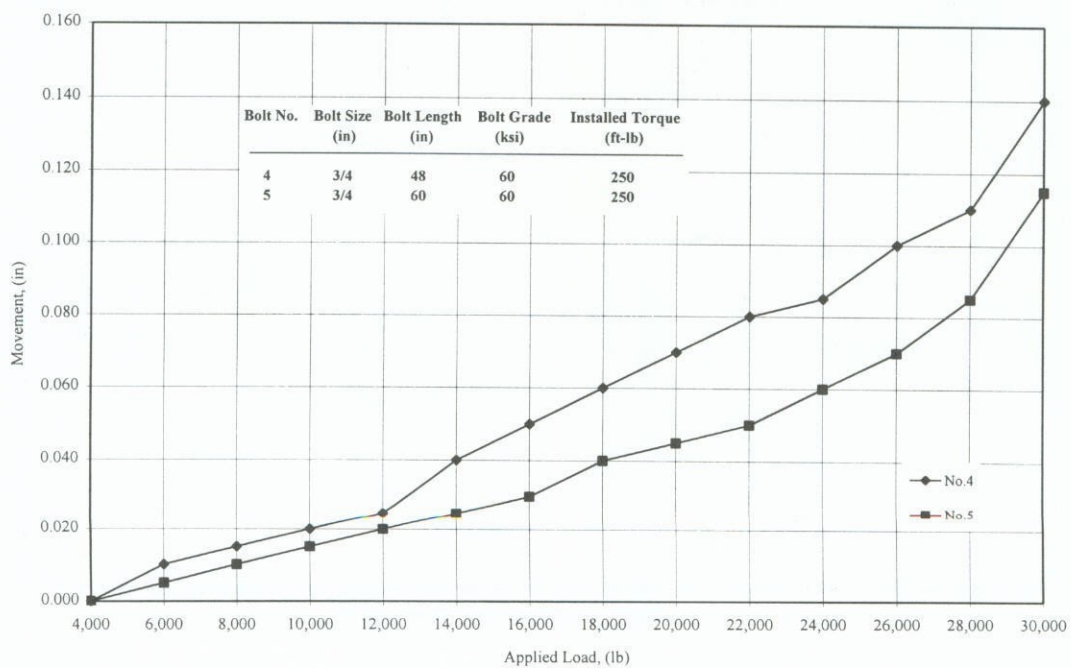


Figure 6 Pullout Tests Results for Folly-grouted Resin Bolts, Mine B

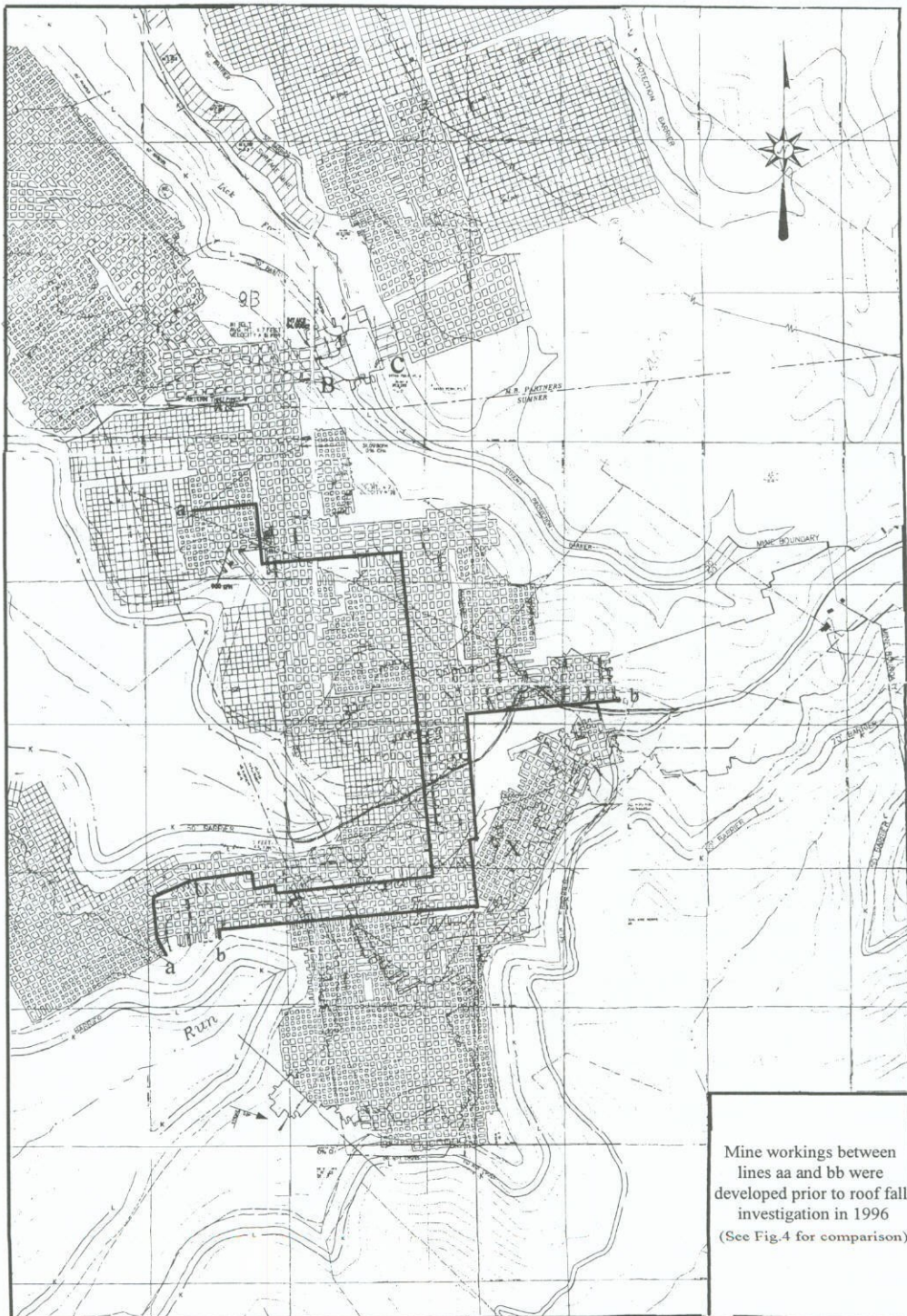


Fig.7 Mine B Layout in 1998



Catastrophic Collapse of Highwall Web Pillars and Preventative Design Measures

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ABSTRACT

In highwall mining, once a sufficient number of web pillars are developed, they can fail catastrophically in a domino-type failure, posing a safety hazard to highwall miners and sterilizing mineable reserves. Three recent case histories of catastrophic web pillar failure in U.S. and Australian mines are documented and analyzed with two design approaches. The conventional design approach uses only a strength criterion, while the postfailure design approach uses both strength and stability criteria to insure an acceptable failure mode should strength failure occur. Catastrophic highwall web pillar failures can cost money through equipment losses, production losses, coal reserve sterilization and lost opportunity costs. Coal recovery may be higher, but the benefits of higher recovery must balance the potential costs of failure. Application of the conventional or postfailure design approach will decrease coal recovery and revenues, but the risk of potentially costly failure is much less.

DOMINO-TYPE FAILURES IN HIGHWALL MINING

In typical highwall mining systems, various mining machines excavate a series of openings that are separated by narrow web pillars. Unfortunately, these layouts can fail catastrophically via a domino-type failure mechanism if not designed properly. One pillar fails which immediately overloads the neighboring pillars causing them to fail and so forth until all pillars in the layout have failed. Figure 1 shows plan views of two highwall mining layouts that apparently failed via this mechanism.

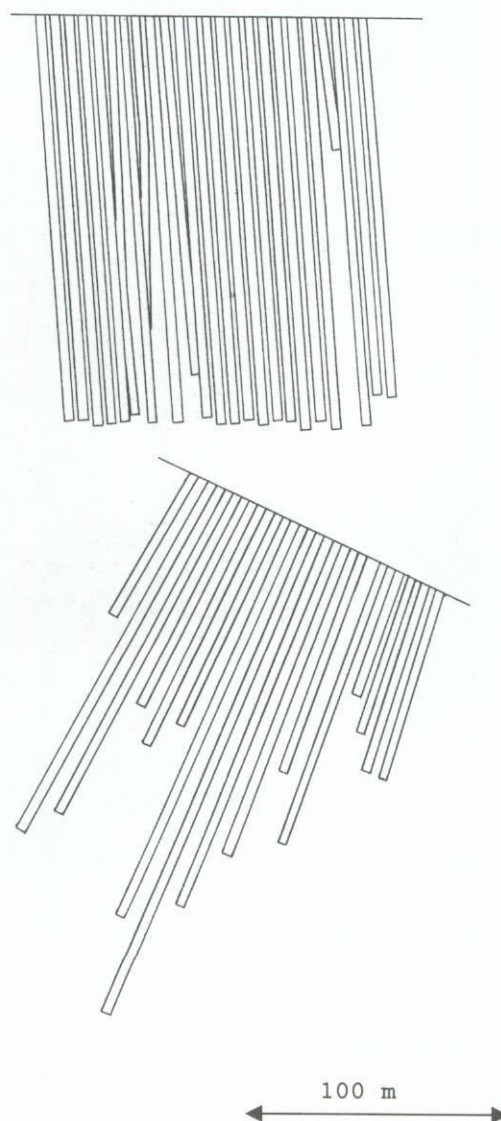


Figure 1 - Plan views of highwall mining layouts where web pillars collapsed. Case A top and case C bottom. (After 1-2)

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Several safety hazards arise from such failures. The overlying strata subside suddenly, which may induce slope failure of the highwall itself, which may in turn harm miners and their equipment. Figure 2 shows a highwall that apparently failed due to web pillar collapse. The rockfall material that originated in the upper parts of the slope was sufficient to bury a coal haulage vehicle.

Another safety hazard is that rapid collapse of the openings can expel rock from these holes with the force of a cannon blast as noted by Willis (3) and Boyd (4). Properly sized and spaced barrier pillars created by not mining select holes in the layout or by backfilling of some holes are the only means to diminish the risk of domino-type failures of web pillars and the associated safety hazards.

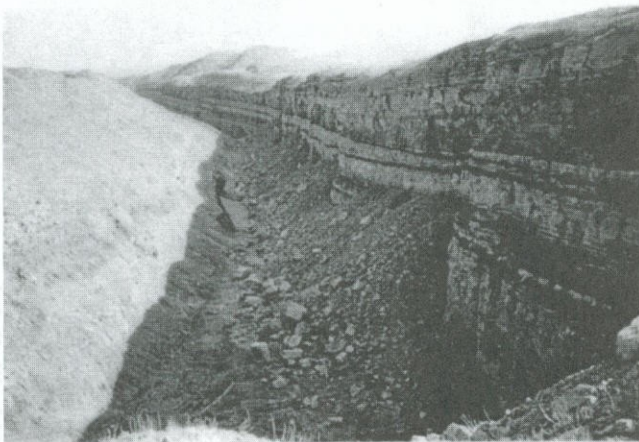


Figure 2 - Failed highwall slope resulting from sudden web pillar collapse.

As summarized in Table 1, several classic domino-type failures of web pillars in highwall mining have occurred in the past few years with expensive consequences. In two cases, mining machinery was severely damaged or lost as a result of highwall slope failure induced by the failure. Fortunately, no mining personnel were injured in any failure occurrence.

Table 1 - Mining Geometry for Domino-Type Highwall Web Pillar Failures

Case	A	B	C
Hole Width (m)	3.66	1.30	3.50
Web Width (m)	1.80	0.80	3.50
Web Height (m)	3.00	1.30	2.00
Depth (m)	60 to 100 4-10° dip	30 to 40	50 to 75 8-10° dip
Estimated # Failed Pillars	75	45	14
Remarks	120 ton truck lost, loader damaged, no injuries	No damage	Highwall mining system buried for several weeks

Case A Description

This failure occurred in 1995 when the highwall mining system was finishing up at one end of the pit. Evidently, the webs between holes failed suddenly which caused about 400 meters of the highwall to partially fail. Looser material in the upper 15 meters of the 50-meter-high highwall fell into the pit. A large triangular shaped wedge of debris formed on the pit bottom (figure 2). Parts of the highwall mining system were hit; a large (about 4-cubic-meter) loader was damaged beyond repair, and a large (about 110 tonne) coal hauler was buried completely and never recovered. The operating crew was on break, and there were no injuries.

There is some debate as to whether the failure happened suddenly within a few seconds or slowly over about an hour. The large amount of rockfall from the highwall suggests strongly that the web failure happened catastrophically.

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Case B Description

This failure happened in 1991 at an auger mining operation (3). Over a few hours time, 46 holes were crushed, and the entire highwall settled. While the overall failure occurred over several hours, individual web pillars failed explosively. The rapidly collapsing auger holes expelled debris like a cannon blast. Fortunately there were no injuries or equipment damage.

Case C Description

This failure happened in 1997 (4). Evidently, it was very similar to case A. Witnesses described how puffs of smoke emanated sequentially from the highwall holes. Failure of the highwall slope, induced by the domino-type web pillar failure, buried the highwall mining system for several weeks and caused considerable equipment damage. Fortunately, there were no injuries.

ANALYSIS METHOD

Strength and Stress Analysis

In designing a highwall mining layout, the mining engineer must specify the opening spans, the width of the web pillars, the number of web pillars which determines the spacing between barrier pillars, and finally, the width of barrier pillars. Normally, size of the highwall mining equipment dictates the required stable opening span, and that issue is beyond the scope of this paper.

Mark and Iannacchione (5) review many common formulas and present one that is especially applicable to highwall web pillars which is

$$S_p = S_i[0.64 + 0.54 W/H - 0.18 W^2/(LH)] \quad (1)$$

where

S_p = web/barrier pillar strength for $W/H < 5$

S_i = in situ coal strength

W = web or barrier pillar width

H = web or barrier pillar height

L = web or barrier pillar length.

Beyond a W/H ratio of 5, Salamon and Wagner (6) developed a "squat pillar" formula as

$$S_p = S_i(W_0/H_0)^b/V^a\{b/e[((W/H)/(W_0/H_0))^c-1]+1\} \quad (2)$$

where

W_0/H_0 = critical W/H ratio = 5

V = web or barrier pillar volume = W^2H

$e = 2.5$

$b = 0.5933$

$a = 0.0667$.

Based on tributary area theory, average vertical stress on a web pillar is

$$S_{wp} = S_v(W_{wp} + W_e)/W_{wp} \quad (3)$$

where

W_{wp} = web pillar width

W_e = entry width

S_v = vertical stress computed as γD

D = depth.

Tributary area theory is useful because it usually provides a conservative, worst-case estimate of the applied stresses on a web pillar, unless nearby web pillar failures have led to more complex stress redistribution.

If barrier pillars are left after every "N" web pillars, then the spacing between barrier pillars (outer edge to outer edge) is given by

$$W_{PN} = N(W_{wp} + W_e) + W_e \quad (4)$$

Should each of these N web pillars fail, then the stresses carried by these pillars must be born by the adjacent barrier pillars. Again based on tributary area theory, maximum vertical stress on one of the barrier pillars is

$$S_{BP} = S_v(W_{PN} + W_{BP})/W_{BP} \quad (5)$$

where

W_{PN} = spacing between barrier pillars

W_{BP} = barrier pillar width.

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Postfailure Pillar Behavior

Once a pillar has reached peak strength, its load carrying capacity will begin to decrease as it fails. It is important to understand whether the failure will proceed in a steady, progressive manner or whether the pillars will collapse catastrophically. The local mine stiffness stability criterion makes this determination (7).

Stable, nonviolent pillar failure occurs when

$$|K_{LMS}| > |K_p| \quad (6)$$

Unstable, violent pillar failure occurs when

$$|K_{LMS}| < |K_p| \quad (7)$$

K_{LMS} is the local mine stiffness, and K_p is the post-failure stiffness at any point along the load-convergence curve for the pillar. If the local mine stiffness is less steep than the post-failure stiffness for the pillar, then the pillar fails violently, and if the local mine stiffness has a steeper slope than the post-failure stiffness, then pillar failure occurs in a stable nonviolent manner.

In other studies on this topic, Zipf (8) proposed the following approximate relationship for the full-scale post-failure modulus of coal pillars:

$$E_p(\text{MPa}) = -1750(W/H)^{-1} + 437 \quad (8)$$

Assuming a unit width for the pillar, the post-failure stiffness is related to the post-failure modulus as:

$$K_p = E_p(W/H) \quad (9)$$

or

$$K_p(\text{MN/m}) = -1750 + 437(W/H) \quad (10)$$

The other factor in the stability criterion is the stiffness of the rock mass surrounding a system of mine pillars. The local mine stiffness depends on the modulus of the rock mass and the geometry of the mining excavations. It is evaluated via numerical or analytic methods. Based on such studies (8), the following simple approximate expression is proposed for a practical design methodology:

$$K_{LMS}/(E W_{cc}) = -0.20(3/N)^{1.5} \quad (11)$$

where

E = rock mass modulus,

N = number of web pillars in each panel,

W_{cc} = center to center spacing between web pillars.

This approximate expression should only be applied in a relatively homogeneous rock mass away from the structural influence of major geologic discontinuities. In the worst case, K_{LMS} becomes zero and violent, unstable failure always results. This situation is equivalent to full dead-weight loading of the web pillars with no bridging of the overlying rock mass.

SPREADSHEET IMPLEMENTATION

The analysis method described above is implemented into a simple spreadsheet shown in the appendix and utilized in the back analyses and subsequent redesigns.

Initial information includes the entry width (hole diameter), entry height, depth below surface, and vertical stress gradient. Normally the user can design around a depth somewhat less than the maximum depth, since the "three-dimensional" effects at the back of the excavations will tend to shield the web pillars from full tributary area load.

Web pillar strength is determined empirically via equation 1 given appropriate values for the in situ coal strength and the constants a and b . For long web pillars in U.S. coals, 6.2 MPa, 0.64 and 0.54 may be good starting values. Mean pillar strength and its standard deviation are evaluated assuming that the coal strength and pillar width are normally-distributed random variables with known mean and standard deviation.

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Web pillar stress is evaluated via equation 3. Assuming that the depth is a randomly distributed variable, an estimate of the standard deviation on web pillar stress is calculated. The effective depth ratio provides a convenient way to adjust the vertical stress estimate if bridging or another form of stress shielding of the web pillars is occurring, such as near the end of holes. Normally, the effective depth ratio is 1.

Safety factor is the web pillar strength divided by the web pillar stress. Mean safety margin and its standard deviation is computed from the difference between web pillar strength and stress. The statistical parameter Z is the ratio of mean safety margin to its standard deviation. Estimates of the probability of pillar failure follow directly from Z and statistical functions. A safety factor of 1 corresponds to a failure probability of 50%.

The design number of web pillars determines the panel width. The post failure stiffness is calculated from the pillar geometry via equation 10, and the local mine stiffness is determined from the panel width and number of web pillars via equation 11. Finally, the nature of web pillar failure (stable or unstable) is assessed via application of the local mine stiffness stability criterion (equations 6 and 7).

Barrier pillar strength is computed with equation 2 for pillars with W/H ratios greater than 5. Mean barrier pillar strength and its standard deviation follows from the statistics of the coal strength and the barrier pillar width. Mean barrier pillar stress is calculated with equation 5, and its standard deviation follows from the statistics of the depth and barrier pillar width. Safety factor and probability of failure for the barrier pillar is computed similarly to the web pillars. Post failure stiffness for the barrier pillar is also calculated with equation 10. Local mine stiffness is determined via equation 11 using barrier pillar and panel width and assuming that the number of barrier pillars is 6. This assumption may be a reasonable upper limit on the number of panels mined from any particular highwall block.

BACK ANALYSIS OF FAILURES

The simple spreadsheet is used to back-analyze the case histories. Figure 3 shows the idealized geometry used for the back-analyses. Table 2 summarizes the results.

Table 2 - Back-Analysis of Highwall Web Pillar Failure Case Studies with Pillar Geometry, Safety Factor and Probability of Failure.

WEB PILLAR	Case A	Case B	Case C
Entry Width (m)	3.660	1.300	3.500
Web Height (m)	3.000	1.300	2.000
Web Width (m)	1.830	0.800	3.500
Safety Factor	1.002	1.125	1.089
Prob. of Failure	0.492	0.137	0.159
WEB PILLAR STABILITY			
# Web Pillars	25	45	14
K_{LMS} (MN/m)	-183	-29	-555
K_p (MN/m)	-1483	-1481	-985
Failure Nature	violent	violent	violent

The following major assumptions enter this back analysis:

- 1) Web pillar width, entry width and depth are extracted from table 1.
- 2) Coal strengths of 6.2 and 3 MPa with a standard deviation of 10 % are used for U.S. and Australian coals, respectively.
- 3) Standard deviation of the critical depth, which ultimately governs the statistical distribution of applied pillar stress, is 3.5%.

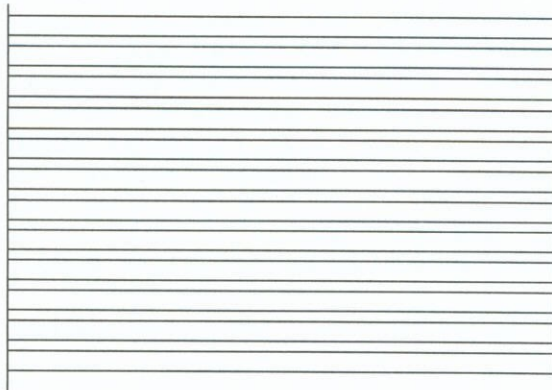
Back analysis suggests safety factors near 1 with high probability of failure. The above cases also violate the local mine stiffness stability criterion, which leads to a rapid unstable collapse of a large number of web pillars once failure begins.

Clear guidelines or regulations determining the frequency of barrier pillars are generally lacking. At some highwall mining operations, barrier pillars are spaced every 20 to 25 web pillars, and in some situations in the US, as few as every 15 web pillars. In Australia, barriers are usually avoided.

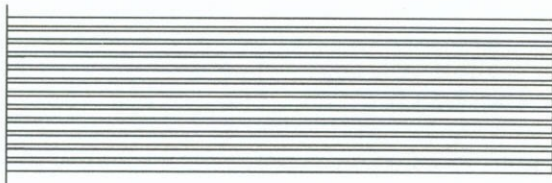
Barrier pillars serve several functions. First, they can arrest a domino-type web pillar failure and keep it contained within one panel should a failure get started.

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FAILED LAYOUT SCHEMATIC - CASE A



FAILED LAYOUT SCHEMATIC - CASE B



FAILED LAYOUT SCHEMATIC - CASE C

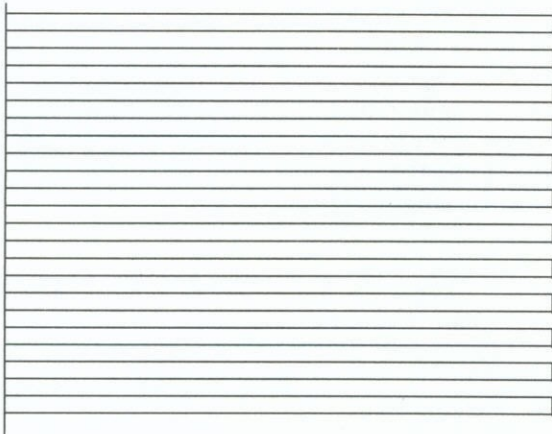


Figure 3 - Idealized layouts of failed case histories used in back-analysis.

Case A - entry width - 3.67 m; web width - 1.83 m; no barrier pillars
 Case B - entry width - 1.30 m; web width - 0.80 m; no barrier pillars
 Case C - entry width - 3.50 m; web width - 3.50 m; no barrier pillars

Second, they can limit the overburden stresses on the web pillars and thereby increase their safety factor. Most important though, barrier pillars stiffen the loading system and thereby prevent the possibility of catastrophic collapse. Barrier pillars must have adequate strength to withstand the additional stresses imposed on them by the failure of all web pillars within a panel, and they should also have an acceptable failure mode, i.e. steady and nonviolent.

Probability of failure considerations may limit the number of web pillars in a panel. Figure 4 shows how the overall probability of failure for the panel increases as the number of web pillars increases. Very simplistically, given a probability of failure for one pillar p_f , the probability of failure for N pillars is

$$P_f = 1 - (1 - p_f)^N \quad (12)$$

This relation assumes that applied stresses and strengths of all pillars in the panel are the same and that the pillar failures are independent events. In the absence of barrier pillars, preventing catastrophic failure of highwall mining layouts may require sizing web pillars with a safety factor of 1.5 or more and limiting web pillar numbers per panel to less than about 10.

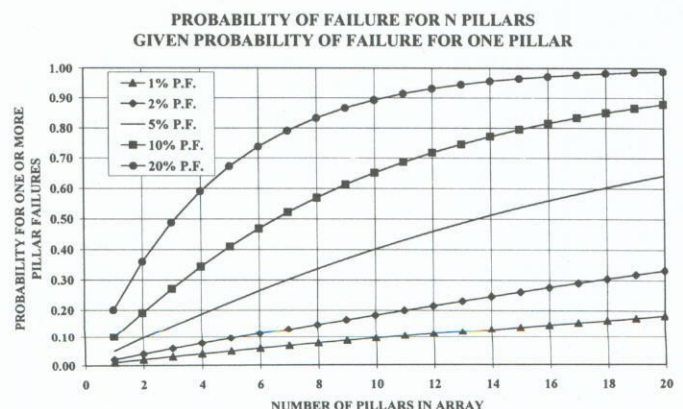


Figure 4 - Probability of failure of N pillars given probability of failure of one pillar.

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DESIGN METHODS

Conventional Strength-Based Design Method

In this approach, the applied stresses must never exceed the strength of a structure. In evaluating a design, the engineer can begin by considering the safety factor (strength/stress). Designing on the basis of a probability of failure that addresses the economic risks and consequences of failure is ultimately a better approach when possible.

Table 3 and figure 5 show the re-design of the three case histories using a conventional strength criterion design method. A risk of web pillar failure of about 10% is assumed economically acceptable, which leads to safety factors of about 1.1. Required web pillar widths increase slightly from the failed cases presented in table 2. Substantial barrier pillars are added. The number of web pillars in the panel is such that the overall probability of web pillar failure in the panel as given by equation 12 is less than about 50%. Since the barrier pillar design makes the conservative assumption that all web pillars in the panel have failed, a liberal safety factor of 1 is allowed for the barrier pillars.

Table 3 - Conventional Design Approach for Highwall Web Pillars

WEB PILLAR	Case A	Case B	Case C
Entry Width (m)	3.660	1.300	3.500
Web Height (m)	3.000	1.300	2.000
Web Width (m)	2.000	0.800	3.500
Safety Factor	1.095	1.125	1.089
Prob. of Failure	0.151	0.137	0.159
WEB PILLAR STABILITY			
# of Web Pillars	7	5	10
K_{LMS} (MN/m)	-1270	-781	-920
K_p (MN/m)	-1459	-1481	-985
Failure Nature	violent	violent	violent
BARRIER PILLAR			
BP Width (m)	8.000	3.000	15.500
Safety Factor	1.006	1.022	1.024
Prob. of Failure	0.476	0.430	0.412
BARRIER STABILITY			
K_{LMS} (MN/m)	-6741	-1946	-11699
K_p (MN/m)	-585	-742	1637
Failure Nature	stable	stable	stable

The design is based solely on strength criterion, and the nature of pillar failure is not considered. A violent failure could occur; however, its extent is limited by the panel size. The local mine stiffness stability criterion has been evaluated, and as shown, the web pillars in most cases violate the local mine stiffness stability criterion. The barrier pillar design satisfies the stability criterion in all cases, and any failure of this structure proceeds in a stable, nonviolent fashion.

Post-Failure Analysis and Design Method

In a post-failure analysis and design method (table 4 and figure 6), the desired failure characteristics of the pillars are selected first, which then leads to a pillar width. Number of pillars and then panel width follow. The design procedure sizes a pillar on the basis of both strength and failure characteristics. The strength criterion assures that the pillar performs its load bearing duties at the required time. When that performance is no longer required, the stability criterion assures that the pillars fail in a stable progressive manner with little risk to the mining equipment and no danger to the operating personnel. Because the failure mode is acceptable, lower safety factor (higher probability of failure) for the web pillars is possible.

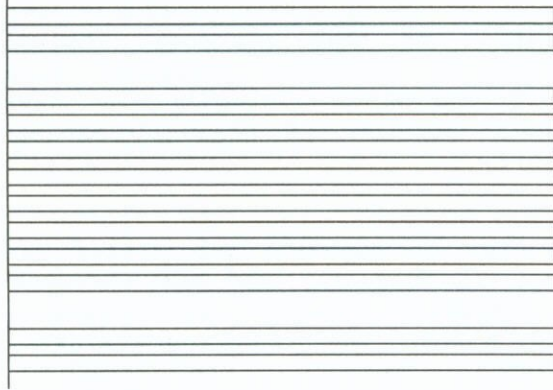
Table 4 - Post-Failure Design Approach for Highwall Web Pillars

WEB PILLAR	Case A	Case B	Case C
Entry Width (m)	3.660	1.300	3.500
Web Height (m)	3.000	1.300	2.000
Web Width (m)	1.850	0.700	3.250
Safety Factor	1.013	1.001	1.011
Prob. of Failure	0.442	0.498	0.447
WEB PILLAR STABILITY			
# Web Pillars	6	3	8
K_{LMS} (MN/m)	-1558	-1600	-1240
K_p (MN/m)	-1481	-1515	-1040
Failure Nature	Stable	stable	stable
BARRIER PILLAR			
BP Width (m)	7.300	2.250	13.500
Safety Factor	1.005	1.020	1.001
Prob. of Failure	0.481	0.433	0.497
BARRIER STABILITY			
K_{LMS} (MN/m)	-5787	-1255	-9333
K_p (MN/m)	-687	-994	1200
Failure Nature	stable	stable	stable

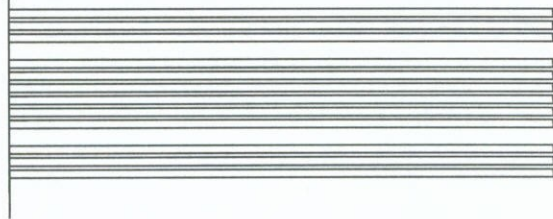
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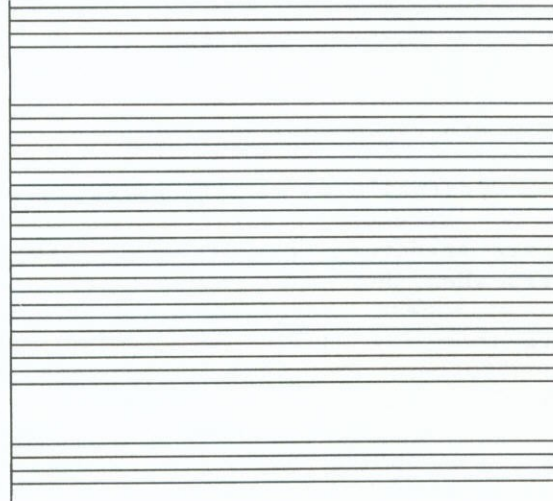
CONVENTIONAL RE-DESIGN - CASE A



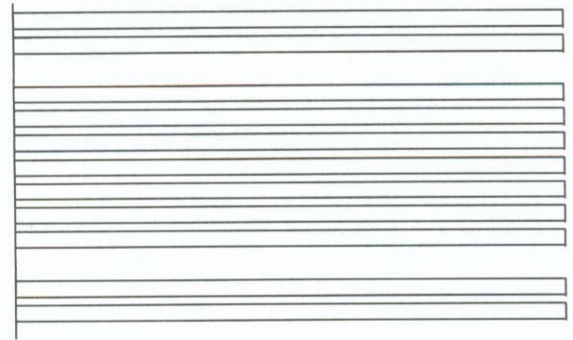
CONVENTIONAL RE-DESIGN - CASE B



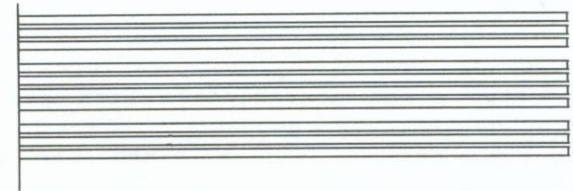
CONVENTIONAL RE-DESIGN - CASE C



POST-FAILURE RE-DESIGN - CASE A



POST-FAILURE RE-DESIGN - CASE B



POST-FAILURE RE-DESIGN - CASE C



Figure 5 - Schematic layouts of conventional re-designs.

Case A - entry width - 3.67 m; web width - 2.00 m; barrier width - 8.0 m; 7 webs
 Case B - entry width - 1.30 m; web width - 0.80 m; barrier width - 3.0 m; 5 webs
 Case C - entry width - 3.50 m; web width - 3.50 m; barrier width - 15.5m; 10 webs

Figure 6 - Schematic layouts of post-failure re-designs.

Case A - entry width - 3.67 m; web width - 1.85 m; barrier width - 7.3 m; 6 webs
 Case B - entry width - 1.30 m; web width - 0.70 m; barrier width - 2.25 m; 3 webs
 Case C - entry width - 3.50 m; web width - 3.25 m; barrier width - 13.5m; 8 webs

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DESIGN METHODS COMPARED

Figure 7 compares the geometry of the failed cases to design results applying the conventional and post failure method. In these re-designs, entry width and height remain constant. The number of web pillars in a panel decreases dramatically (a factor of about 9 in one case). Application of the post-failure design approach leads to further decreases in the number of web pillars in a panel.

Web pillar width does not change much in the re-designs. For the conventional re-design, it remains the same or increases slightly compared to the failed case histories. With the post-failure re-design, web width actually decreases somewhat, because of the acceptable failure mode (i.e. steady and nonviolent). Both design approaches utilize barrier pillars, and generally, barrier pillar width for the post-failure design is less than that of the conventional design due to the lower number of web pillars.

Compared to the failed cases, safety factor of the web pillars using the conventional approach increases or remains the same. It actually decreases using the post-failure design approach due to the acceptable failure mode. Again, the distinguishing feature of the post-failure design approach is the stable nature of failure once the strength criterion is exceeded.

The most important design consideration is economics. Use of either the conventional or post-failure design approach leads to a decrease in coal recovery on the order of 10 to 25%. The significance of such changes on project economics is not clear. Higher recovery methods risk failure with potential costs including equipment loss, reserve sterilization and most important safety. The potential additional coal revenues must exceed the risk of incurring these potential costs.

CONCLUSION

A strength criterion governs whether a pillar can successfully resist the imposed load. Once the strength criterion is exceeded, the local mine stiffness stability criterion (7) governs whether

failure proceeds in a stable, progressive fashion or in an unstable, violent manner. Violation of this criterion explains a variety of unstable collapses seen in underground mines including the unstable, violent collapse of highwall mining web pillars.

Back-analysis of the three case histories of highwall web pillar failure (Table 2) suggests that the strength and the stability criteria are violated. In re-designing the case histories two different methods are used. The conventional design approach (Table 3) uses only a strength criterion. The number of web pillars in the panel is limited by an acceptable overall probability of failure for all web pillars in the panel. Barrier pillar width is determined assuming that all web pillars in the panel have failed. The nature of web pillar failure does not enter the design procedure. Pillars could fail violently, and the main safeguard against such an event is an adequate safety factor or low probability of failure. Applying the local mine stiffness stability criterion to the re-design shows that the web pillars could fail violently.

The postfailure design approach uses both a strength criterion to insure adequate structural performance of the web pillars when they are needed and the local mine stiffness stability criterion to insure that any failure which might occur is nonviolent. An ideal design has both marginally acceptable strength and marginally acceptable stability. Table 4 shows a re-design using the postfailure design approach. Both web pillar width and the number of web pillars in a panel are adjusted to achieve an ideal design. In this approach the risk of collapse is even lower since the pillars are designed to fail in a stable, progressive manner and not in an unstable, violent manner.

Relative to the failed designs, the conventional and post-failure design methods can have a substantial impact on highwall mining revenues. Preliminary estimates show that total recovery with the conventional strength-criterion-based design approach decreases by 10 to 15%. Using a postfailure design approach leads to additional coal recovery and revenue decreases. The benefit from these approaches is increased safety and

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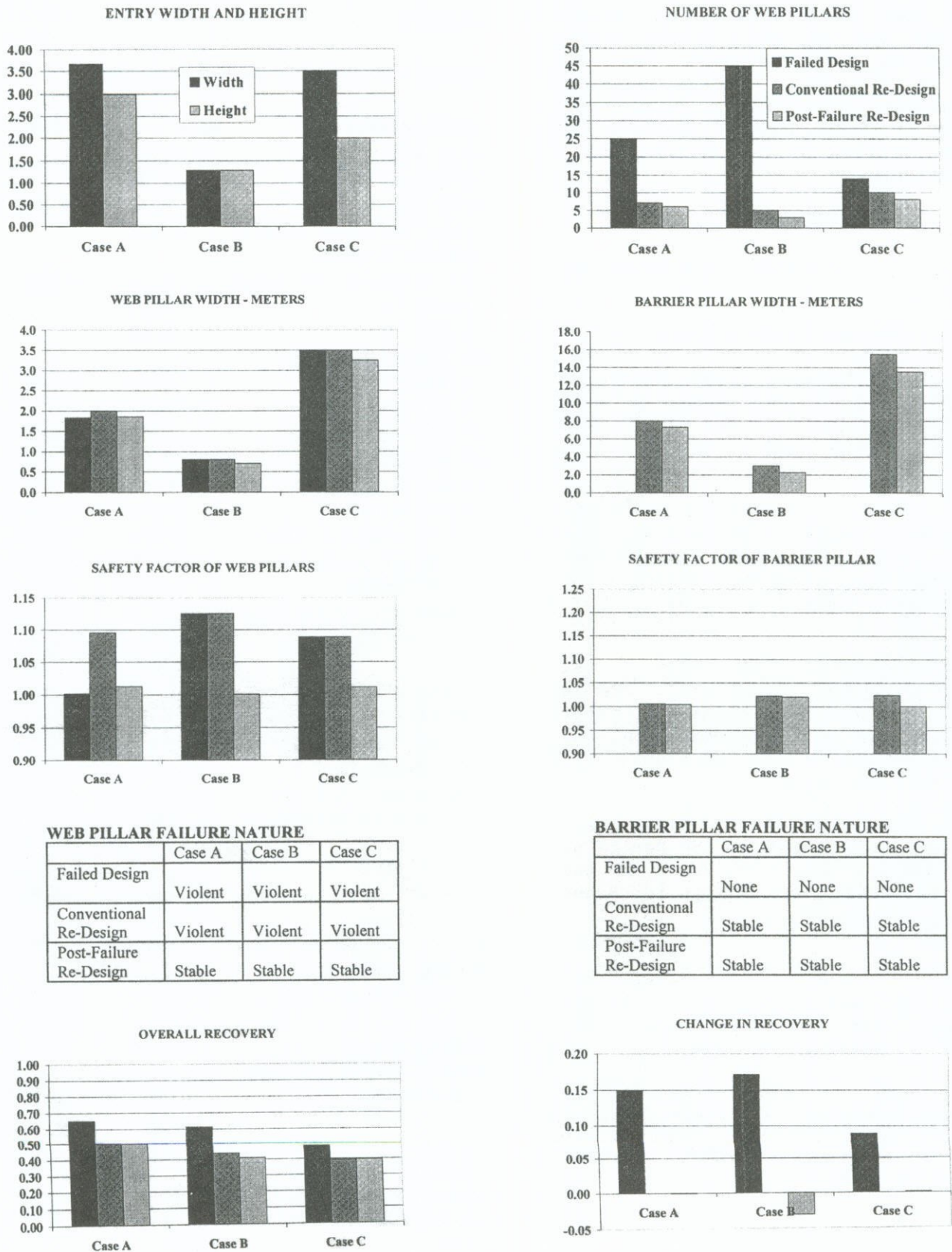


Figure 7 - Comparison of Failed Designs with Conventional and Post-Failure Re-Designs

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decreased risk of equipment and reserve losses. The use of wider rooms during retreat mining could increase the coal recovery if the proper mining equipment were developed. Conducting a proper risk assessment is recommended to evaluate the costs and benefits of implementing these design procedures.

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APPENDIX

Highwall Web Pillar Failure Analysis

FIXED GEOMETRY	Mean	Std. Dev.
Entry Width (m)	3.66	
Web Height (m)	3.00	
Depth (m)	80.0	2.80
Stress Gradient (MPa/m)	0.025	
STRENGTH FORMULA		
Coal Strength 1m3 (MPa)	6.20	0.620
Constant A	0.64	
Constant B	0.54	
WEB PILLAR STRENGTH		
Web Width (m)	2.05	0.103
W/H Ratio	0.68	
Web Pillar Strength	6.26	0.472
WEB PILLAR STRESS		
Effective Depth Ratio	1.00	
Effective Depth	80.0	2.80
Vertical Stress (MPa)	2.00	0.070
Web Pillar Stress	5.57	0.229
SAFETY FACTOR	1.123	
PROBABILITY OF FAILURE		
Safety Margin (MPa)	0.685	0.525
Z	1.305	
Probability of Failure	0.096	
STABILITY OF FAILURE		
Number of Web Pillars	7	
Panel Width (m)	43.6	
Local Mine Stiffness	-1282	
Post-Failure Stiffness	-1451	
Nature	violent	
BARRIER PILLAR STRENGTH		
BP Width (m)	8.00	0.400
BP W/H Ratio	2.67	
Barrier Pillar Strength	12.90	1.075
BARRIER PILLAR STRESS		
Vertical Stress (MPa)	2.00	0.070
Barrier Pillar Stress	12.91	0.669
SAFETY FACTOR	0.999	
PROBABILITY OF FAILURE		
Safety Margin (MPa)	-0.011	1.266
Z	-0.009	
Probability of Failure	0.504	
STABILITY OF FAILURE		
Local Mine Stiffness	-6787	
Post-Failure Stiffness	-585	
Nature	stable	



Mine Panel Collapse - Two Case Studies

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ABSTRACT

A mine panel collapse may occur when pillar sizes are too small or the surrounding rock strata (roof or floor) yields. After a small pillar fails, its loading is rapidly transferred to adjacent pillars that in turn fail. If the surrounding rock strata yield, the loading on pillars will redistribute, which may result in some pillars under high stress. This paper analyzes the causes of two cases of mine panels failures in two room and pillar mines. One case of massive pillar failure occurred in a phosphate mine and the other, mine panels failure, is in a coal mine. In the phosphate mine, both the roof and floor strata were a thick competent shale stratum. The finite element analysis shows that the major reason for the massive pillar failure is that the pillar sizes were too small in a panel. In addition, the high cliff topography has some influence on the failure. In the coal mine, the failed area was about 1,000 ft long by 760 ft wide (covering more than 3 panels) involving 390 pillars. The barrier pillars between the panels were punched on retreat into small chain pillars and the extraction ratio in this failed area was more than 80%. The roof remained intact, and pillars were crushed and punched into the floor. Based on the stress analysis, it is found that multiple panel collapse occurred because the barrier pillars were punched into small pillars and the loading on panel edges increased due to floor failure.

INTRODUCTION

A massive pillar collapse is a type of catastrophic failure in underground mines. Generally, it occurs when the roof rock is competent. In this study, two mine panel failure cases have been analyzed; one occurred in a phosphate mine and the other in a coal mine. In both mines the roof strata were very competent. But, the causes of failures are different in these two mines. This paper describes these two cases of massive failure and analyzes the causes of failures by using the finite element method.

PHOSPHATE MINE

1. Geological and Mining Conditions

The phosphate mine was extracting the 1.6 ~ 1.9 m (5.2 ~ 6.2 ft) thick phosphate bed. The near-seam geological conditions are shown in Fig. 1. The immediate roof, 0.9 ~ 1.3 m (3 ~ 4.3 ft) thick, is composed of sequence of siltstone, phosphate shale and siltstone. The main roof is shale, about 2.4 m (7.9 ft) thick. Their rock mechanics properties are shown in Table 1. The uniaxial compressive strength of phosphate is about 7.4 MPa (1,000 psi) while the roof and floor uniaxial compressive strengths are about 24 MPa (3,500 psi) and 28 MPa (4,000 psi), respectively. In the phosphate mine, a room-and-pillar mining method is used and the width of the room is 6 m (19.8 ft). The roof is self-supporting and requires no reinforcement. The overburden depth ranges from 60 m (197 ft) to 400 m (1,312 ft).

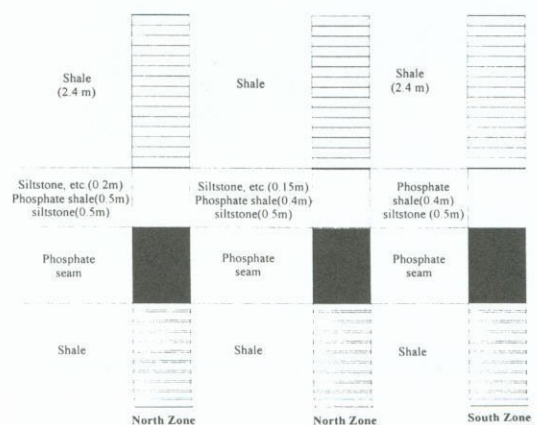


Fig. 1 Near-Seam Lithological Columns in the Phosphate Mine

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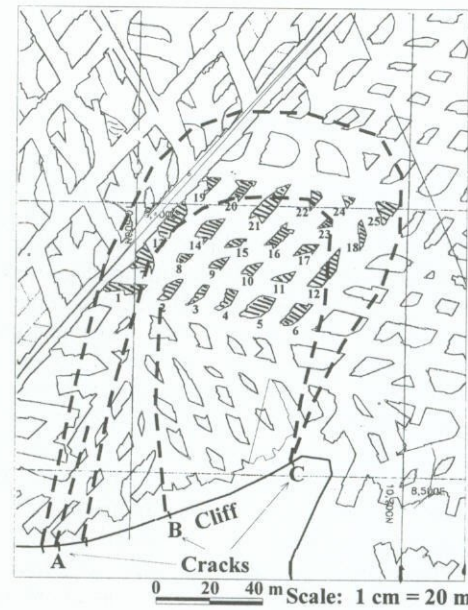
Table 1 Rock Mechanics Properties for Phosphate Mine^[1]

Rock	Density (N/m ³)	Tensile Strength (MPa)	Shear Strength (MPa)	Compressive Strength (MPa)	Young's Modulus (x 10 ³ MPa)	Poisson's Ratio
Phosphate	18,840	1.01	2.03	7.41	0.52	0.3
Sandstone	20,423	3.28	1.96	48.03	2.74	0.23
Phosphate sandstone	19,323	2.59	2.88	32.24	1.62	0.15
Siltstone	19,952	5.22	2.34	24.70	4.08	0.12
Sandy shale	19,009	4.32	2.76	23.20	1.94	0.12
Shale	18,067	5.25	2.31	27.95	3.04	0.08

The pillar failure occurred near the southern boundary of the phosphate mine, as shown in Figs. 2 and 3, about two weeks after this panel was mined out. Because of the competency of the roof and the floor, no caving occurred while the panel was being mined. There were 25 pillars involved in the failure, covering an area about 90 m (295 ft) long by 55 m (180 ft) wide. The pillar sizes are listed in Table 2. Notice all pillar widths were less than 6 m (19.8 ft). Some pillars, such as pillars 22~24, were very small. In this area, the mining height was about 1.65 m (5.4 ft) and the overburden depth was about 100 m (328 ft). After the massive pillar failure, several large cracks occurred on the ground surface, several of which can be observed on the cliff face (Figs. 2 and 4). The mine portals were driven from a high cliff in the southern end of the mine (Fig. 2 and Fig. 4).



(a) Mine Layout



(b) Failure Area

Fig. 2 Pillar Failure Area in Phosphate Mine

Table 2 Pillar Dimensions in the Failure Area

Pillar No.	Pillar Area (m ²)	Pillar Width (m)	Pillar No.	Pillar Area (m ²)	Pillar Width (m)
1	39	2.5	14	40	5.5
2	40	4	15	12.5	2.5
3	24	2	16	35	3.5
4	30	3	17	18	3
5	60	6	18	18	2
6	38	4.5	19	12	2
7	40	2	20	28	3
8	12	3	21	75	5
9	20	3	22	14.5	3
10	20	4	23	8.5	3
11	16	2	24	9	3
12	60	4	25	35	4
13	75	4.5			

Fig. 2 Pillar Failure Area in Phosphate Mine



Fig. 3 Massive Pillar Failure in Phosphate Mine

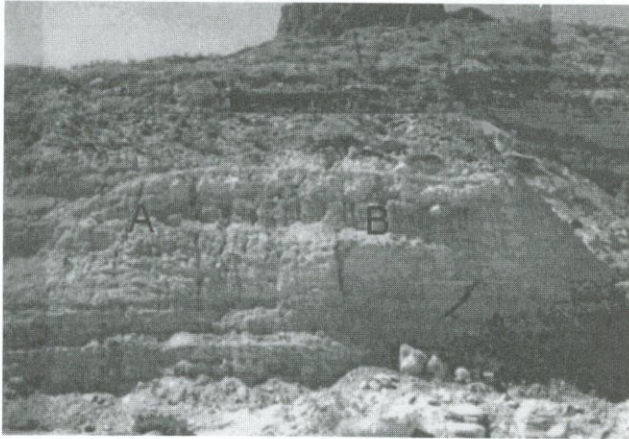


Fig. 4 The Topography in the Pillar Failure Area

2. Finite Element Analysis

The three-dimensional finite element analysis was used to study the phosphate pillar failure. In order to evaluate the stability of pillar, the Mohr-Coulomb failure criterion was used. It is defined by the following equation:

$$[\sigma_1] = \sigma_c + \tan^2(45^\circ + \frac{\alpha}{2}) \times \sigma_3$$

where $[\sigma_1]$ – allowable maximum principal stress;
 σ_3 – minimum principal stress in the finite element;
 σ_c – uniaxial compressive strength of pillar; and
 α – angle of internal friction of phosphate, $\alpha=25^\circ$

The safety factor (SF) is defined by the following equation:

$$SF = \frac{[\sigma_1]}{\sigma_1}$$

where σ_1 – maximum principal stress in the finite element

Pillar stability is evaluated by using its average safety factor. Based on the stress distribution in a pillar, the safety factor of pillar is determined by using the Mohr-Coulomb failure criterion. Fig. 5~7 show the safety factor contours of pil-

lars when the pillar width is 4 m (13 ft), 5 m (16.4 ft), and 6 m (19.8 ft), respectively. Fig. 8 shows the change of average safety with pillar width. The safety factor of pillar is less than 1.0 when the pillar width is less than 6.5 m (21.3 ft). Although the safety factor in the pillar core is larger than 1.0, the yield zone (SF<1.0) of the pillar is large. For example, the core width is only about 1.75 m (5.7 ft), when the pillar width is 4 m (13 ft). In this case, the loading on the yield zone may gradually be transferred to the pillar core, and results in the yield zone becoming larger and larger. Finally, the whole pillar may yield. From the safety factor point of view, the pillar width should be at least larger than 6.5 m (21.3 ft) in the study site. Otherwise, the pillars may be unstable.

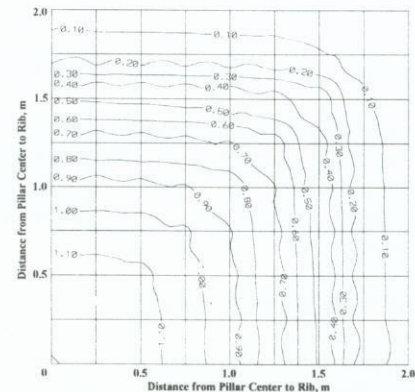


Fig. 5 Safety Factor Contour in 4m-Pillar

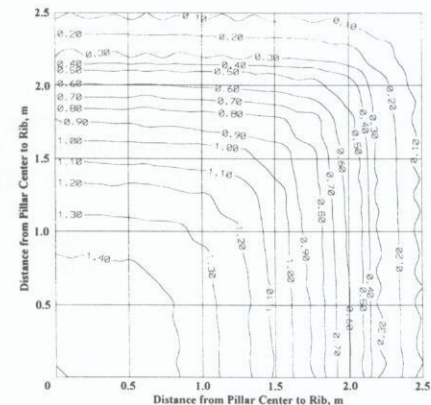


Fig. 6 Safety Factor Contour in 5m-Pillar

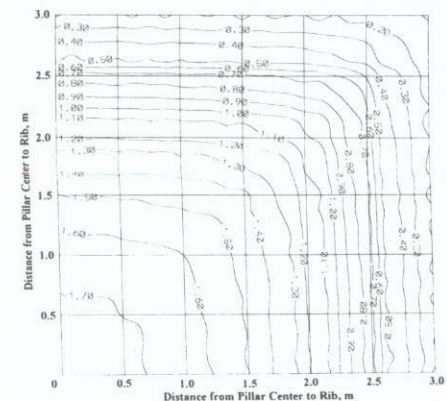


Fig. 7 Safety Factor Contour in 6m-Pillar

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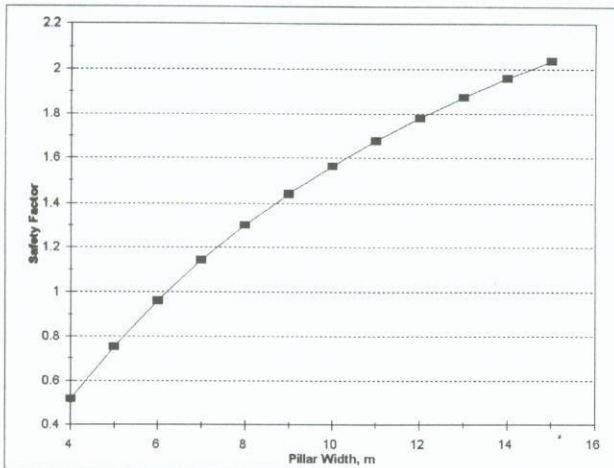


Fig. 8 Safety Factor Change with Pillar Widths

Since in the failure area, most pillars were less than 6 m (19.7 ft) wide (Table 2), the average safety factors of pillars were less than 1.0, as shown in Fig. 8. Therefore, the main reason for this massive pillar failure is that pillar sizes were too small and uneven.

3. Influence of the Surface Topography

In order to analyze the reason why the surface cracks occurred, the surface strain has been predicted by using the subsidence prediction program package CISP version 2.0 (Peng and Luo^[2], 1994). The strain indicates whether the ground surface is stretched or compressed. The predicted final surface strain is shown in Fig. 9 (Note: the failed panel is sub-critical in size). A positive strain indicates that the ground surface is subjected to tensile strain while a negative one for compression. Along the panel transverse direction, the maximum predicted tensile strain is about 4.5×10^{-3} ft/ft (Fig. 9(a)) while it is about 2.0×10^{-3} ft/ft along the panel longitudinal direction (Fig. 9(b)). The predicted maximum compressive strain is about 9.2×10^{-3} ft/ft. Since the predicted maximum tensile strain is 4.5×10^{-3} ft/ft, ground cracks may occur depending on local soil condition. If the ground is very dry, ground cracks may occur. Possible location of ground cracks over the failure area is shown in Fig. 9(c). The actual ground cracks are shown in Fig. 10. The distance from the nearest edge of the failure area to the high cliff boundary ranges from about 150 ft to 300 ft. Generally, surface subsidence should not affect this cliff, but it did. Because the overburden near this boundary tends to move towards the cliff. Surface subsidence triggered overburden movement that created the ground cracks.

In summary, this massive pillar failure was caused by pillars being too small. In the failure area, the average safety factors of all pillars were less than 1.0. In addition, the pillar shapes were not the same. Some pillars were triangular while others were rectangular. Under this condition, the smallest pillars yielded first. Then the loading on these pillars were transferred to the adjacent ones. Finally, all pillars in the area failed. Moreover, since the overburden had a potential movement toward the high cliff boundary, the pillars near this boundary may be subjected to shear stress at the interfaces

between pillars and roof/floor. This further reduced pillar stability.

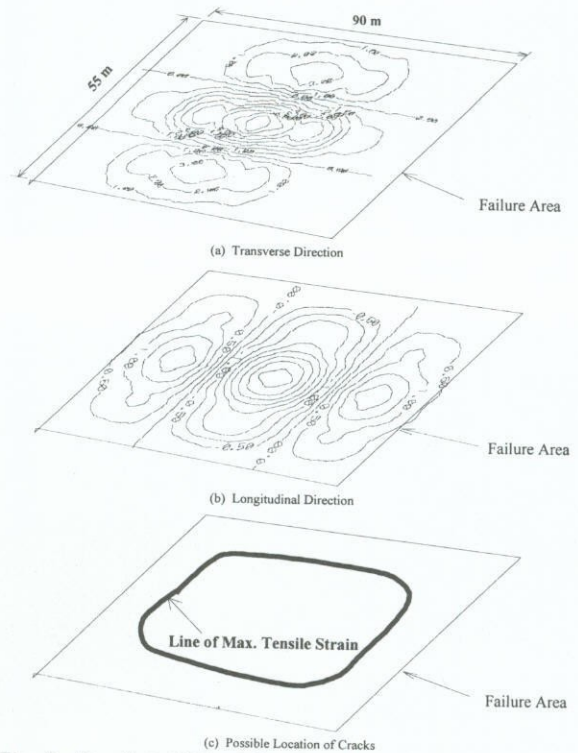


Fig. 9 Predicted Final Surface Strain and Possible Location of Ground Cracks (Plan View)
(see Fig. 2 for shape of failure Area)

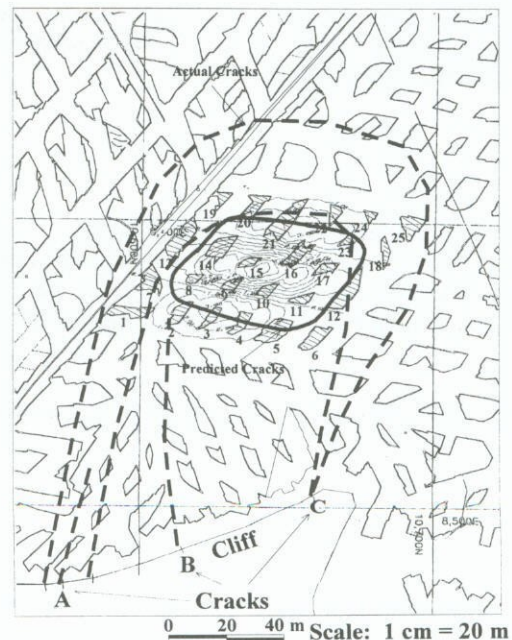


Fig. 10 Actual Cracks and Predicted Ones (Plan View)
(see Fig. 4 for crack location)

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COAL MINE

1. Geological and Mining Conditions

The coal mine was extracting the No.4 Rider seam by using the room-and-pillar mining method. The immediate roof is gray shale, about 5 ft thick. The main roof consists of shale and sandstone. In some areas, the immediate floor consists of claystone while in other areas, the floor is shale or shale with claystone. The rock mechanics properties are listed in Table 3. The roof and coal are both very competent. The overburden depth ranges from 200 ft to 650 ft.

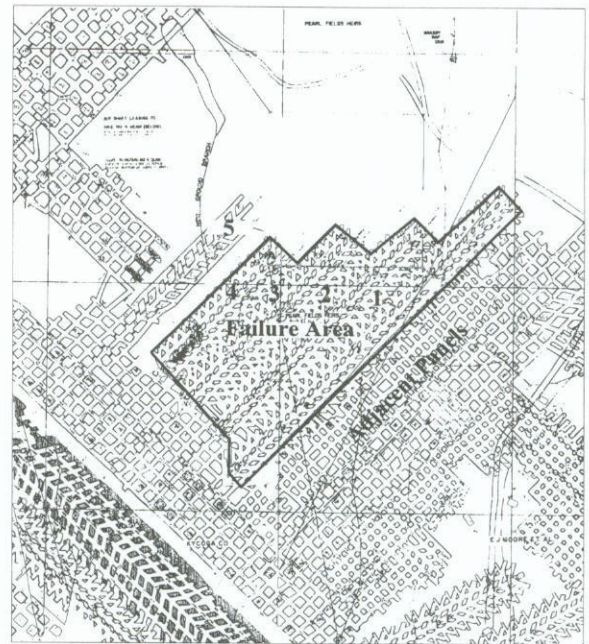
Table 3 Rock Mechanics Properties for Coal Mine^[3]

Rock Type	Compressive Strength (psi)	Tensile Strength (psi)	Young's Modulus ($\times 10^6$ psi)	Remark
Shale	5,220	1,195	2.86	in roof
Sandstone	8,040	500		
	~12,000	~1,190		
Shale	5,720			in floor
Claystone				broken in water
Coal	3,730			

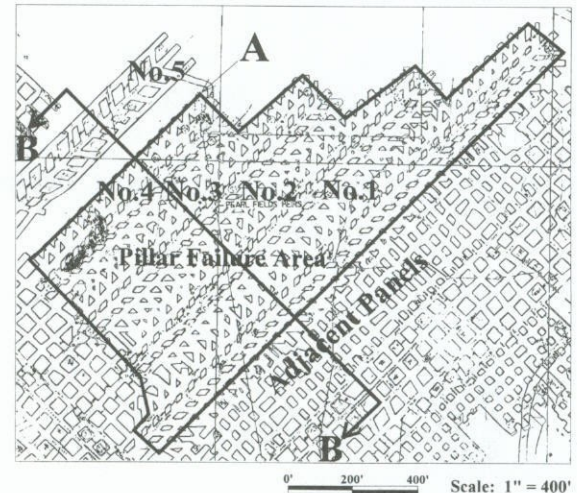
Fig. 11 shows the failure area that covered 4 panels. The overburden depth in this area ranges from 200 ft to 560 ft, as shown in Fig. 12 (note the elevation of the coal seam is about 1,100 ft). The mining height was about 48 in. The width of entries was about 20 ft. The pillar sizes were 20 ft by 20 ft and recovered partially. The remnant pillars in the panels were triangular. The panel width was about 120 ft (rib to rib) and the width of barrier pillars between the panels was about 60 ft. The barrier pillars from panels No.1 to No.4 were punched on retreat into small chain pillars. The extraction ratio in this failed area was about 80%. According to mine management the mine never had any problem in all other areas using 72% extraction ratio.

The massive failure initiated at point A (Fig. 11) of panel No.4, where the roof fell suddenly and covered up all face equipment. It propagated outby quickly and extended to the whole panel in 15 ~ 20 minutes, and then spread to panels No.1 ~ No.3 overnight. The roof remained intact, but the pillars were punched into the floor, as shown in Fig. 13. The original entry height was 48 in and after failure, its height was reduced to only 24 ~ 36 in. However, the roof and pillars appeared to be fine. In the failure area, the geological conditions are shown in Fig. 14^[3]. The immediate floor was claystone, 2.2 ft thick. The claystone was very weak, and broke in water. For that reason, its mechanical properties are not available.

When the roof fall began at point A, a production face crew were working there. All crew escaped unharmed. A 3-entry panel (Panel No.5 in Fig. 11) was developed to reach point A and to recover all face equipment later.



(a) Mine Layout Surrounding the Failure Area



(b) Failure Area

Fig. 11 Massive Pillar Failure Area in the Coal Mine

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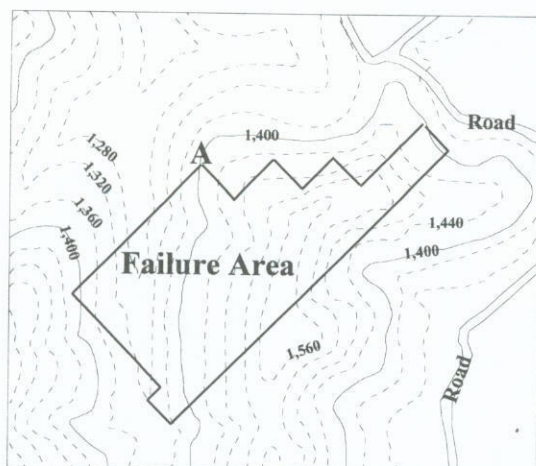


Fig. 12 Surface Elevation

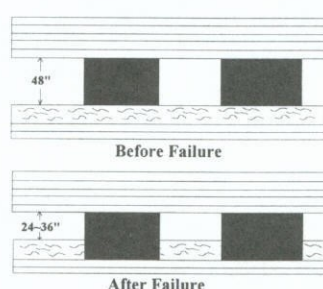


Fig. 13 Conceptual Drawing Showing Entry Condition Before and after Failure

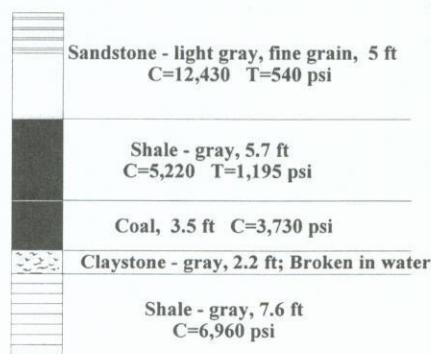


Fig. 14 Near-Seam Geological Column in the Failure Area^[3]

2. Analysis of Mine Collapse

As shown in Fig. 14, the immediate floor consisted of claystone, about 2.2 ft thick. Since the claystone was very weak, its mechanical properties could not be obtained by standard specimen preparation procedure. In addition, the roof and pillars appeared to be fine after the failure has occurred. Therefore, in this study it is assumed that the roof and pillars were in the elastic state and the floor was in the ideal elastic and plastic state. In the analysis, the Mohr-Coulomb yield criterion has been used for the floor material.

The model used in this study is shown in Fig. 15. It simulates the stress conditions along the cross section B-B in Fig. 11. There are 7 steps in this simulation procedure. In step 1, the original in-situ stress condition before entry development is simulated. In step 2, the adjacent panels to the right of this failure area are developed and in step 3, panels No.1 and No.2 are developed without punching into the barrier pillar between them. In step 4, the barrier pillar is punched and in step 5, panel No. 3 is developed without punching the barrier pillar between panels No.2 and No.3. In step 6, the barrier pillar between panels No.2, and No.3 is punched and the barrier pillar between panel No.3 and No.4 is also punched. Finally, in step 7, panel No.4 is developed.

In order to analyze the stress changes in those steps, only the principal stresses at the floor line level and the roof line level are studied. Since the floor is weak and yield first, the stress in the floor at different levels is also analyzed. There are three levels in the floor, numbered Level 1~3 in floor as shown in Fig. 15. Level 1 in floor is at the floor line level. The distance between levels 1 and 2 is 2 ft while the distance between levels 2 and 3 is 1.5 ft. The stress in the immediate roof is also studied at the roof line level (level 1 in roof, as shown in Fig. 15).

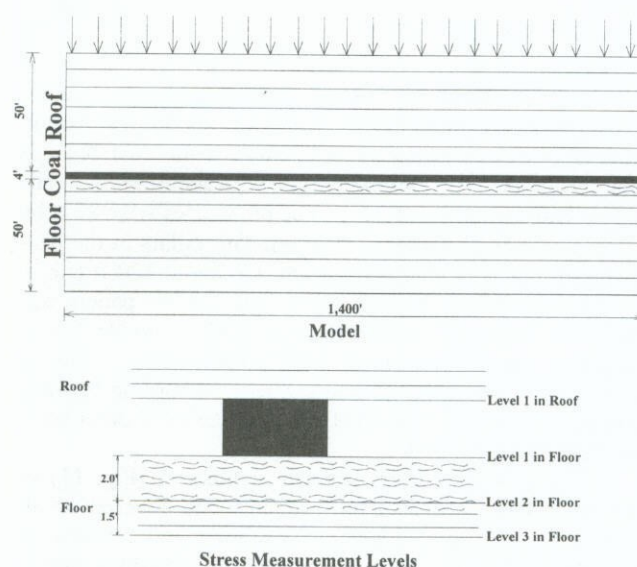


Fig. 15 Finite Element Model and Locations of Measurement Levels

Maximum Principal Stress

The maximum principal stress at the floor line level (level 1 in floor in Fig. 15) is shown in Fig. 16 for different steps. Before the barrier pillar between panels No.1 and No. 2 is punched into small pillars (step 3), the maximum principal stress under the barrier pillar is small (Fig. 16a). In panel No.1, the stress under the outside row of pillar is larger than that under the other pillars. After the barrier pillar is punched into small pillars (in step 4), the stresses under all pillars increase slightly (Fig. 16b), but those under the barrier pillars increases much larger. When panel No. 3 is developed, the stress under the pillars in panels No.1 and No. 2 hardly change (Fig. 16c). After the barrier pillar between panels No.2 and No.3 is punched into small pillars, the stresses under the pil-

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lars adjacent to panel No.1 and the pillars on panel No.3 increase slightly (Fig. 16d). During the development of panel No.4, the maximum principal stress at the floor line level in this panel increases significantly (Fig. 16e). For example, the maximum principal stress at level 1 in floor in panels No.1 ~ No.3 is about 700 psi, while in panel No.4 it reaches near 900 psi.

Fig. 16 also indicates that at the floor line level the maximum principal stress under the outside rows of pillar in panels No.1 ~ No.3 is slightly larger than that under the central rows of pillars. Under normal condition, the stress under the central pillars is larger than that under the outside pillars. In this case, since the floor stratum (claystone) yields first, the loading on the central pillars is transferred to the other pillars. Therefore, the floor stress under the outside pillars is larger than that under the central pillars. In addition, the stress under the punched barrier pillars is less than that under panel pillars, because the width of the punched barrier pillars is larger than that of the panel pillars.

At the roof line level, the maximum principal stresses for different steps are shown in Fig. 17. Generally, the stress in the roof is larger than that in the floor. However, the stress in the roof changes in the similar way as those in the floor. Before the barrier pillar is punched into small pillars, the stress at the roof line level over the barrier pillar is small (Fig. 17a). After the barrier pillar is punched into small ones, the roof stress increases slightly. Generally, the roof stress over the pillar hardly increases with the panel width, after the width of panel has exceeded some value. Compared the roof stresses in steps 4 ~ 6, it is found that the stresses over the panel pillars remain nearly unchanged (Fig. 17b ~ d). In panels No.1 ~ No.3, the maximum principal stress at the roof line level over the outside pillars is larger than that over the central pillars. When panel No.4 is developed, the stress in the roof increases significantly (Fig. 17e).

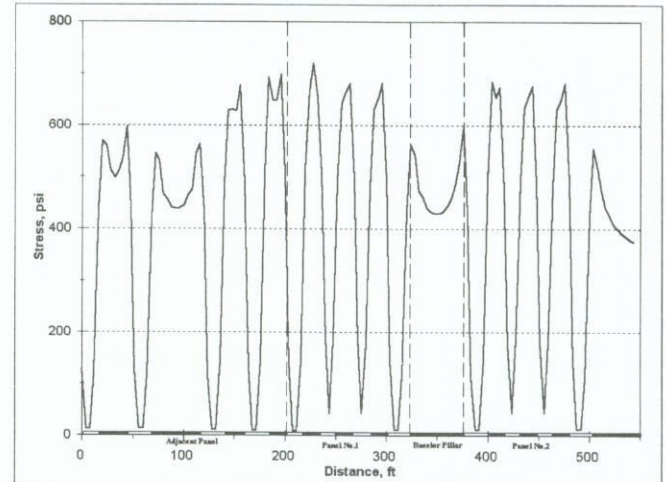
As shown in Figs. 16 and 17, the maximum principal stresses at the floor and roof line levels in panel No.4 are much larger than those in panels No.1 ~ No.3, when panel No.4 is being developed. Since the roof and coal is competent and the floor is weak, the floor in panel No.4 yields first.

Minimum Principal Stress

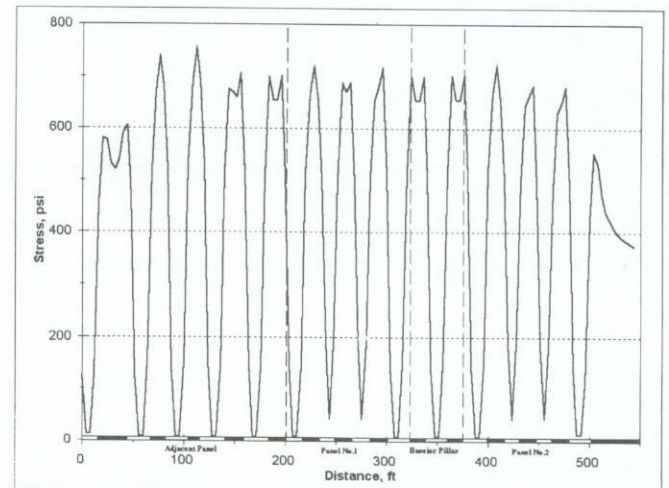
The minimum principal stress at the floor line level is shown in Fig. 18 for different steps. In these figures, a negative value denotes tensile stress. All entry floors are in tension and the maximum tensile stress occurs in the floors of central entries. As the panel width increases (i.e. when the barrier pillars have been punched into small ones), but before panel No.4 is developed, the minimum principal stress in the floor remains almost the same (Fig. 18a ~ d). When panel No.4 is being developed, the tensile stress in the floor increases significantly (Fig. 18e). The tensile stress in the floor under the outside entries is about 50 psi. However, in panel No.4, it reaches to near 100 psi, which is almost as the same as that under the central entries.

The minimum principal stresses at the roof line level are shown in Fig. 19 for different steps. It indicates that all entry roofs are in tension and the maximum tensile stress occurs in the central entry roofs. In addition, the roof stress hardly increases with the panel width (note that punching out

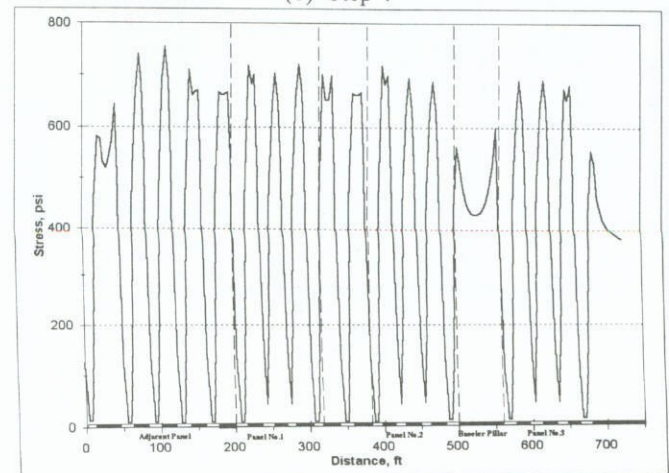
barrier pillars means increasing the panel width). Since the sizes of chain pillars (punched barrier pillars) are larger than those of the remnant pillars in panels, the maximum tensile stress occurs in the central entries. When panel No.4 is being developed, the roof stress increases (Fig. 19e). For example, the tensile stress is about 300 psi in panel No.4 while in all other panels, it is about 200 psi.



(a) Step 3

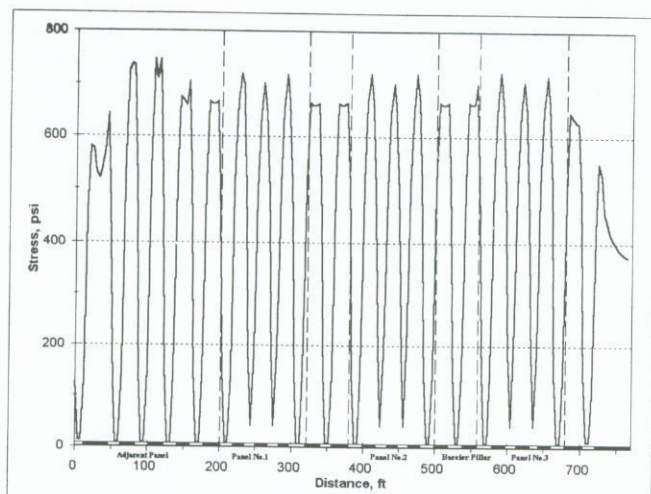


(b) Step 4

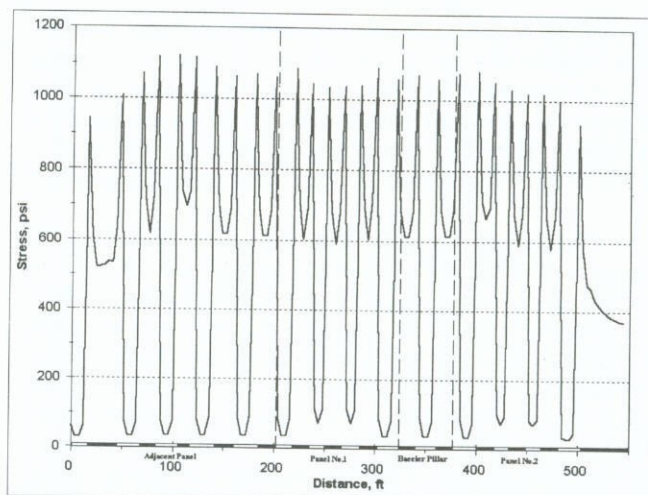


(c) Step 5

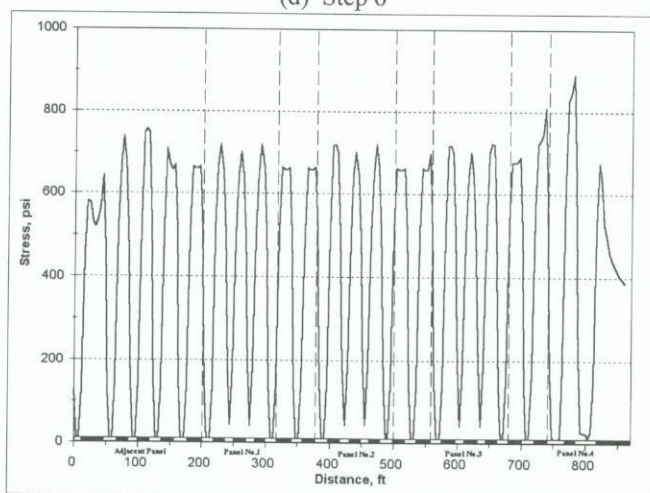
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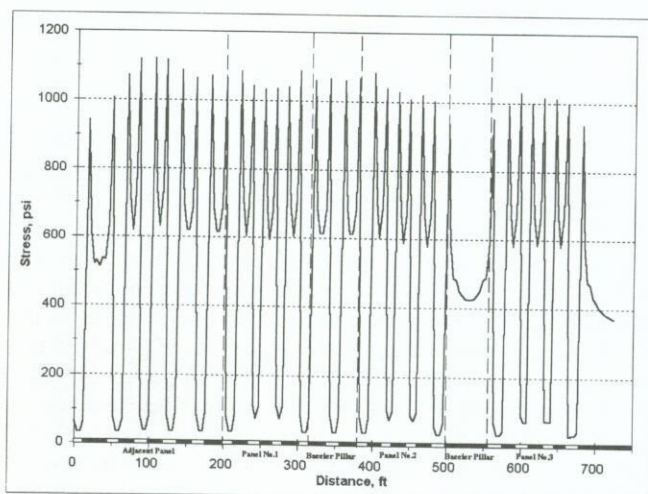
(d) Step 6



(b) Step 4

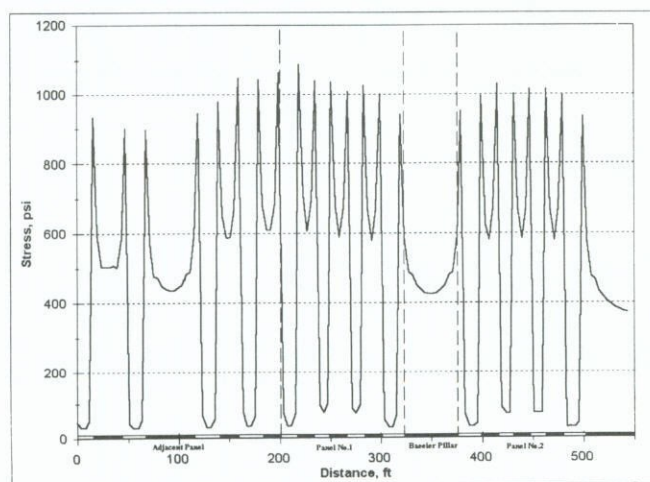


(e) Step 7

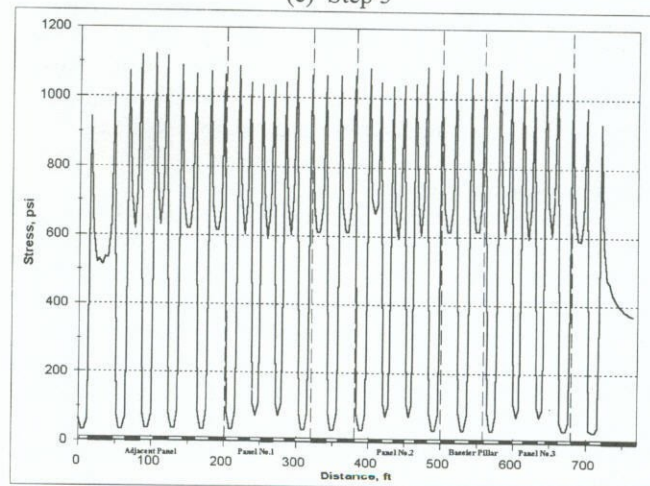


(c) Step 5

Fig. 16 The Max. Principal Stress at Level 1 in Floor

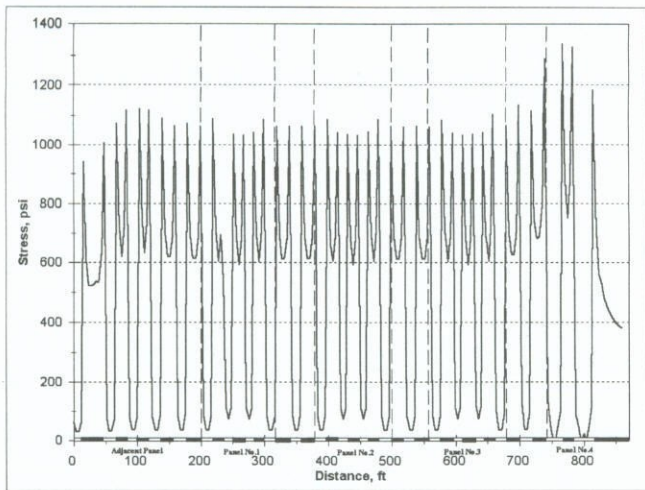


(a) Step 3



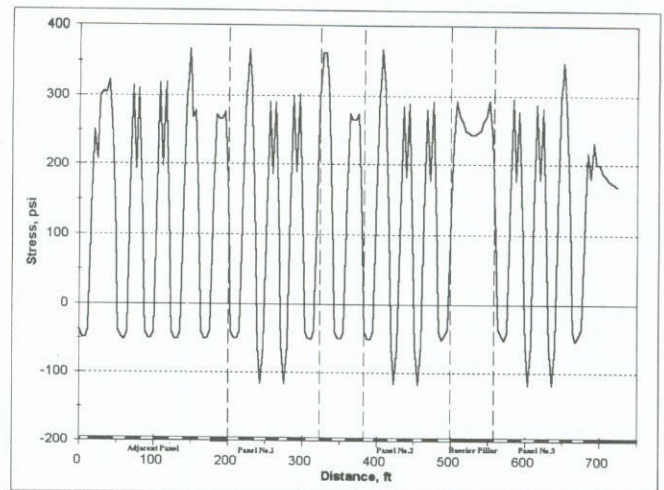
(d) Step 6

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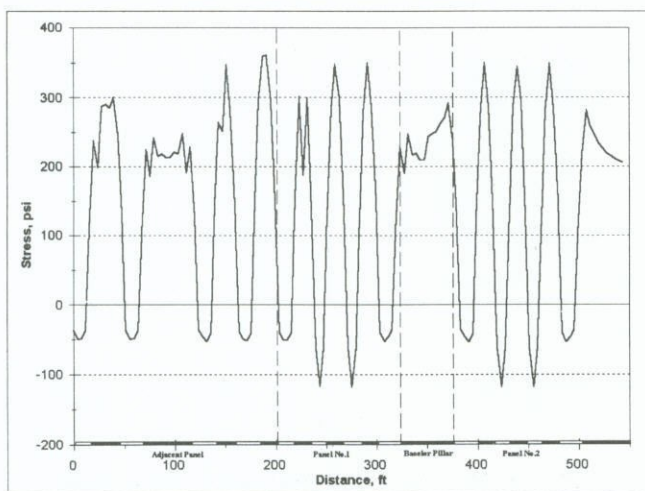


(e) Step 7

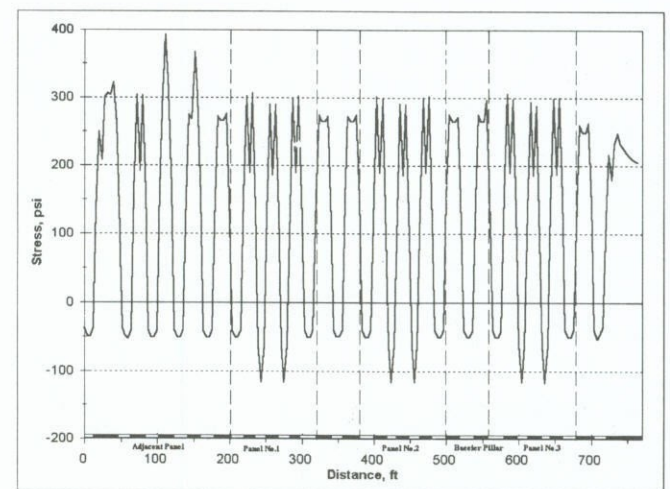
Fig. 17 The Max. Principal Stress at Level 1 in Roof



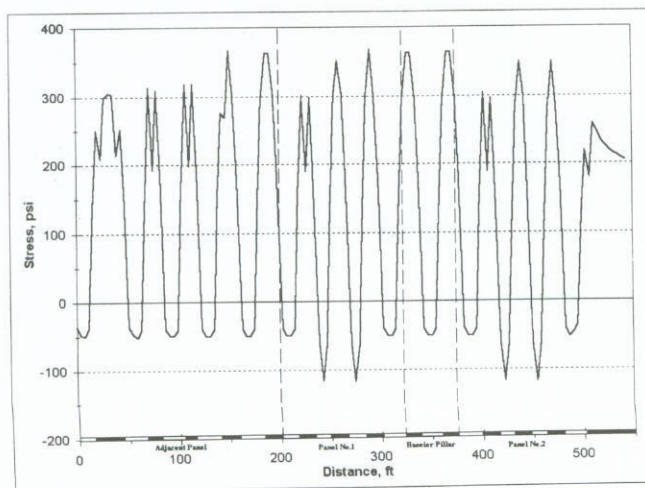
(c) Step 5



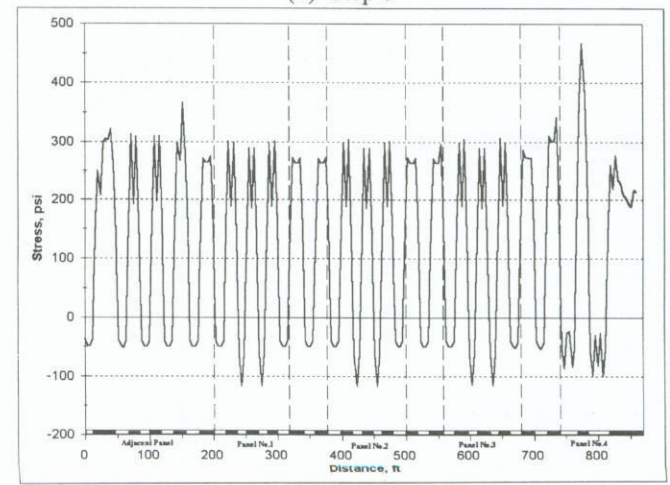
(a) Step 3



(d) Step 6



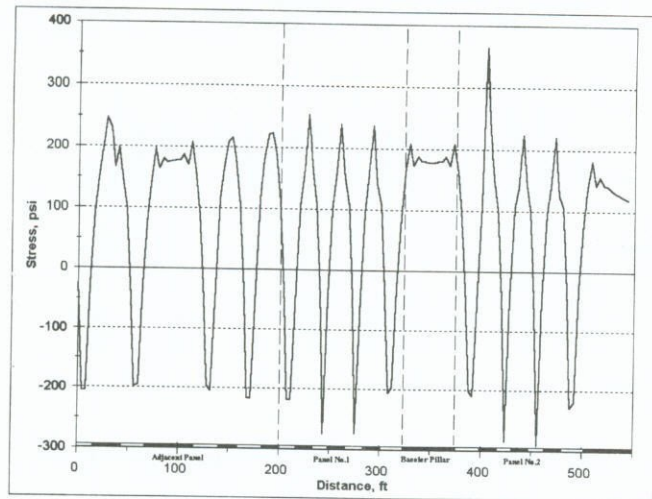
(b) Step 4



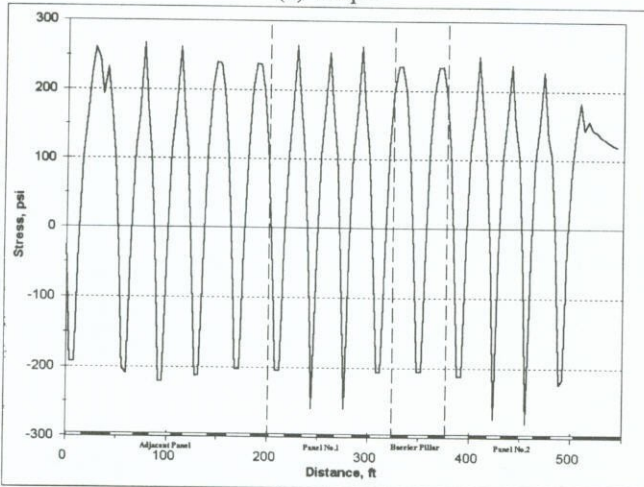
(e) Step 7

Fig. 18 The Min. Principal Stress at Level 1 in Floor

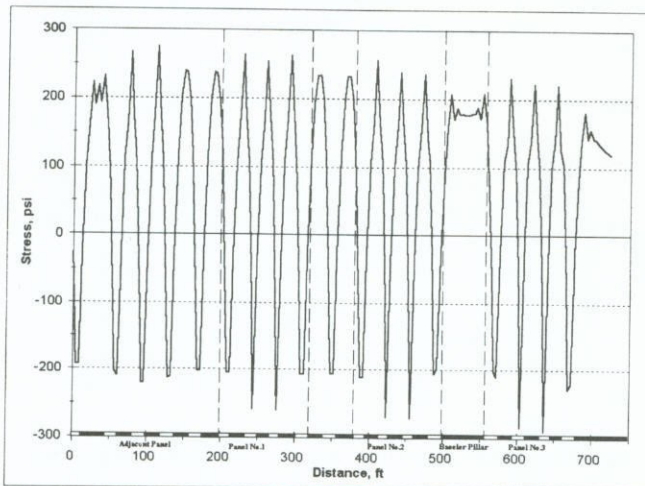
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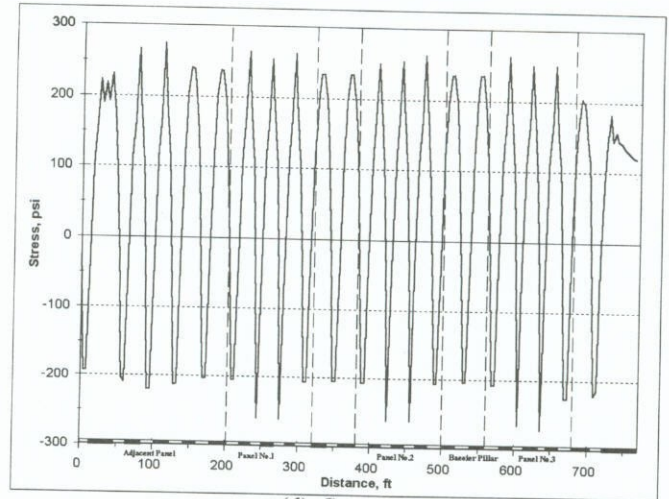
(a) Step 3



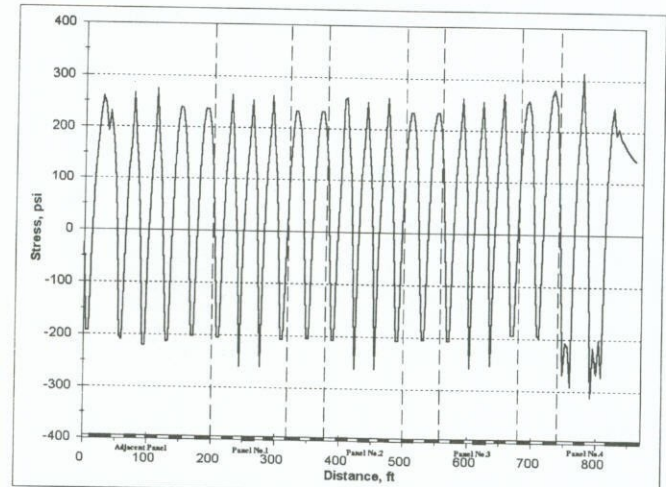
(b) Step 4



(c) Step 5



(d) Step 6



(e) Step 7

Fig. 19 The Min. Principal Stress at Level 1 in Roof

The principal stresses at the different floor levels are shown in Fig. 20. It indicates that the maximum principal stresses at levels 1 ~ 3 in the floor are near the same (Fig. 20a). However, the differences of the minimum principal stress between these levels are significant (Fig. 20b). Generally, the minimum principal stress at level 3 is less than that at levels 1 and 2. However, the difference of stress between levels 1 and 2 in panel No.4 is very small. This indicates that the whole claystone under pillars in panel No.4 may have yielded.

CONCLUSIONS

The causes of two mine panels collapses have been analyzed in this paper. In the first case, the main reason of the massive pillar failure is that the pillars were too small. In addition, the cliff boundary also has some influence on the pillar stability. Since the pillar failure occurred about two weeks after the panel was mined out, "time" played a role in the pillar collapse. Since the average safety factors of all pillars were less than 1.0 and the edges of pillars were yielded, the loading originally acting on the yielded zones was transferred to the core of the respective pillars. This resulted in the yielded zone becoming larger and larger. Finally, the whole pillar yielded.

In the second case, since the floor consisted of a weak 2.2ft-thick-claystone in the failed area, the floor yielded first in the roof-pillar-floor system. In the floor, the stress under the outside rows of pillars in the panels is larger than that under the central pillars. Before panel No.4 is developed, the floor stress increases only slightly with the panel width. After panel No.4 has been developed, the maximum vertical compressive stress in the floor increases significantly and its tensile stress is also large. In addition, in the floor layer of claystone the tensile stresses are almost the same, which means the claystone under the pillars of panel No. 4 may have yielded. This might account for why the floor failure initiated in panel No. 4.

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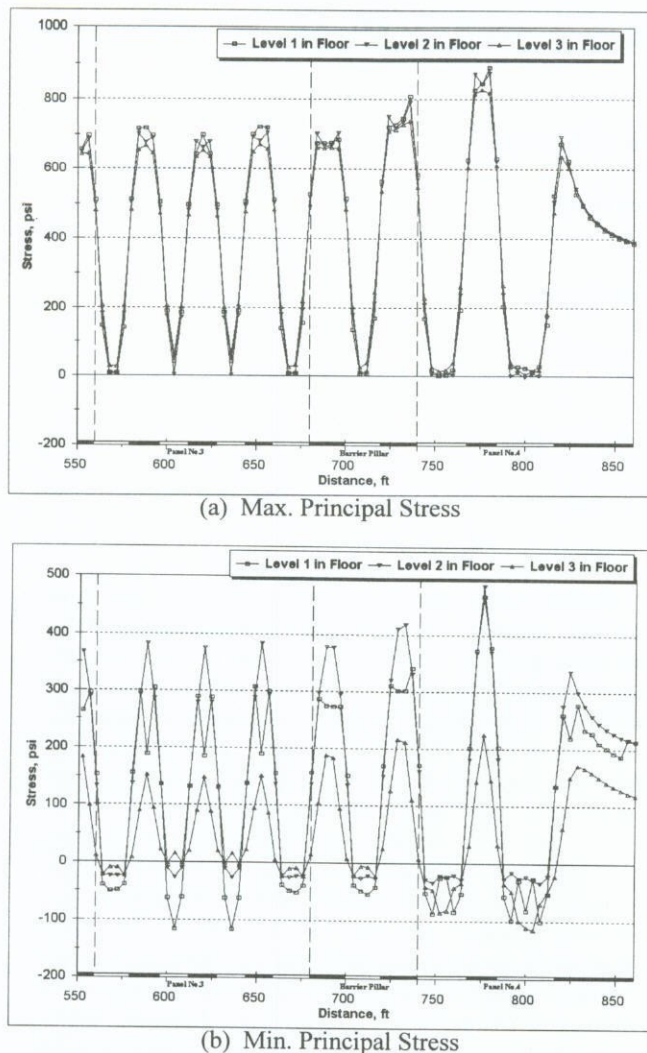


Fig. 20 The Principal Stresses at Different Floor Levels

Based on the above stress analysis, it is found that in the roof and floor of panels No.1 ~ No.3 the maximum principal stresses under/over the outside rows of pillars were larger than those under/over the central rows of pillars because the floor had yielded. However, the tensile stress in the floor and the roof of the central entries was larger than those in the other entries. When panel No.4 was developed, the maximum and minimum principal stresses in the floor of panel No.4 increased and became larger than those in the same locations in panels No.1 ~ No.3. In addition, in the floor the minimum principal stresses at the different levels were almost the same, which means the claystone under the pillars of panel No. 4 may had yielded. This might be the reason why the floor failure initiated in panel No. 4.

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Roof Geology Mapping in Underground Coal Mines

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ABSTRACT

Underground roof geology reconnaissance is being conducted by CONSOL Inc. to enhance mine productivity. Often, roof instability within mine entries is the consequence of highly variable geology that cannot always be detected from information provided by widely-spaced surface core holes. Utilizing a combination of roof drilling data, geophysical logs, and borehole camera and fiberscope observations, roof geology mapping is conducted within longwall development entries in areas of specific concern prior to commencement of longwall mining. Depending on the uniformity of the site-specific roof geology, data point spacing along the longwall development generally ranges from one to five crosscuts. Surface core hole data are correlated with the roof geology mapped underground to validate the underground observations. Detailed roof geology maps can then be produced to enable ground control engineers and mine management to select cost-effective ground control techniques. However, this procedure is not necessary or useful in all situations and it is employed only when there is strong geologic evidence to justify a closer inspection.

INTRODUCTION

The geomechanic, geologic, and geometric conditions that result in coal mine roof falls are both complex and site dependent. Overburden and tectonic stresses, near-seam lithology and structure, mine geometry and development cycle, and ground support techniques may all interact to influence the stability or instability of a mine opening (1). However, while interaction among these factors can be significant, it is often difficult to assess due to many possible variations, as well as the considerable range and areal variability of these factors.

Among the previously mentioned factors, the most complex, variable, and difficult to assess are often those related to geology. With the exception of close multiple seam mining scenarios, overburden-related stress and influence on typical coal mine environments are generally well understood as functions of depth, rock density, and abutment influence angle. Measurements of in-situ horizontal stresses (2,3) have shown that tectonic-related stresses at typical coal mine depths are quite consistent in both orientation and magnitude over large regions, with minor changes within normal stream valley environments and localized exceptions near major fault features. In addition, considerable work has been performed to assess the impacts of mine extraction, orientation, and development cycle on in situ stress concentration and impacts (2,4). Likewise, within the past 40 years, ground support technology has developed from an art to a science as a result of an abundance of studies focused on understanding the material properties, support mechanics, and in-situ performance of both bolt and standing support methods (5,6). Although some technologies such as tensionable cable bolts and yieldable wood prop alternatives are still being perfected, the majority of the ground support technologies have long and proven track records. In addition, access to an abundance of reliable support options has provided the ground control practitioner with a formidable toolbox to address almost any type of in situ condition encountered, assuming that the conditions are known.

On the other hand, the geological factors that influence coal mine ground stability are not as well defined. The potential for rapid variations in geology makes assessing site specific support needs difficult because such changes are not always detectable or obvious from surface drill data, mine opening observations, and standard roof and rib bolting operations, although the latter could prove quite beneficial if reliable, cost-effective geologic measurement and interpretive methods could be developed.



As a means of better defining site-specific near-seam geology to address ground support, CONSOL has developed and implemented in selected applications underground geologic reconnaissance techniques with a primary emphasis on minimizing unstable roof occurrences in critical areas such as longwall gateroad, setup, and bleeder entries. Maintenance of such entries is crucial and directly influences the large majority of longwall coal production. In addition, since these entries are subject to high abutment and concentrated tectonic stresses, they often require more ground support than mains development. However, it is important to note that this reconnaissance procedure is not necessary or useful in all situations and that it is employed only when there is strong geologic evidence to justify a closer inspection.

TYPICAL COAL MINE GEOLOGIC RECONNAISSANCE

For many coal mines, most of the near-seam geologic information for assessing roof stability and ground control needs results from surface exploration coring programs (7). Occasionally, additional information may be provided by highwall and overcast cut information and, ironically, from roof falls. In the advent of a fall, as much information as possible should be gained to determine the local geologic conditions, possible causes of the fall, and roof support inadequacies.

Most surface core hole programs are primarily designed to assess coal quality and thickness, and are usually drilled on 2000 to 3000 foot centers because of the typically gradual trends of coal quality, as well as cost and surface access constraints. In addition, many coal reserve drilling programs, particularly those prior to the 1980's, only provide generic information regarding mine roof and floor lithology with little attention paid to detailed lithologic characteristics and structure. While such sparse information may be sufficient in defining the generic mine-wide stratigraphy, it does not provide the relevant site-specific detail that could be crucial for assessing the ground support needs.

INFLUENCE AND DETECTION OF ANOMALOUS GEOLOGY

In most coal mine environments, normal geologic conditions, particularly those influencing roof geology, are not problematic from a control and support standpoint. Representative conditions can usually be ascertained from core hole data and experience. Generalized ground support plans are designed on the geologic norm, and are typically based on conservative designs that have proven to work in

similar conditions. Primary and standard supplemental mine roof control plans are typically established on the basis of such normal conditions.

However, a large majority of ground support problems in coal mines results from anomalous conditions, rather than normal ones, and the normal conservatism built into roof support plans is usually sufficient to address minor changes in the geologic and stress environment. While it is recognized that inadequate support design, material deficiencies, and improper support installation may occasionally lead to unstable roof conditions, experience suggests that these instances represent only a small minority of the cases of unstable roof. Therefore, given that changes in the in situ stress state are likely to be gradual (with the exception of pillar stress concentrations), geologic anomalies come to the forefront as a factor in unstable roof conditions.

Based on this assessment, geologic exploration geared toward reducing the number of unstable roof conditions must be capable of identifying anomalous geologic environments. To do so requires that geologic conditions be identified at a spacing that will capture most, if not all, of the anomalous conditions. Experience indicates that the majority of geologically anomalous conditions occur within a linear span of 50 to 500 feet. Given the physical and cost limitations of available surface exploration technology, sufficient cost-effective identification of geologic anomalies on this scale is best done underground within the mine openings and entryways, utilizing either drill hole detection or remote sensing methods.

The most direct and accurate means of collecting near-seam geologic data is through underground drill-hole exploration. Toward this end, both coring and boring methods are available. While coring provides the most accurate data (i.e., rock core samples), it is also more complex, labor intense, time consuming, and costly than boring. Conversely, while boring does not provide readily identifiable samples, it can accommodate other borehole inspection equipment for rapid, cost-effective geologic interpretation. In addition, cuttings can still be collected, if warranted, for elemental or mineralogical analyses. The borehole method can also make use of the existing face and track-mounted bolting equipment available to every underground coal mine. Where necessary, coring can be utilized to augment and calibrate the borehole data.

Although geophysical remote sensing methods, such as in-seam seismic, radio imaging, and ground probing radar offer much promise as underground exploration tools, at the present time these methods are geared mostly toward identification of large anomalies with major displacement offsets or very abrupt changes in lithology with sharp

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lithologic contacts (8). Accurate interpretation of remote sensing results relies heavily on calibration from more definitive means of geologic identification. Nevertheless, because well-calibrated geophysical surveys have the potential to provide remote access to geology in advance of mining and to continuously map geologic signatures between drill holes or other observation points, future development of underground geophysical sensing technologies to define near-seam lithology and structure is warranted.

PROBLEMATIC GEOLOGIC ANOMALIES AND ENVIRONMENTS

Based on CONSOL's experience, the most problematic geologic anomalies are usually those related to sudden changes in lateral roof lithology and structure. Although near-vertical features, such as faults and clay veins, can pose production and ground control difficulties, their lateral effects are usually localized and limited to within several tens of feet of the mine-level encounters. In addition, vertical anomalies are often readily identifiable from the seam level and can be visually delineated. However, lateral changes in geology, particularly those in the roof, can be areally extensive, difficult to delineate from seam level, and difficult to control without adequate knowledge of the condition and its boundaries.

Examples of problematic changes in lateral geology include the following:

- The sudden presence of weaker units, such as claystone, roof coal, and slickensided shale, resulting from variations in the coal-depositional environment or from differential compaction.
- Major transitions in roof deposition facies, such as marine or brackish (limestone) to flood plain/river deposits (sandy shale and sandstone).
- The presence of highly laminated strata produced by rapid cyclical variations in depositional environment or differential movement following deposition.
- The presence of sand channel structures, whether intrusive or not, that typically result from post-coal-deposition river bank scouring and sand deposition (i.e., washouts), or from contemporaneous sand deposits resulting from a complex, highly variable depositional environment.

Changes in seam structure, such as the presence of severe rolls, have been noted to further compound ground control problems associated with the above-stated anomalous features, as has the presence of major shallow-cover stream valleys. The problematic association with rolls is thought to be the results of fractures and localized stress concentrations due to the bending of the seam and from a further degradation of the depositional environment; slickensided and laminated strata are occasionally found in the vicinity of seam rolls. The problematic association with stream valleys is likely due to horizontal stress concentrations under the valley bottom or to predisposed structural weaknesses associated with the very existence of the stream valley.

UNDERGROUND ROOF GEOLOGY RECONNAISSANCE

CONSOL's underground geological reconnaissance efforts involve data collection, interpretation, and mapping, the application of practical exploration tools and techniques based on anticipated geologic and engineering conditions, the use of detailed core hole control to maintain interpretation accuracy, and, ultimately, the marriage of geology with proven and accepted coal mine ground control principles to address support needs. While a detailed discussion of the latter is beyond the scope of this paper, the principles which are referred to, including concepts such as pillar and entry sizing, influence of abutment stress and horizontal stress concentration, roof support design, floor bearing capacity evaluation, and assessment of roof failure mechanisms, are known to most ground control practitioners, although specific design methods and applications may vary. Nevertheless, it is the direct linkage of these two disciplines, rock mechanics and geology, that ultimately makes the program successful. For the ground control practitioners, the two disciplines are inherently intertwined.

Underground geologic reconnaissance begins with a detailed assessment of all available geologic information, which primarily includes data from surface exploratory core programs, although underground observations may also be utilized. This information is the basis for the remainder of the program. Since the 1970's, CONSOL's surface exploration program has provided detailed in-house interpretation of core hole lithology and structure, with particular attention to near-seam geology. The information from each core hole, in turn, has been compiled into a single computer database. From this extensive database, detailed roof and floor geology information can be readily retrieved to create scaled cross sections and direct-retrieve (i.e., unit or lithologic thicknesses) or manipulated (e.g., overburden or interburden thicknesses, and outcrop intersection) isopach maps. Coding of coal and major noncoal strata permits further detailed

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three-dimensional manipulation of the data. Figure 1 illustrates a typical near-seam roof geology map which was constructed from CONSOL's geologic database. The Redstone Limestone-to-Pittsburgh Seam interval contours are plotted to show a roof geology transition from a limestone to sandstone environment.

Plans for underground reconnaissance are developed utilizing the existing core hole data, anticipated geologic trends, and best judgement relative to possible areas of concern. Vertical boreholes (roof bolt holes of 1 3/8-inch diameter) are spaced according to crosscut/intersection spacing along the entryways. Typical spacing is from 1 to 3 crosscuts or about 200 to 600 feet. Where possible and prudent, the track entry is utilized to take advantage of track transportation and track-mounted bolting equipment. This also eliminates dependence on sometimes-difficult-to-obtain face equipment. To acquire data sufficient for evaluation of a range of possible bolt or standing support needs, single roof holes are drilled at each preselected intersection. Maximum hole depth may vary according to entry width. For example, setup entries as wide as 20 to 22 feet may need to be logged to greater depths than standard width (16-foot) entries due to the deeper influence of the wider entries. Typically, however, the site-specific geology and the geologic trends dictate the final depth at each location. For example, holes that encounter thick massive sandstone may be terminated several feet into the sandstone since it is likely to represent a stable geologic environment for support purposes.

Since boreholes do not yield cores for direct observation, accurate interpretation of borehole lithology is best accomplished through a suite of logging techniques. CONSOL typically relies upon three techniques: a drill log based upon drill bit penetration and vibration observations; visual inspection via fiberoptic or borehole camera; and gamma emission detection via a portable gamma logger. Figure 2 illustrates a typical gamma log, drill log and fiberoptic log along a 21-foot deep hole.

The key to the drill log is consistency, which is best ensured by having someone who is trained in geologic interpretation conduct the log. Although a bolter operator's observations may be very helpful (particularly with respect to the ground conditions and support), elimination of individual variability in the geologic interpretation process is paramount for consistency and accuracy. In addition to the drill log, valuable information can also be collected from the bolter dust box (where applicable), since a dust box that has been cleaned prior to drilling of each hole will provide a reasonably accurate scaled geologic section made up of drill cuttings. This can be useful in double-checking the stratigraphic

section that has been created using the drill log. It may also serve as a sample point for subsequent mineralogical analyses, if necessary.

Although generic visual logs can be performed by most anyone, accurate visual logging takes experience and is best accomplished by those trained in geologic interpretation. For example, subtle differences in the interpretation of grain size, type, and orientation can be the difference between the interpretation of a unit as a shale or a limestone.

Logging to detect natural background gamma emission can be accomplished through portable gamma loggers. Such logging units were initially developed for portable logging of surface holes. However, they can be adapted for underground inspection of coal mine roof and floor. Gamma detection serves a vital role in distinguishing between varying degrees of clay within rocks, allowing segregation of clays, shales, sandstones, and limestone.

In addition to the underground logging of boreholes, underground coring is sometimes performed for calibration purposes where surface core hole data are limited or where more detailed examination of rock or laboratory strength testing is desired. CONSOL R&D maintains underground core drilling capabilities that permit the collection of nominal 1-inch cores.

Finally, the extent of the underground reconnaissance effort will ultimately depend on the extent of the issue being addressed. For example, if a question arises regarding setup entry stability and support requirements in a mine block where prior setup entry experience has been gained and one or more surface core hole is located nearby, one to three underground boreholes may be logged to sufficiently fill in the data gap necessary to assess the site-specific geology and support requirements. Similarly, validation of predefined geologic trends/anomalies across a mine block which has been extensively mined may only require logging of a few holes along a relatively narrow path. However, in a new mine block, where prior experience is limited and core hole spacing is sparse, a more extensive logging and mapping effort may be required to delineate suspected features.

INTERPRETATION AND MAPPING OF ROOF GEOLOGY

Following underground field data collection, the geology within each roof borehole is interpreted by compiling the information from separate exploratory surveys (e.g., drill log, visual log, and gamma log), and reconciling differences

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between observation points. Obviously, experience plays a large role in this process. Available surface core hole information is used as a control to further validate nearby underground observations, thereby allowing extrapolation to other boreholes, if necessary. This cross-correlation process is extremely important since direct viewing of the rock is not possible. For cases where surface core holes are not available, underground coring has been conducted to provide the necessary correlative data.

Once the data have been interpreted and correlated, detailed roof geology and structure cross-sections are then produced to visualize the entryway conditions. If sufficient core hole or prior development (e.g., gateroad) data are available, isopach maps, such as that shown in Figure 1, may be produced to show the three-dimensional nature of the normal and anomalous geologic structures.

Figure 3 illustrates a roof geology cross section along a section of longwall development entry in the Pittsburgh seam that was obtained from an underground roof geology reconnaissance program. Overall, many variations were observed along this 9000-foot long longwall development, and the roof geology was far from uniform. However, the most notable anomalous feature was the thickening and seam-encroaching nature of the Pittsburgh sandstone between crosscuts 29 and 33. Subsequent mapping showed that the thickening sandstone was channel-like and extended across several panels. It is noteworthy that weak roof conditions were present in many of the crosscuts, particularly on the flank of the channel.

SUPPORT RECOMMENDATIONS BASED ON INTERPRETED ROOF GEOLOGY

Because underground reconnaissance collects near-seam geologic data from previously mined entries, the most effective ground support design use of the information is for supplemental support, which is typically not installed on-cycle and is thereby accommodated by postdevelopment data collection. This is particularly useful for headgate and tailgate entry supplemental support designs. In such designs, consideration of concentrated horizontal and abutment stresses are obviously important, as is proper pillar design, which can serve to minimize the effects of such stresses.

Timely surveys can also help in the design of proper postslab longwall setup entry support plans. Slabbing of a longwall setup entry typically results in an increase in entry width of 4 to 6 feet (from 16-foot wide to 20- to 22-foot wide), which in turn can increase the roofline rock weight by as much as 200% and result in more roof deformation. As a result, site-specific knowledge of roof geology, particularly the location of a competent bolt anchorage horizon, is very important for selecting the proper support capacity and methodology to insure stability of the setup entry. Figure 4

shows the geologic cross-section result of a setup entry survey, with potential 8- and 12-foot deep anchorage horizons. Although an 8-foot anchorage was considered sufficient for primary support and initial development, a 12-foot deep horizon was selected for slabbing support due to the presence of massive sandstone and sandy shale strata at that horizon, as well as the presence of laminated (LAM in Figure 4), weakly bedded (WBD), and thinly bedded (TBD) units at, and below, the 8-foot deep horizon.

From a bolt design standpoint, experience with this program has suggested that the selection of the bolt anchorage horizon is the most critical step. For any bolt system to work properly, whether based on suspension or beam building, it must have good anchorage. Therefore, selection of a geologic horizon that is both capable of providing consistent anchorage and, in the case of suspension systems, self-supportive, is necessary. At the extreme, claystone and rider coal make for poor anchorage units, not only because of strength deficiencies, but also because they tend to be both prefractured and susceptible to borehole breakout, thereby increasing hole bore size and void space, which subsequently reduces the effectiveness of both mechanical and grout anchorage systems.

The strength and geologic integrity of the intervening strata between bolt anchorage and bolt head is important regardless of the condition of the anchorage horizon. Of particular concern are inherently weak lithologies (e.g., claystone, coal), highly laminated strata, and strata with abundant discontinuities (e.g., slickensides, fossils). Such weak intervening geology reduces the self supportive capacity of roof strata and makes it difficult to attain full beam building potential; in turn, the support design becomes reliant on anchorage in an overlying self-supportive unit. In addition, highly laminated strata tend to be susceptible to progressive degradation (buckling) from high horizontal stresses. Finally, the immediate roof geology is important from the standpoint of surface control. Poor immediate roof may fail between bolts and standing supports and, over time, such failures may contribute to unloading of the bolts and subsequent loss of bolt head contact. Weathering of the immediate roof strata may also play a role in such degradation. The occurrence of immediate roof degradation is particularly noticeable in the Pittsburgh seam, where immediate roof draw slate and roof coal sequences are usually present.

CONCLUSIONS

Underground geologic reconnaissance can provide an awareness and detailed knowledge of anomalous geologic conditions that may impair productivity and increase costs. Roof geology reconnaissance has been initiated by CONSOL to assess longwall development near-seam geology. This program utilizes a combination of drill data, geophysical logs,

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and visual observations coupled with detailed core hole information and underground entry observations. Detailed roof geology cross sections and isopach maps produced from the reconnaissance effort can provide indications of potential ground control problems and enable ground control specialists and mine management to select cost-effective ground control methods through integration of detailed geologic information and rock mechanics principles. However, this procedure is not necessary or useful in all situations and it is employed only when there is strong geologic evidence to justify a closer inspection.

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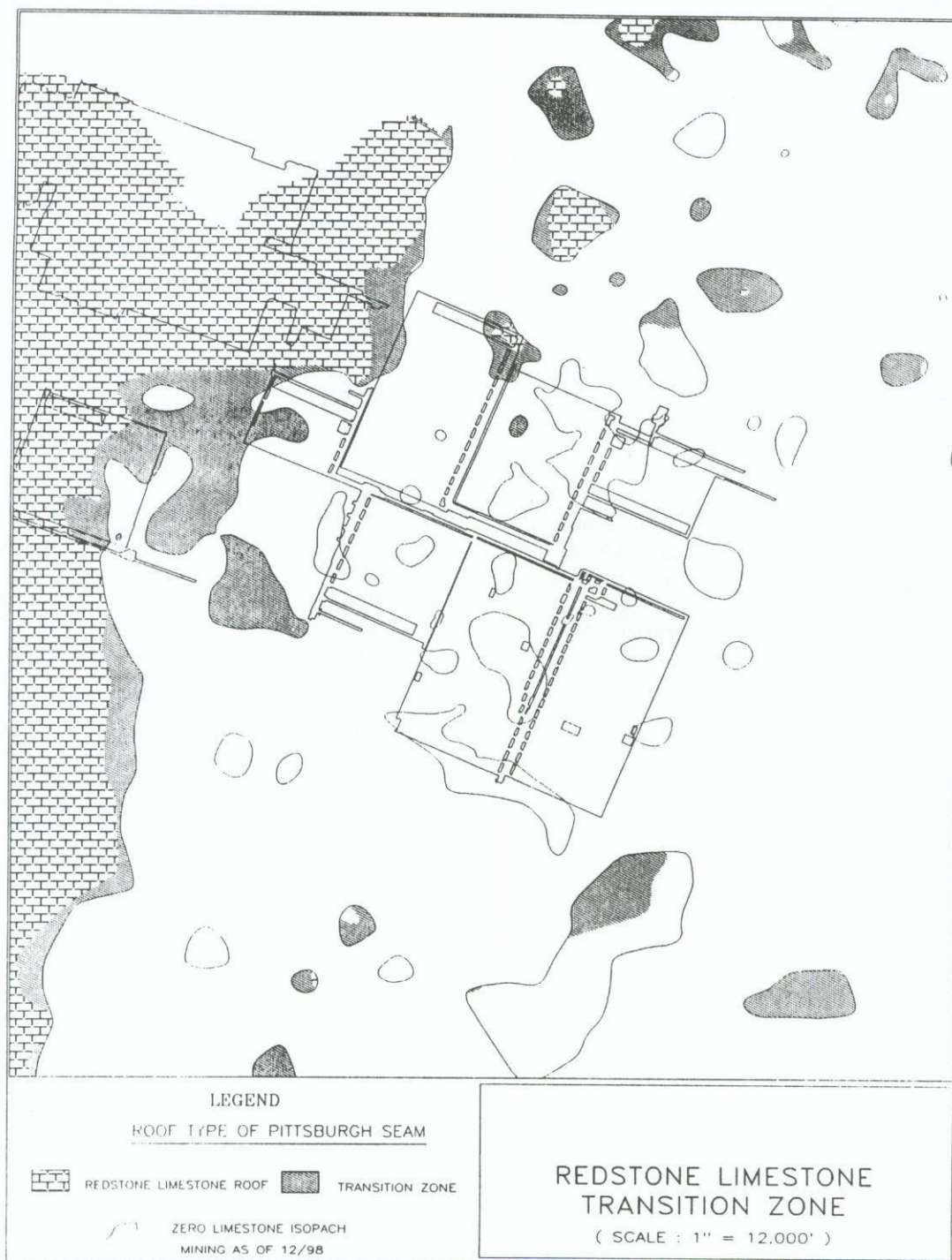


Figure 1. A typical near-seam roof geology map showing the limestone to sandstone transition zone.

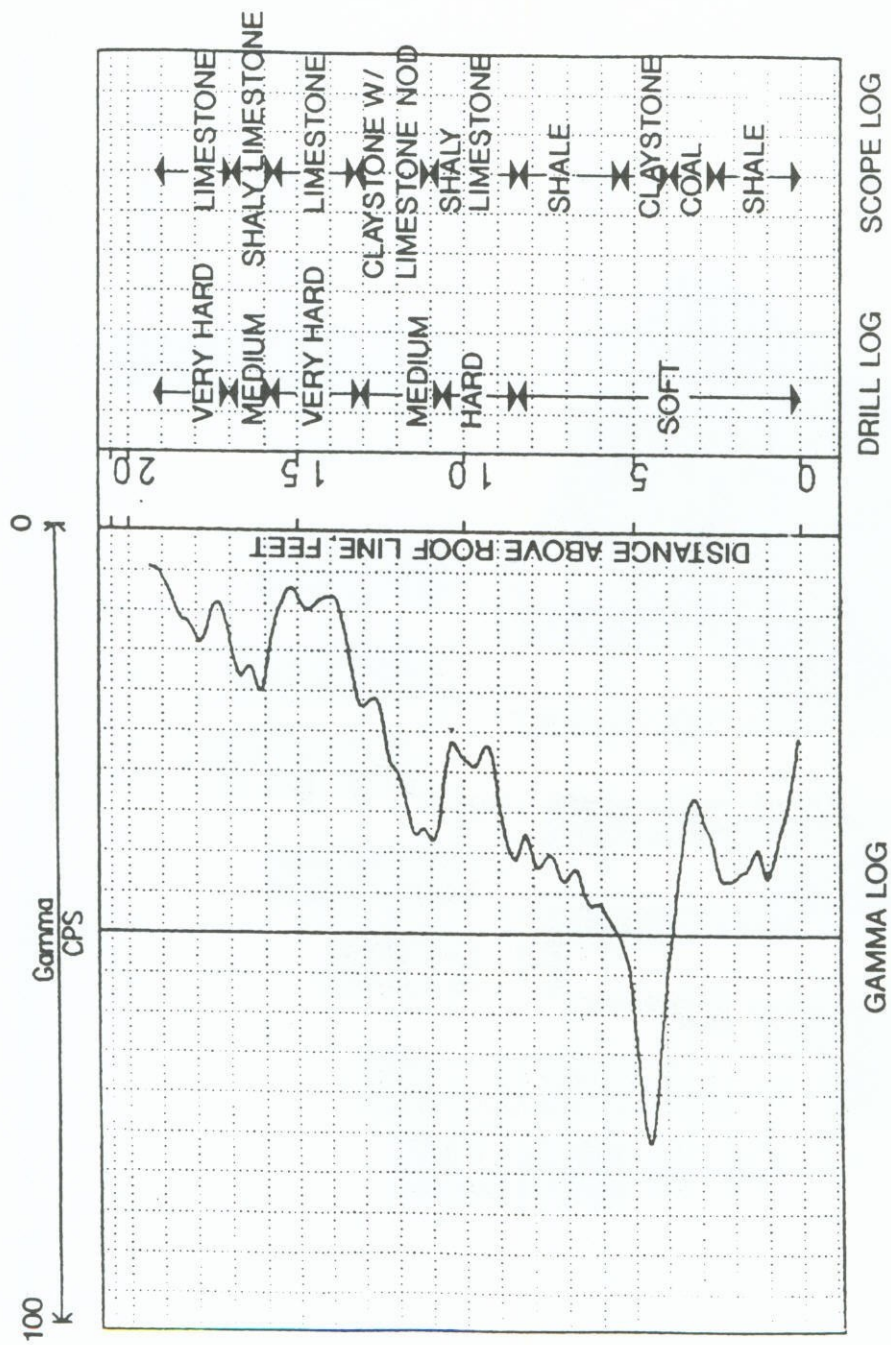


Figure 2. A typical gamma log, drill log, and fiberscope log along a 21-foot deep roof hole.

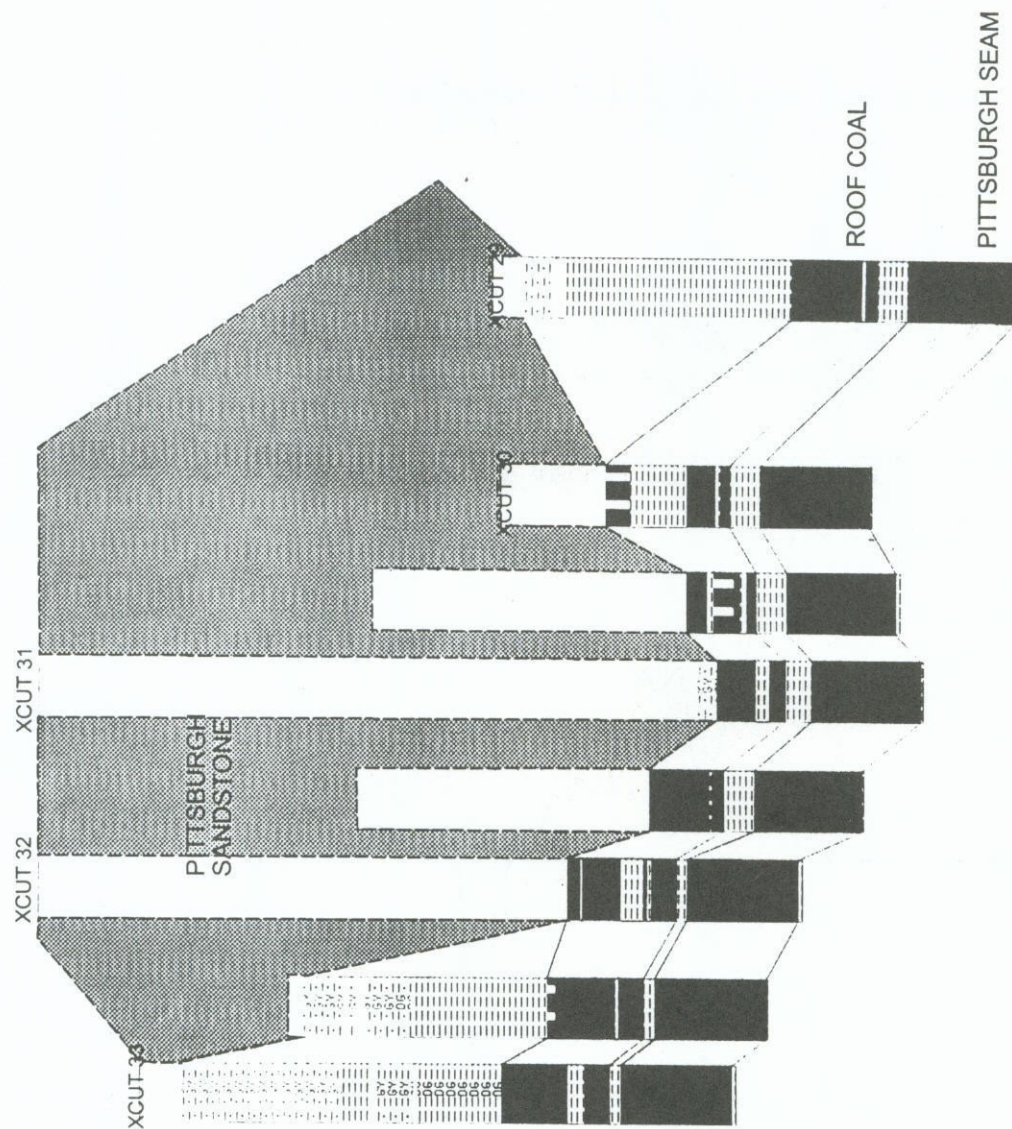


Figure 3. A roof geology cross section along a section of a Pittsburgh-seam longwall development entry.

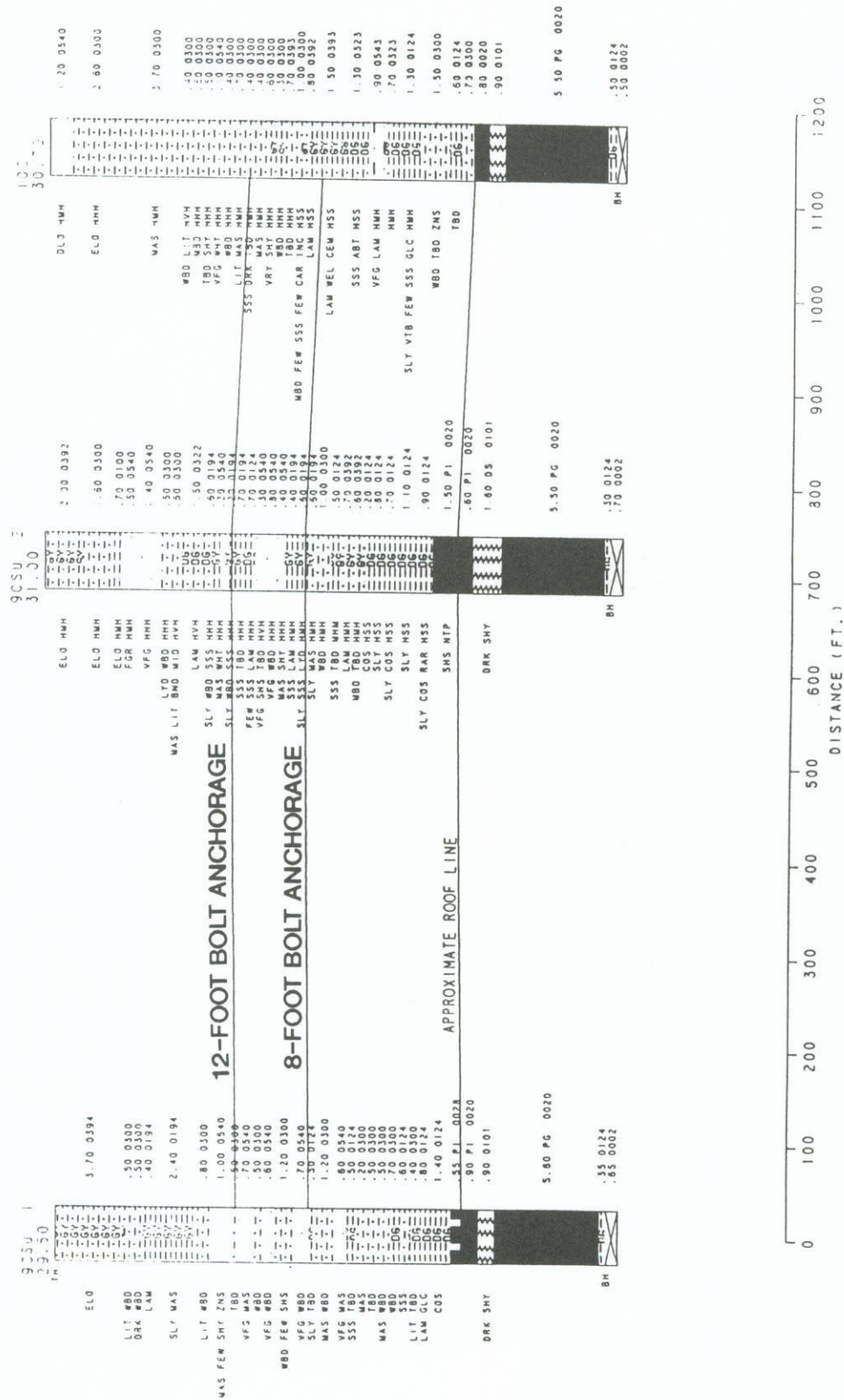


Figure 4. A geologic cross section along a Pittsburgh-seam setup entry with potential 8- and 12-foot deep anchorage horizons.

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Electromagnetic Seam Wave Mapping of Roof Rock Conditions Across a Longwall Panel

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ABSTRACT

The mining industry would benefit greatly by imaging geologic conditions well in advance of mining. In layered deposits such as coal, trona, quartz, and potash, natural waveguides form and enable electromagnetic seam waves to travel great distances. In the early 1980s, the Radio Imaging Method (RIM) was developed to capitalize on electromagnetic seam wave propagation in the assessments of seam conditions in longwall panels. The initial applications of RIM tomography and comparisons to in-mine mapping of geologic conditions proved that faults, paleochannels, and rapidly thinning coal could be detected and imaged with seam waves. This paper describes the application of electromagnetic seam waves in mapping the margins of a paleochannel crossing a longwall panel and the interesting possibility of adopting the newly developed Full Wave Inversion Code (FWIC) (Newman 1995) to significantly improve imaging resolution. When tomography images are combined with ground control science, a significant reduction in cost, roof fall potential, and waste rock in underground mining may be realized.

INTRODUCTION

In the mid 1970s, theoretical studies of electromagnetic wave transmission phenomena in underground mines determined that the already installed electrical conductors, such as ac power cable, hydro/air pipe, and conveyor belts, form natural waveguides for low attenuation rate transmission of radio signals (1)(2). During our development and demonstration of an inductive radio communications system in the York Canyon Mine near Raton, New Mexico, we discovered that coal seams also form a natural waveguide for low attenuation rate transmission of radio signals (3). Subsequent tests showed that electromagnetic seam waves also form in trona, potash, and quartz deposits. Radio communications distance in coal seams exceeded 700 feet and confirmed the 1962 prediction by Jim Wait that natural waveguides exist in the layered earth (4). The natural waveguide exists because the resistivity of the bounding sedimentary rock layers is oftentimes three orders of magnitude

less than the resistivity of coal, trona, quartz, or potash (10,000 ohm-meters) (5). Our rediscovery of the natural waveguides led to the idea of applying electromagnetic seam waves in the detection and imaging of geologic anomalies ahead of mining (6) (7) (8) (9) (10) (11) (12) (13).

Because a full seam fault crosses the York Canyon Mine main entries, a confirmation experiment was set up to measure the electromagnetic wave propagation phenomena of transmission reflections near a geologic anomaly. A transmitter was located outby the fault, and a hand-carried receiver was used to measure electromagnetic seam wave intensity in the outby entry and through the fault. The seam wave spreading (amplitude decay with distance) was determined to be to the half power of distance. If the seam waveguide were not present, the decay would have been measured to be the first power of distance. This confirmed the existence of a seam wave. The decrease in spreading factor makes a dramatic difference in instrumentation design requirements because less transmit power would be required to reach a measurements receiver. Approaching the fault from the transmitter side, the receiver measurement of the magnitude of the total electromagnetic wave (sum of the incident and reflected wave) rapidly increased in front of the fault and significantly decreased inby the fault. This experiment proved that electromagnetic seam waves were reflected back toward the transmitter by the fault. Seam waves could be used to detect faults ahead of mining. Although faults were a definite mining problem, the Raton Basin coal deposit exhibited a number of other difficult mining problems. Because the coal-peat formed in a delta region, paleochannel flow during the seam burial episode cut into the mudstone sedimentary layer overlying the coal seam. Oftentimes, these sandstone channels scoured into the coal seam resulting in partial to full seam washouts. Because of the meandering nature of paleochannels, it was impossible for the mine geologists to predict where high-energy cutbanks and scouring would occur. The difficult mining condition is illustrated in figure 1.

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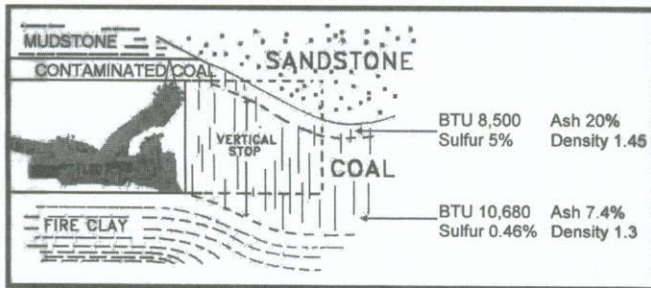


Figure 1. Vertical cross section illustrating a paleochannel scour, roll, and vertical cut stop limit in a coal seam.

Along the margins of the channel, the mudstone sedimentary rock is fractured by the mechanics of differential compaction occurring during burial. Fracturing leads to weak roof and rock falls. Waste rock increases in the run-of-mine (ROM) coal. Because mining machines are not equipped with cutting-edge sensors on the drum, machine operators cannot judge the distance to the roll and, therefore, cut into the sandstone roof rock. This causes out-of-seam dilution, decreases ROM coal quality, and introduces silicon dust into the ventilation airstream.

Although it was a simple in-mine demonstration problem to show that faults can be detected with electromagnetic seam waves, it was another and more challenging matter to show/prove that paleochannels and the associated fractured roof margins could also be detected with electromagnetic seam waves. To approach the solution to this detection problem, theoretical studies were conducted by David Hill, which resulted in the determination that seam wave attenuation rate strongly depends on roof/floor rock resistivity. These data are shown in figure 2 (14).

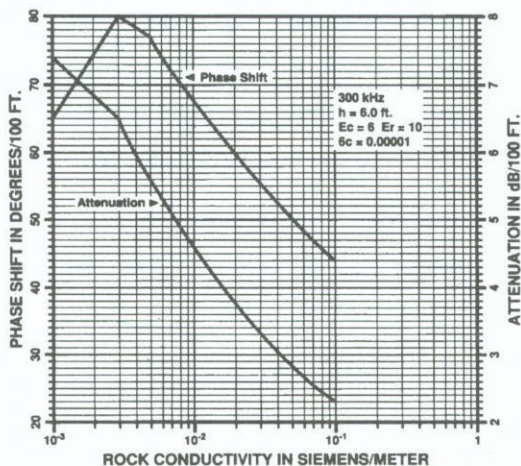


Figure 2. Seam wave attenuation rate versus boundary rock conductivity (after Hill).

Hill graphically illustrated that the attenuation rate rapidly increases under sandstone roof rock where sandstone resistivity is approximately 100 ohm-meters as compared to mudstone with

10 ohm-meters. Electrical conductivity used by Hill in figure 2 is the inverse of resistivity and has been given the designation of Siemens per meter (S/m). Now that the theoretical possibility of detecting paleochannels in advance of mining was advanced by Hill, it was left to in-mine trials to validate the use of electromagnetic seam waves in mapping paleochannels ahead of mining.

DATA COLLECTION AND INTERPRETATION

Instrumentation was developed for electromagnetic seam wave data collection. First, in-mine instruments were developed and then followed by down-the-hole instruments. The application of these instrument configurations is illustrated in figure 3.

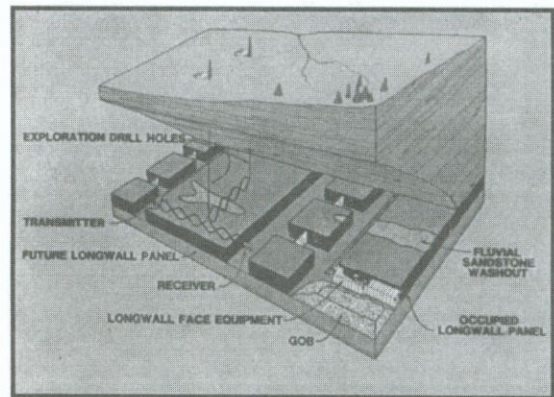


Figure 3. Adaptation of the Stolar-RIM instrumentation in a longwall mine.

The transmitter features a loop antenna that initiates electromagnetic seam wave. The seam wave is broadcast (propagates) through the coal layer and appears as an attenuated electromagnetic wave at the receiver location in the tailgate entry. The quasi transverse electromagnetic (TEM) wave is polarized by the waveguide to have a vertical electric (E) field component and a horizontal magnetic (H) field component. There is a third electric field component that is horizontally polarized and reaches maximum value at each coal-sedimentary rock boundary. In a uniform seam, this horizontal electric field has zero value in the center of the seam. The horizontal electric field component is not measured in routine surveys, although it can be made a diagnostic tool in assessing roof rock stability conditions along an entry. This possibility could lead to a simple method of assessing the potential for rockfall in developed entries in a layered deposit.

The mapping of a paleochannel across a longwall panel was achieved by conducting a tomographic survey in a longwall panel as indicated in figure 3.

Measurements made at multiple receiver stations were processed in the Algebraic Reconstruction Technique (ART) tomographic algorithm to map the radiowave attenuation rate in the plan view plane of the block of coal. The ART

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algorithm assumes that the radiowave energy travels along a straight path between the transmitter and receiver. Although the assumption is not valid in a highly disturbed geologic zone, slightly anomalous zones in an otherwise uniform coal seam are accurately mapped with RIM. The image of a roof rock fracture zone under a paleochannel in a longwall panel of coal is illustrated in figure 4.

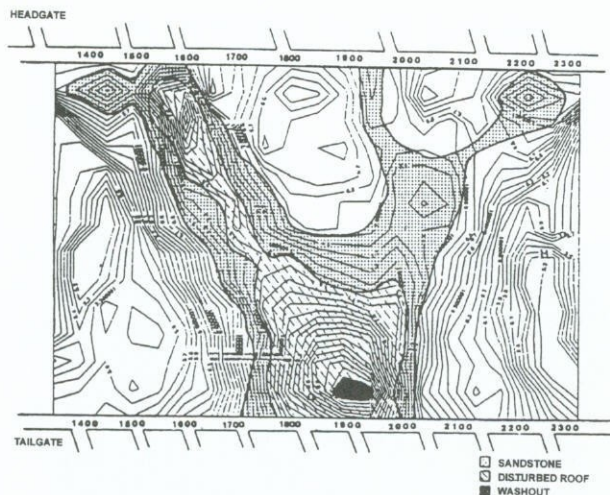


Figure 4. Reconstructed tomography image illustrating constant attenuation rate (in decibels per 100 feet) contours of a geologically disturbed zone in a 600-foot (181-meter) wide retreating longwall coal panel.

Early detection of anomalous seam conditions prior to mining has proven to be cost effective in mine design, planning to ensure a smooth-running mine, and enhancing ROM coal quality. The EM field propagation constants along each path were determined by measuring the magnitude and phase shift at each receiver location. The measured data was subsequently processed in the ART tomography algorithm that first divides the geologic image zone into pixels (plan view two-dimensional image). The tomography algorithm then determines the attenuation rate and phase shift occurring in each pixel based upon the measured data. Applying a contouring program, an image of attenuation rate or phase shift variations throughout the longwall panel illustrates the geologically disturbed zone. The plan view longwall panel image illustrates contours of constant attenuation rate of the seam wave propagating in the coal seam waveguide. To interpret the image, the fractured coal seam waveguide allows energy in the electromagnetic wave to leak out of the waveguide and into the surrounding fractured rock as predicted by Hill. If water is contained in the pore space of the overlying sandstone, its leakage into the coal fractures causes the coal's electrical conductivity to increase. Both conditions cause the attenuation rate of the seam wave traveling through the anomalous zone to increase. The rapid change in attenuation rate (gradient) illustrates the margin of the river channel and the location of fractured roof rock. A "roll" in the coal seam begins along the margin of the channel. The black region of very high attenuation rate locates a zone of rapidly thinning

coal. High energy flow in the paleochannel has scoured the coal, producing a localized washout in the seam.

The practical application of ground control science can be improved when combined with electromagnetic seam wave imaging. The impact on mining under paleochannels can be significantly reduced by leaving a bridge coal layer thickness of 12 inches along the channel margins. This will minimize the waste rock caused by rock fall into the ROM product. Additional roof support should be added in the ground control plan where the gradient increases across the entries.

CONCLUDING REMARKS

Several hundred RIM surveys have been conducted since 1983. This practical experience resulted in the recognition of technical limitations that, if overcome, would create an effective seam mapping technology. Since the original development of electromagnetic seam wave technology, longwall panels have increased in width from 500 feet (152 meters) to more than 1,000 feet (304 meters). To achieve greater operating range, the RIM operating frequency was reduced from 500 kHz to 50 kHz, which decreased image resolution. To overcome the limitation, the instrumentation has been redesigned to include synchronous detection, which significantly increases the receiver threshold sensitivity. The increase in sensitivity restores operation at 500 kHz.

The application of RIM in seams with multiple anomalies and larger blocks of coal produced images with poor resolution. In the case of multiple anomalies, the straight ray path assumption was invalid resulting in detection failure. Full-Wave Tomography Inversion Code (FWIC) processing algorithms are currently under development by industry, university, and national laboratory researchers. These FWIC tomography algorithms do not require the straight ray path assumption and can resolve multiple anomalies in the anomalous zone. Because these algorithms require accurate amplitude and phase measurements along with the measurement of multiple radiowave field components (caused by the scattered wave), a new instrumentation system is required—called Stolar-RIM. Our company and Sandia National Laboratories are participating in a Cooperative Research and Development Agreement (CRADA) in the development of the Stolar-RIM instrumentation, which features synchronous detection and FWIC interpretation algorithms.

The limitations in RIM technology have been overcome by state-of-the-art improvements in instrument design and the development of the FWIC software (15). The limitations in the ART tomography as well as the significant improvements in the FWIC tomography are illustrated in figure 5. In this geologic simulation model, the Stolar-RIM transmitter's stations are located on the left side of the image and the Stolar-RIM receiver measurement stations are located on the right side of the reconstructed image. The model to be imaged represents a sandstone washout and is shown in the upper left illustration. Present-day ART imaging results in image distortion in the direction of wave propagation as shown in the upper right corner of figure 5. The early ART images had very poor lateral resolution, therefore, predicting the exact location of

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washout or faults heading perpendicular to the mining face would not be possible. The lower right FWIC tomography image shows the significant improvement in resolution and the possibility of detecting faults and paleochannels perpendicular to the face with a heading. FWIC appears to be a quantum leap forward in coal seam imaging. We anxiously await in-mine data collection with Stolar-RIM instrumentation and processing of collected data to confirm the improvement in imaging.

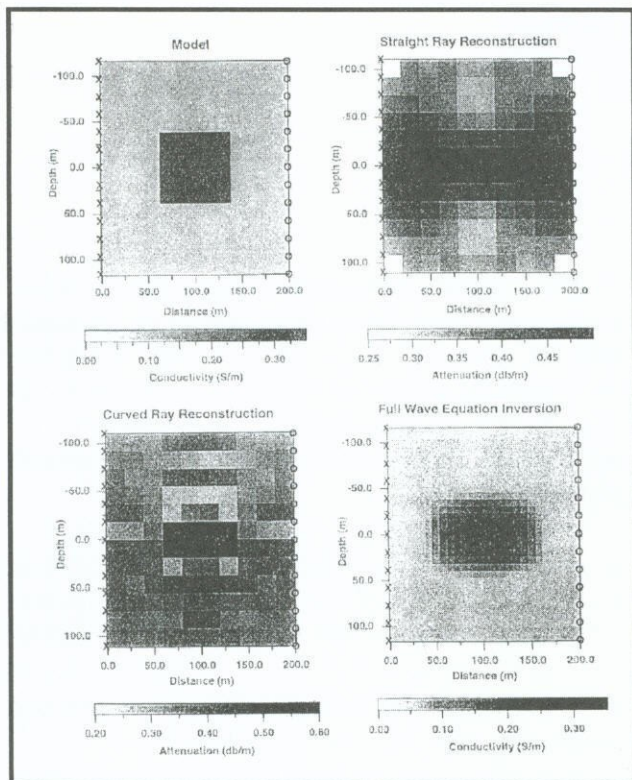


Figure 5. Comparison of reconstructed tomography images using straight ray path assumptions and the state-of-the-art full wave inversion technique developed by Gregory Newman.

ACKNOWLEDGEMENT

We would like to dedicate this paper to the memory of James R. Wait for his many contributions to the development of our understanding of how electromagnetic waves propagate in the earth.

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Geological Conditions at Continuous Miner Sections; Examples from Marrowbone Development Company, Mingo County, West Virginia

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ABSTRACT

Marrowbone Development Company operates a large drift mining complex in the central Appalachian coal field. In 1997, five continuous miner supersections produced close to 9 million tons of raw plant feed. All production is from the Coalburg seam, a 5 to 10 ft thick coal seam with multiple coal benches and shale binders. The immediate roof lithologies consist of laminated shales, sandstones, or a mixture of sandstones and shales. The mine area is bisected by the regional Warfield anticline and associated Warfield fault. Mining conditions vary considerably due to frequent changes in seam and roof geology. The following geological categories can be distinguished: 1) presence of coal riders in the immediate shale roof, 2) excessive seam slopes around the Warfield fault, 3) splay faults associated with the Warfield fault, 4) zones of abundant slickensides, 5) abrupt roof lithology changes (sandstone - shale), 6) splitting and merging of different benches of the Coalburg seam, 7) sandstone washouts, 8) excessive roof dilution, 9) pillar sizing problems due to seam anomalies (soft fireclay binders). Several examples of these features are discussed in detail. Also, their relationship to roof fall occurrences is evaluated. Early recognition of potentially adverse geological conditions is of prime importance for the economic viability of the mining plans. Detailed core drilling in advance of the mining faces and coordination of the core data with underground mapping are the main forecasting tools.

INTRODUCTION

Room and pillar mining is still the preferred mining technique in the central Appalachian coal field, despite

the steadily increasing pressure from longwall and surface mines. In 1997, continuous miner operations accounted for 46% of the total West Virginia coal production. Surface operations accounted for 31%, and longwalls for 23% (1). Most room and pillar mines are relatively small and yearly production for individual mines rarely exceeds 2 million tons.

A good understanding of the geological framework of a coal deposit is one of the most important factors controlling mine plans, budgets, and ultimately the overall performance of a room and pillar mine. Good reviews of geology-induced ground control problems can be found in Molinda and Mark (2) and Shea-Albin (3).

The present account gives an overview of the wide range of geological conditions encountered at the mines of Marrowbone Development Company, a large room and pillar mining complex located in Mingo County, southern West Virginia. Marrowbone has been in operation since 1978, with an annual output of approximately 4 to 5 million clean tons. The continuous miner sections in the Coalburg seam are the backbone of the operation. In 1997, Marrowbone operated five continuous miner supersections in three interconnected mines covering an area of approximately 8 by 5 miles. All underground production is transported over an intricate system of underground and partially overland conveyor belts to a 1750 ton/hr central processing plant (Tug Valley Processing Plant).

Physiographically, the area is located in the heavily dissected Appalachian Plateau. Elevation differences between the bottom of the valleys and the top of the ridges exceed 1000 ft. The relatively flat lying Coalburg

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seam crops out halfway up the hillsides, and all mines are above drainage drift mines. The ultimate size of individual mining blocks is dictated by the topographic shape and size of the ridges. To complicate matters, the Marrowbone area is dissected by the regional Warfield anticline and associated Warfield fault. Due to the combined effect of the anticline and the fault the Coalburg in the northern reserves is located 400 ft higher along the hillsides (Fig. 1). This makes for smaller mining blocks, and harder access. The northern mines are connected to the southern areas by means of two overland conveyor belts across the fault (6000 ft and 2400 ft long).

SUPERSECTION MINING

Each supersection consists of two continuous mining machines, two bolting machines and six shuttle cars, and one three-way dump feeder. In 1997 the five supersections were not involved in pillar operations, and mine plans were constructed using a smallest center mining layout. In general, pillars are laid out on 60 by 80 ft centers in mains and 60 by 60 ft centers, occasionally down to 50 by 50 ft centers, in panels and rooms. Mining is advanced preferably over 9 to 11 entries, to maximize the efficiency of two continuous miners. The entry width is maintained at 20 ft, and under normal conditions the miners can take a 40 ft deep cut before the place is bolted. The bolting pattern is 4 bolts per row with a maximum row spacing of 4 ft. Over the years several different types, sizes, and lengths of roof bolts have been used. Most commonly used now are 4 ft (in approximately two third of the area), and 6 ft fully grouted bolts. Some sections use a staggered configuration of 4 and 6 ft bolts. Resin assisted point anchors and tension rebars are sometimes used in areas with less than optimum roof conditions.

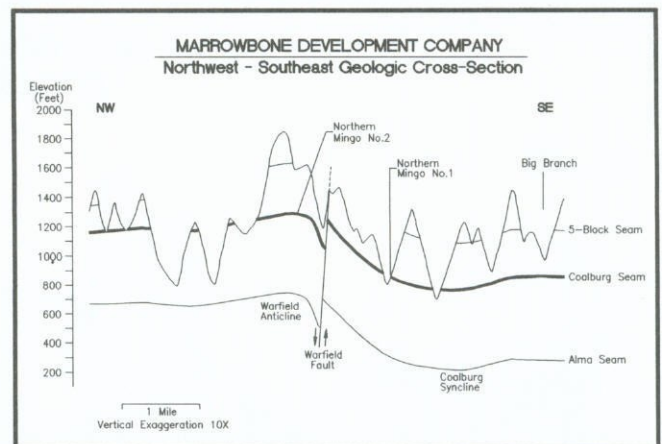


Figure 1. Generalized NW- SE geologic cross section in the area around Marrowbone Development Company, Mingo County, WV.

GEOLOGICAL SETTING

The Coalburg seam is the second largest producer in West Virginia (4). Stratigraphically, the Coalburg is situated in the upper part of the Middle Pennsylvanian Kanawha Formation (4,5). This formation consists of a cyclic succession of sandstones, shales and coal seams, interpreted as a sedimentological response to fluctuating sea levels in an environment of coastal plain mudflats and fluvial valleys (5).

The Coalburg seam at Marrowbone is a thick but dirty seam with a reject percentage ranging from 40% to 60%. Seam thickness varies between 5 and 15 feet, but averages between 6 and 8 feet for the majority of the area. The seam is a composite of several 1 to 2 ft thick coal bands separated by black coaly shale binders. The seam thickness variations are mainly caused by splitting

Roof Type	Description	CMRR
Sandstone	massive	70 - 92
	stackrock	50
Shale	thin shale, bolts anchored in sandstone	60 - 75
	silty, sideritic	45 - 60
	silty, sideritic, rider within or just above bolted interval	40 - 55
	rich in plant fossils	40 - 50
	rich in plant fossils, rider within or just above bolted interval	35 - 45

Table 1. Roof Types and CMRR Values at Marrowbone Development Company (6).

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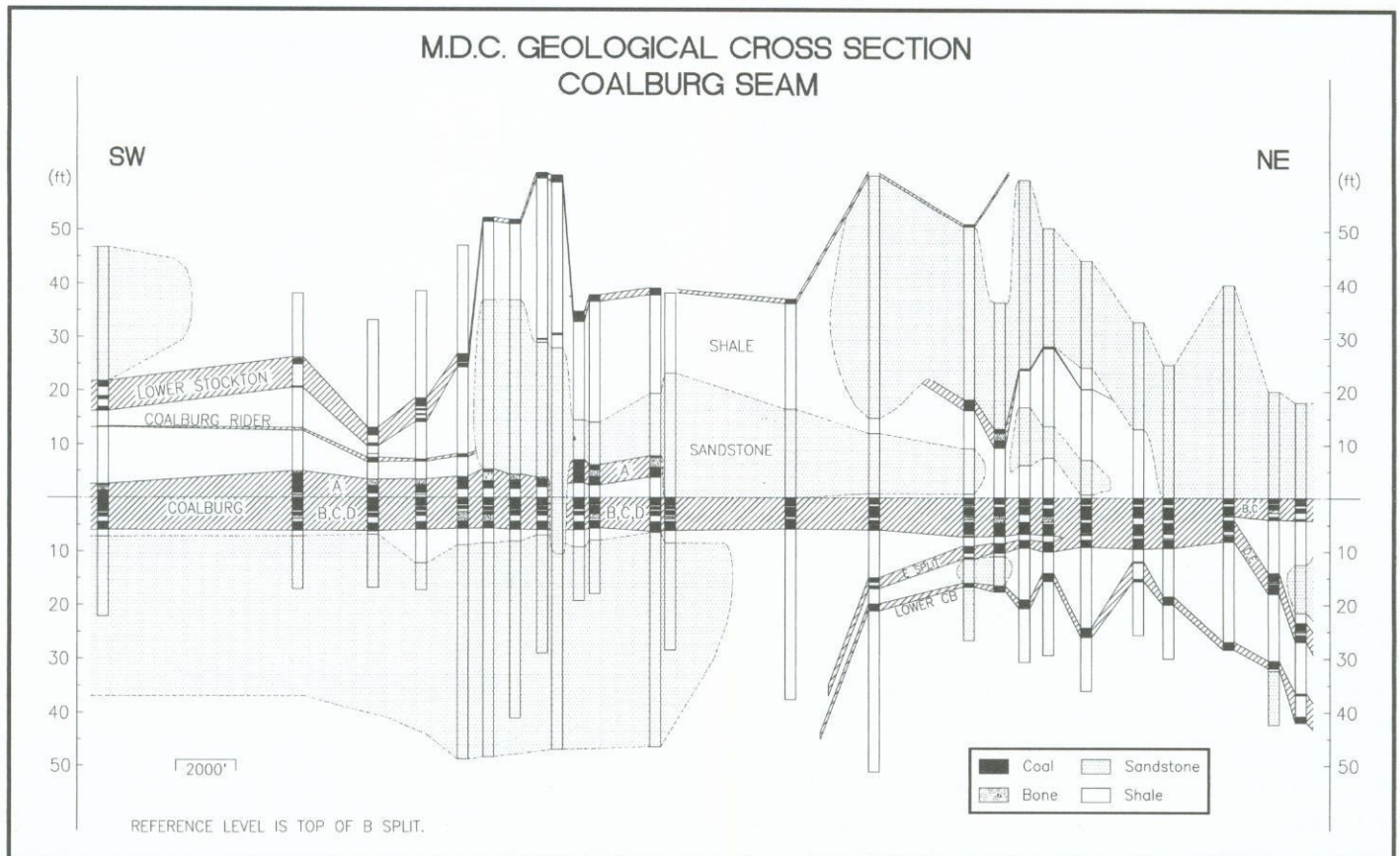


Figure 2. Detailed core hole cross section showing lateral changes in seam and roof geology, Coalburg seam.

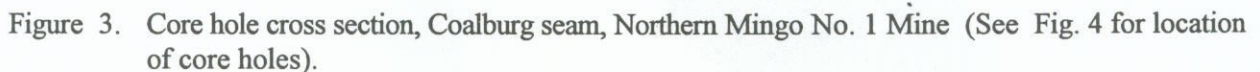
and merging of the top and bottom coal bands (Fig. 2). The roof lithologies up to the next coal interval (two thin splits of the Stockton seam) consist of a shale package up to 40 ft thick which alternates laterally with elongated zones of sandstone. The sandstones form at least two mappable units, stacked up to 75 ft thick in the central parts of the belts. Along the margins of these belts thinner sandstone sheets alternate with shale. Sandstone forms the immediate roof in approximately one third of the areas. The sandstones are generally massive and very competent. Laminated micaceous sandstone (stackrock) is locally present along the margins of the sandstone zones. The shale roof is highly variable. Most shales are medium gray with fine sideritic laminations. Horizons with abundant plant fossils may occur within the first several feet of the shale roof. The fossils include small scale leaf litter, calamites stems, and large flattened lycopod tree trunks. Rider coal seams, up to several feet thick, are common in the central part of the shale dominated domains. The shales, sandstones and Stockton series are overlain by a laterally continuous 100 to 150 ft thick sandstone package.

The CMRR's of the different roof types at Marrowbone show a wide range of values, reflecting the highly variable roof lithologies (see Table 1). Most roof types fall in the fair to good categories, as defined by CMRR values above 45 (2).

GEOLOGICAL FEATURES IMPORTANT TO MINING

Coal rider seams

Rider seams, with and without underlying soft fireclay zones can be a source of mining problems. Figure 3 shows a core hole profile along the mains of Northern Mingo No. 1 Mine. Two rider seams are present in the first 10 ft of roof shale: the 6 inch Coalburg Rider, and the two foot thick Lower Stockton. The core data suggest a gradual thinning of the interburden between the Coalburg rider and the Coalburg seam from 8 to less than



Upon entering the area it became rapidly apparent that the behavior of the rider seams was not as regular as expected. The interburden thickness to the first rider could change from 7 to 3 ft and back over short distances, causing roof falls and forcing the section to change direction. The bolting crew monitored the distance to the

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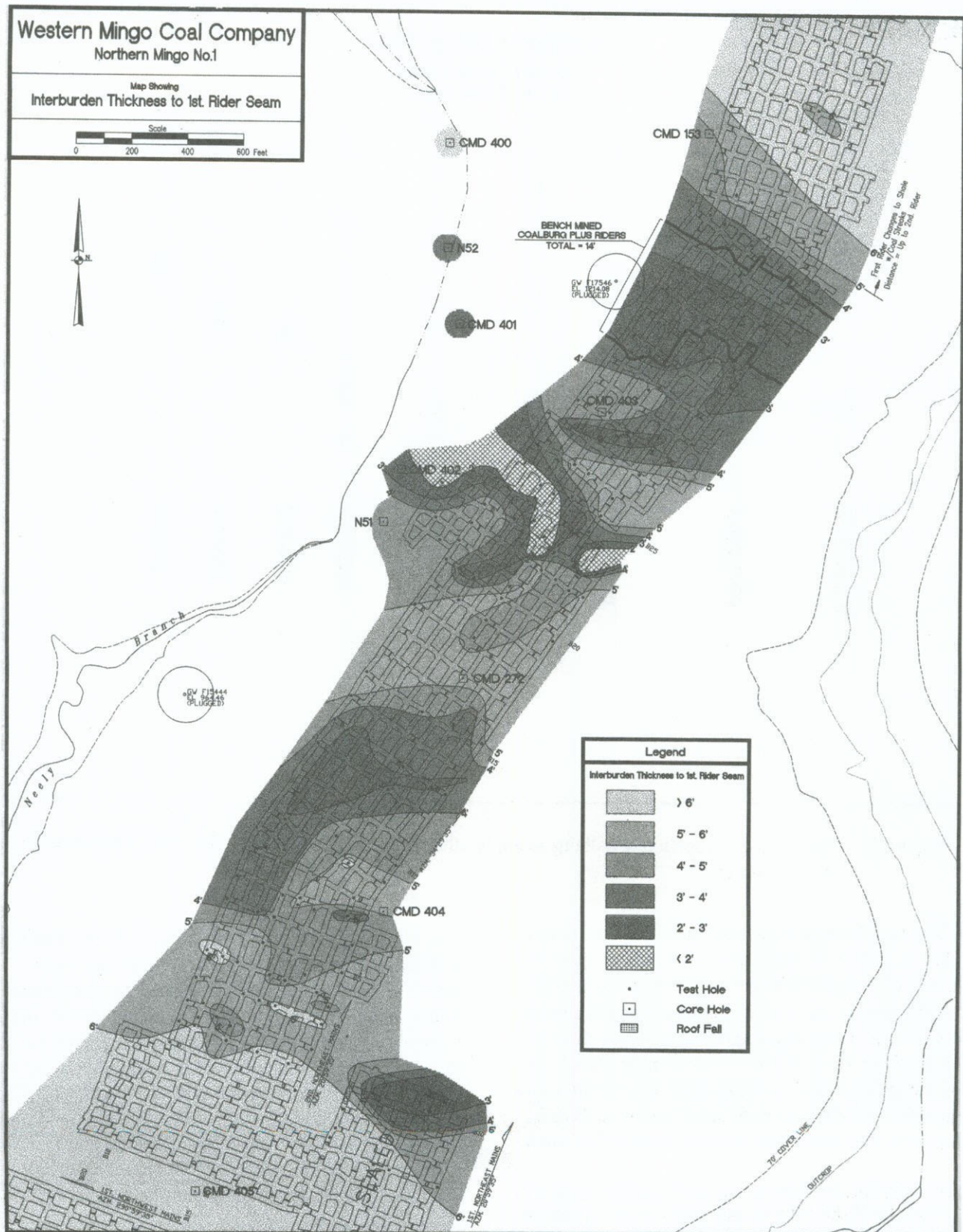


Figure 4. Isopach map of interburden between Coalburg seam and the 1st rider seam, based on underground testhole drilling, Northern Mingo No.1 Mine.

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Seam slopes around the Warfield Fault.

The southern limb of the Warfield anticline is cut by the Warfield fault. This is a normal fault, dipping 45 degrees North, with offsets ranging from 100 to 250 feet. The strata show a gradual increase in dip from 0 to over 25 % over a distance of 1000 to 2000 feet adjacent to the fault. Slopes up to 10% pose no serious problems for uphill mining (South of the fault), as well as downhill mining (North of the fault). Beyond a 15% slope, however, it is too steep for the equipment to operate adequately.

Faulting associated with the Warfield Fault

Subsidiary faults associated with the Warfield fault have been encountered at several places, up to a distance of several thousand feet both north and south of the main fault. These faults are predominantly normal faults with a 30 to 60 degree dip, mimicking the Warfield Fault. Offsets can range from a few inches to up to 15 feet. The passage through these faults is a major mining hurdle, but also the long term upkeep of the immediate footwall strata along the fault line can be a problem. The fault plane is generally very sharp, and slickensided.

Figure 5 shows a cross section and plan map of one of the largest splay faults encountered. The fault affected all entries at the junction of two mains, and showed offsets in the 15 ft range along a 40 degree dip. This feature was not forecasted by lineament studies or drill hole data. Seam elevations based on the drill hole model vary widely in this area due to the effect of the Warfield anticline, and they easily mask a 15 ft fault offset. The

fault trends NW, and is located in an area where the main Warfield fault (250 ft offset here) makes a major turn from a northeastern to eastern direction.

Zones of slickensides

Low angle slickensides without apparent offset of strata (often referred to as slips) can create very irregular problem zones in otherwise stable shale roof. The slickensided plane is generally oriented in a dominant direction, but the dip and dip direction can change over short distances. Also part of this group are the horsesbacks, a set of slickensides that are arching upwards in the roof. The slips and horsesbacks occur as isolated pockets, and as semi-linear discontinuous zones that can be traced for several hundreds of feet. The origin of these slips is questionable. Frequently, they appear to be associated with lithology changes (sandstone-shale contacts) or thickness changes in the immediate roof and therefore they could be a function of differential compaction. Sometimes they occur in close association with faults, or as extensions of faults. If the slips can not be stabilized with short mining cuts, longer bolts and roof straps, they will be mined out as roof dilution.

Sandstone - shale boundaries

A lateral change in roof lithology from shale to sandstone is often accompanied by a series of slickensided surfaces (slips and/or faults) close to or coinciding with the sandstone contact. These slips are interpreted to be the result of differential compaction of the original roof strata (2). The intensity of slickensides along a shale to sandstone transition is highly variable,

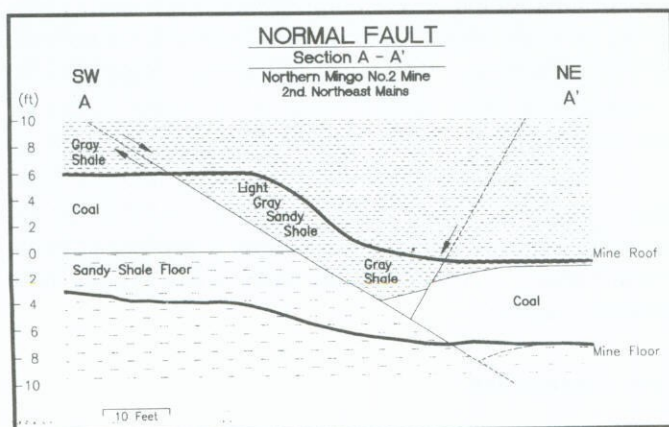


Figure 5. Cross section and plan map of normal splay faults off the Warfield fault, Northern Mingo No. 2 Mine.

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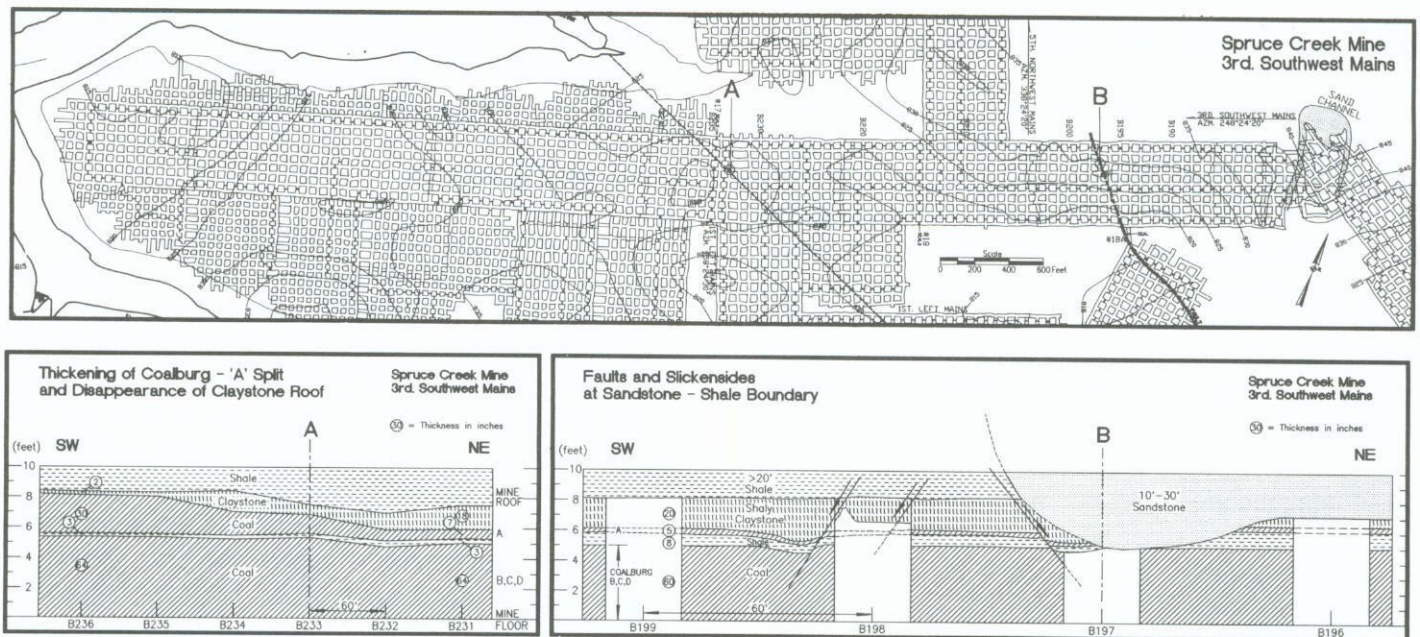


Figure 6. Cross section and plan map of roof geology changes at Spruce Creek Mine. A: claystone - coal transition in the A bench of the Coalburg seam. B: faulting and slickensides at sandstone - shale boundary

and sometimes they are even missing. The size and abruptness of the transition may be the controlling factor, a sudden transition between a thick shale package and a thick sandstone causing more disruptions than a gradual change between thin strata.

Figure 6 shows an example of an abrupt sandstone to shale contact. Note the main boundary fault and several complementary faults, dipping the opposite way, some 30 to 50 ft from the contact. The actual offsets along the slickensided faults are a few inches only. However, the footwall strata of these low angle faults and slips are notoriously prone to roof instability, especially where the trace of the fault is located just along a pillar edge (see for example the most southwestern fault in Fig. 6).

Splitting and merging of coal benches

The Coalburg seam is multi-benched. Where the top bench splits off and rides up in the roof problems may develop similar to the ones described under rider seams. As soon as the interburden between top split (the A split) and main Coalburg exceeds two feet the section will generally mine under the top sequence. Longer bolts are used to stabilize the roof until the rider has reached an acceptable level in the roof (generally 6 to 10 ft). In most cases this transition is relatively fast. The opposite

scenario occurs where a leader coal seam (the E split in Figure 2) rides up in the bottom and merges with the Coalburg. This transition is generally less troublesome and mainly results in zones with an excessive reject percentage.

Sandstone washouts

Washouts are partial or complete cutouts of the coal seam by overlying sandstone channels. At Marrowbone, small scale partial washouts in undulating sandstone roof are not uncommon. Complete washouts are rare. They range in width from 10 to 100 ft., and can be several hundred feet long. Some of the washouts appear to be arranged in a semi-linear fashion in the center of one of the sandstone belts while others form irregular isolated pods. The channel sandstones generally show a high angle internal crossbedding. They are too hard to be cut by continuous miners, and the section either has to mine around them, or mine through them using drilling and blasting methods.

Low overburden

Roofs in drift mines, by nature, are affected by low overburden problems. Mountain cracks (mud seams), water seepage, and incipient weathering of the roof strata



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can lead to an overall deterioration of the support quality of the roof strata (7). At Marrowbone, this type of problem generally starts showing up at 100 ft to 70 ft of overburden. Mountain cracks come in different shape and sizes. The joints generally show one preferred orientation, often parallel or at sharp angles to the outcrop. The complementary set at right angles is mostly poorly developed. Underground mapping shows that the spacing between the joints is often very regular (50 to 70 ft).

Excessive roof dilution

Efficient control of out of seam dilution is an important challenge for any continuous miner section in shale roof. Excessive amounts of roof, or bottom dilution can have a significant economic impact on the yield and overall performance of a continuous miner section. Roof dilution in the 6 to 18 inch range is sometimes unavoidable in some of Marrowbone's shale roof. Examples are thinly laminated black shales with abundant fossil leaf litter, soft claystone layers, or thin weak shale under sandstone. These rocks are prone to weathering upon the action of moisture laden airflows. Sometimes the separation plane is only a minor coal streak, a fraction of an inch thick, which can be missed easily upon core logging. An example of the drastic impact of involuntary roof dilution is shown in Fig. 6. At this section, the top split of the Coalburg consist of a banded/laminated bright coal that changes laterally into a claystone. The change is gradual, with an increasing abundance of shale and clay laminations, and is completed over a distance of 150 ft. This relatively abrupt transition from a thick Coalburg with stable shale roof to a thinner Coalburg with 2 feet of roof dilution has a major impact on the overall product yield (a drop from 55% to 40%).

Pillar sizing problems

Minimum pillar sizes at Marrowbone are calculated using the standard ARMPS approach (8). Stability factors in the 1.70 to 2.00 range are generally satisfactory for short term to medium term mining panels. Smallest pillar sizes are 40 by 40 ft, 40 by 30 ft, and sometimes 30 by 30 ft. On several occasions pillar stability problems developed in small blocks where the Coalburg contained very weak clayey binders, resulting in pillar shifting, rib spalling and subsequent roof failure. Following this experience the minimum safety factors have been raised a notch in areas with anticipated seam anomalies, and 50 by 50 ft center mining has been kept

to a minimum. Similar problems influencing the coal pillar strength are reported by Mark and Barton (9).

ROOF FALLS

The wide range of geological characteristics and anomalies is reflected in an evaluation of roof falls. At Marrowbone, the roof fall frequency for the entire operation is approximately 25 falls per year. The MSHA reportable falls are tracked on a mine map and all physical characteristics of the falls such as size, shape, location, type of bolts used, roof lithology, development date and fall date are entered into a database. Most falls are inspected in detail by the section crew and geology/engineering staff in an effort to determine the probable cause for the roof failure. The roof falls at Marrowbone often occur as clusters of falls, rather than as isolated occurrences, and most falls in a single cluster show comparable geological characteristics. Figure 7 shows a summary of factual and interpretive data derived from the roof fall database, including fall location, standup time, bolt type, and likely fall cause.

Most roof falls at Marrowbone (73%) occur in intersections, rather than in entries or crosscuts. The maximum mining width is 20 ft, and the span width of the intersections, expressed as the sum of the two diagonals, varies between 59 and 65 feet, with an average of 63 ft (6). No relationship is apparent, however, between span width and roof fall frequency.

The time period lapsed between development and fall date is highly variable (Fig. 7). Approximately 50% of all falls occur more than ½ year after development. It should be noted that most of the mining panels are sealed off after development, and most of the falls in this group reflect mains and submains. Only 12% of all falls occur within 10 days of development.

The roof bolts used as original primary support in the roof falls are predominantly 4 ft fully grouted bolts (in 48% of all falls, Fig. 7.). This is not surprising since 4 ft bolts are the most widely used bolts at the Marrowbone mines. Also noteworthy is the relatively large number of falls associated with resin assisted point anchors. It is hard to draw conclusions from the data but overall it seems that areas with weak strata or shale to sandstone boundaries at or slightly above the 4 ft level are relatively susceptible to roof problems. Since

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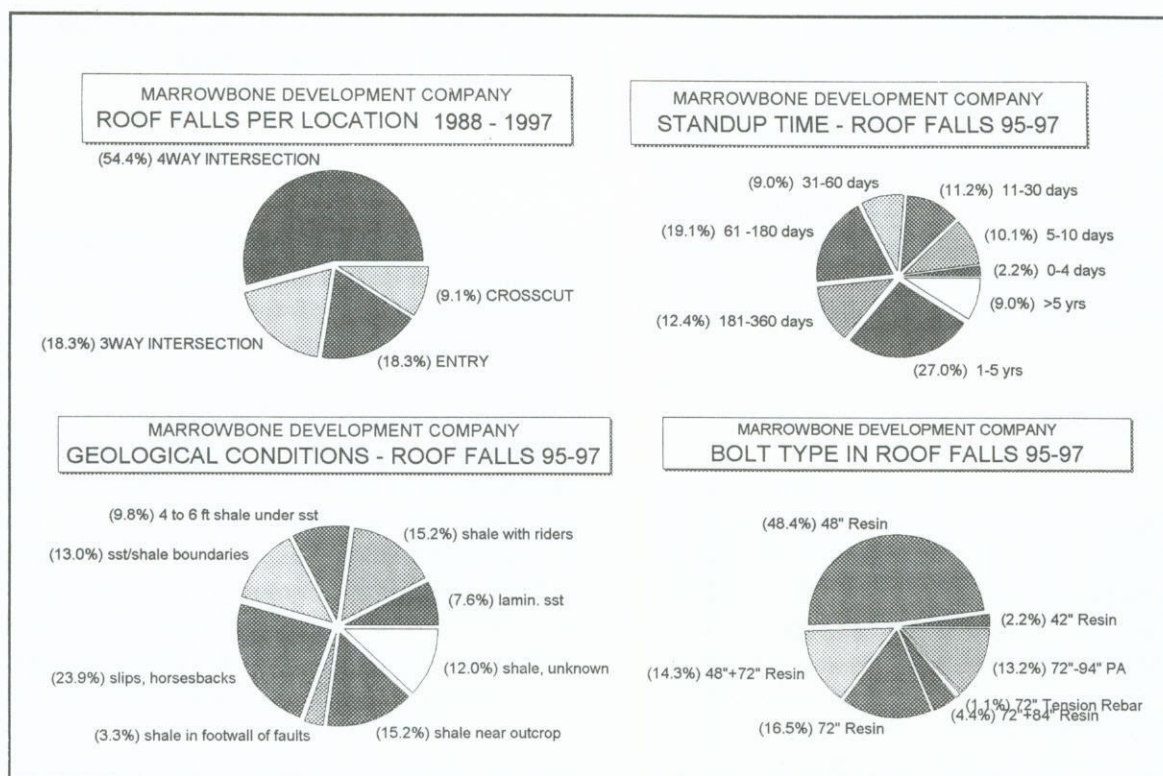


Figure 7. Summary data of reportable roof falls, Marrowbone Development Company.

roof geology can change quickly, it is a challenge to recognize the potential problems in a timely fashion.

An attempt was made (Fig. 7) to arrange all falls of the 1995-1997 time period according to geological and other physical conditions listed to be the main cause of the roof failure. Close to one half of all falls are attributed to the occurrence of specific relatively weak strata in the roof, and the other half to structural geological anomalies such as faults, slips, boundaries, and low overburden (less than 100 ft). The largest and least well defined group of falls (24%) is formed by slickensided low angle slips and horsesbacks in otherwise stable shale roof. This group is followed closely by the shale with riders (15%), sandstone - shale boundaries (13%), and low overburden (15%) groups. Overall, the roof fall evaluation reflects the wide array of geological features present at the Marrowbone mines.

FORECASTING STRATEGIES

Adequate geological information is a basic requirement for sound economic decisions involving mine

planning and budget plans. The two most important tools for collecting geological data are core drilling and underground mapping. The last step, which is often overlooked in coal mining, is an essential link in converting core hole data into a valid forecasting tool.

Before a mining block is opened up, drill core spacing along the projected mains at Marrowbone is typically reduced to around 1500 to 1000 ft. This relatively tight spacing of drill holes provides an excellent geological framework and serves well in predicting seam quality, seam geology and overall roof and floor conditions. It is not sufficient, however, to accurately forecast small scale features such as washouts, rolls, slips and faults. Careful underground mapping of existing problem zones provides valuable information to better utilize the core data as a forecasting tool. Electrologging, geotechnical testwork on core samples, lineament studies, and geophysics are also used to complement the geological data. Lineament studies at the Marrowbone complex have identified the major through going structural features such as the Warfield fault and associated structures, but they have been unsuccessful in predicting the smaller fault splays and other roof disturbances.

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CONCLUSIONS

The examples from the Coalburg seam at the Marrowbone mines illustrate the wide range of geological features and anomalies encountered at continuous miner sections. Some are of structural nature, such as faults, slips, and seam slopes, and are essentially caused by the proximity to the regional Warfield fault. Others are more related to lithologies, such as weak shale roof, rider seams and sandstone to shale boundaries in or slightly above the bolted interval. A thorough evaluation of roof fall data provides good clues on past problems and bolting practices. These data, along with drill hole information, and underground mapping of problem zones are essential forecasting tools for future mining blocks.

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The Role of Engineering and Geology in Analyzing Ground Control Conditions

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ABSTRACT

Severe roof control problems have plagued a West Virginia underground mine since its initial development in the late 1970's. Adverse roof conditions in the Eastern portion of the reserve result from a combination of mining:

- perpendicular to the major principal horizontal stress direction,
- near lineament intersections,
- in areas where the ratio of horizontal stress to vertical stress exceeds 2.00 to 4.00, and
- where laminated strata are present in the immediate roof.

The focus of this study is to identify the engineering (mine orientation, roof safety factor) and geological parameters (lineament and horizontal stress orientation, horizontal to vertical stress ratio, and immediate roof lithology) causing poor roof conditions and to quantify their influence on projected mining conditions in the Western portion of the reserve.

Rose diagrams were used to illustrate the correlation between the orientation of eight hundred thirty-six (836) roof falls with that of forty-four (44) lineaments and the principal horizontal stress. Similarly a composite map was used to demonstrate the relationship between roof falls and the ratio of horizontal stress to vertical stress. The immediate roof strata across the property were categorized as either laminated or non-laminated based upon a thorough examination of one hundred sixty-two core logs. The presence or absence of fireclay, a rider coal, and carbonaceous shale was also noted because of the historic correlation with poor roof conditions. Rock strength and physical property tests were conducted on selected immediate roof strata recovered from coreholes drilled along the mains projected to the Western portion of the reserve.

Potential roof control problems in the Western reserve were quantified based upon the correlations developed by analyzing engineering and geological data. The main conclusion is that laminated roof strata, the rider seam, and its associated fireclay will have the greatest influence on mining conditions. Roof problems attributable to horizontal stresses will not be as severe in the West due to an increase in overburden depth and corresponding decrease in the stress ratio. Recommendations are provided for mining orientation, the layout of mains and submains, panel location, and roof support based on intended service life.

HISTORICAL MINING CONDITIONS

The mine, located in West Virginia, operates in the Lower Kittanning seam. It was developed as a shaft/slope mine in the late 1970's and produced metallurgical coal from multiple room-and-pillar sections. Soon after completing the shaft and slope, ground control problems were encountered while driving mains due North. These mains were abandoned as a sump and a second set of mains were developed due West. The eight (8) entry West mains were driven a distance of 9,500 feet and experienced stress related ground control problems including:

- ◆ long roof falls, primarily in the North-South oriented crosscuts
- ◆ "cutter roof" associated with high horizontal stresses
- ◆ shifting of individual layers within the immediate roof
- ◆ presence of clay veins within the seam and in the immediate roof strata
- ◆ roof falls caused by "stack rock", laminated sandstone in the immediate roof
- ◆ and to a lesser extent, floor heave.

Ultimately the West Mains were abandoned because stable roof conditions could not be achieved. Poor roof continued to a lesser degree on a second set of mains that were developed in a Northeast direction. A decision, based partially on the poor ground conditions, was made to switch

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from room-and-pillar to longwall mining.

During the late 1970's through the mid 1980's the longwall panels of mine became a field laboratory for ground control research conducted by the U.S. Bureau of Mines (USBM), Pennsylvania State University, and private consulting companies under USBM contract. Many alternative methods of ground control and roof support were tested including wider and longer longwall panels, changes in panel orientation, and stress relief entries driven with a roadheader in the longwall gateroads. Mining and ground control conditions improved but the operation proved uneconomical during the rapidly falling coal prices of the mid 1980's. Under new ownership, the Eastern portion of the reserve was successfully mined using room-and-pillar methods. This area of the reserve is partially stress relieved because of proximity to the outcrop, has fewer lineaments, and experienced good to excellent ground conditions.

The investigation discussed in this paper was prompted by the exhaustion of reserves on the East side of the property. Necessity required development of the Western reserves which had been abandoned for fifteen (15) years after sealing the original West Mains. To access the Western reserve a set of "new" mains were projected parallel to the original West Mains. The orientation and location of the "new" West Mains were constrained by prior mining and a "no coal" zone to the South of the "new" West main projections.

The main objective of this study was to identify and locate areas of potentially poor ground conditions within the Western portion of the reserve. A secondary objective was to delineate the cause(s) of roof falls and poor ground conditions within the older mine workings. Once empirical correlations were established between geology, regional stresses, mining orientation, lineaments, and the historical roof problems, the correlations were extrapolated to the Western portion of the reserve.

RELATIONSHIP BETWEEN LINEAMENTS, HORIZONTAL STRESS, LAMINATED ROOF STRATA, AND ROOF FALLS

Lineaments

The orientations of the regional fracture system and of the existing roof falls were examined to identify any directional preference. The length and direction of forty-four (44) primary, secondary, and tertiary lineaments were determined. In Table 1 the number of lineaments in each category is shown with respect to the total.

Table 1

Lineament Type	Number	Percentage of Total
Primary	3	6.98%
Secondary	31	72.09%
Tertiary	9	20.93%
Total	43	100.00%

A rose diagram, shown in Figure 1, was created to illustrate the preferred lineament orientations. The preferred lineament directions are (20°), (210°) (225°), (175° - 180°), (270°), and (340°). None of these orientations are at a ninety (90°) degree angle to the principal horizontal stress orientations of N68°E (68°) and N22°W (338°) as measured by Aggson,(1980). Therefore, it can be concluded that the lineament orientations are not directly related to the horizontal stress field. However, it should be noted that forty-four (43) lineaments is a relatively small population considering the 20,509 acre size of the property. The roof fall population totaling eight hundred thirty-six (836) falls is more statistically significant in terms of examining the potential for a correlation with the principal horizontal stresses.

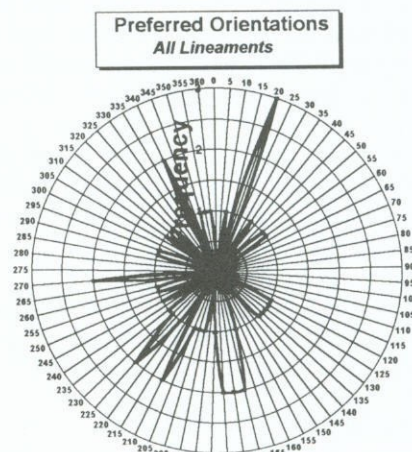


Figure 1. Orientation of All Lineaments.

Horizontal Stress to Vertical Stress Ratio

The presence of high horizontal stresses in West Virginia has been documented by underground stress measurements conducted by the U.S. Bureau of Mines (Aggson, 1979,1980, Molinda et. al., 1992) and by the preferred direction of roof falls in the old mine workings. The in-mine stress measurements (Aggson, 1979, 1980) demonstrated the magnitude of the horizontal stress exceeds the vertical stress. Poor roof conditions are anticipated where the horizontal stress is a multiple of the vertical stress. The horizontal stress acts to delaminate bedded immediate roof strata creating tensile stress



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concentrations at the roof-rib interface and "cutter roof," typical in high horizontal stress areas.

A horizontal to vertical stress ratio map, a portion of which is shown in Figure 2, was constructed to identify areas of the existing mine workings and projections in the Western portion of the reserve affected by horizontal stresses. The stress ratio map depicts:

- ◆ known roof falls within the mine
- ◆ location of primary, secondary, and tertiary lineaments
- ◆ ratio of horizontal stress to vertical stress delineated by hatch patterns

The vertical stress was established by gridding the overburden. A tight grid spacing (100 foot centers) enables good coverage of the valley bottoms where the stress ratio typically spikes by insuring multiple points define the bottom area. The inherent horizontal stress of 1,807 lb/in² was determined as the difference between the total horizontal stress measurement, 1,943 lb/in², (Aggson, 1980) and the horizontal component of the local overburden stress. The horizontal stress to vertical (σ_h/σ_v) stress ratio was contoured and distinct hatch patterns used to illustrate the wide variation in the stress ratio across the property. In the Eastern portion of the property the variation in the stress ratio corresponds with the decrease in overburden depth. The ratio was arbitrarily stopped below three hundred (300') feet of overburden because the high σ_h/σ_v ratio values were not realistic because of horizontal stress relief at the outcrop.

Historically, minimal ground control problems occur with stress ratios between 1.00 and 2.00. However, between 2.00 and 4.00, stress related roof problems including "cutter roof" and sliding along bedding planes are commonplace. Severe roof problems occur when the ratio is above 4.00. A good correlation exists between the location of the falls and areas where the stress ratio exceeds 2.00. This correlation does not hold true everywhere in the mine. The lack of absolute correlation occurs because the roof falls do not result from a singular cause but to the combination of mining orientation, laminated immediate roof strata, and the horizontal stress ratio. However, the stress ratio correlation is strong enough to use it as a criterion for future roof conditions.

From the stress ratio map, it is apparent that the highest stress ratios occur in the stream valleys and toward the outcrop where the overburden is low. In the Western reserve the σ_h/σ_m ratio is lower than in the East because of an increase in overburden depth. Except for the area surrounding two valleys, the σ_h/σ_v ratio in the Western reserve is between 1.00 and 3.00. Consequently, the potential for stress related roof problems is lower in the West compared with the previously mined areas in the East.

Examination of Preferred Roof Fall Orientations

The distribution of roof falls within the mine was examined by using the length and orientation of the roof falls shown on Figure 2. The length and orientation of eight hundred thirty-six (836) roof falls were measured on the digital mine map. Roof fall orientation was examined in terms of the raw data and normalized by the preferred mining directions. Initially a plot of roof fall orientations, shown in Figure 3, was constructed from the raw data to determine the preferred orientations. Based on an examination of Figure 3 there are two major roof fall orientations (140° and 230° - 235°) and several minor directions (50° - 55°, 90°, 180°, and 320° - 325°).

However, after examining the mine map, it is evident that there are several preferred mining directions. A question arose as to whether the preferred roof fall directions, seen in Figure 3, are statistically significant or merely a function of the mining orientation. This was resolved by measuring the length and orientation of each entry and crosscut and tabulating the data in spreadsheet format. There are 2,479,212 feet or 469 miles of entry and crosscut development within the mine workings. Of this total, roof falls are present in 59,019 feet or 2.38% of the mine openings. The information on developed footage and azimuth is summarized in Figure 4. Examining Figure 4, it is clear that almost ninety (90%) percent of the mine workings are developed on two azimuths, 50° and 140°.

To normalize the roof fall footage by azimuth, the total length of roof falls was divided by the total length of development for each five (5°) degree increment of azimuth. The result of this is that the percentage of roof fall footage was determined for each increment of orientation. The normalized roof fall percent is shown in Figure 5. Comparing Figures 3 and 5 provides dramatically different conclusions concerning the probability of initiating roof falls in specific orientations. Any orientation accounting for ten (10%) percent or more of the roof fall percentage is considered critical. Based upon the normalization approach, it is clear that mining in a +/- 15° band around the North - South direction has the highest probability of generating roof falls although total length of development in these directions is small. The most favorable mining orientation lies between thirty (30°) degrees and forty-five (45°) degrees from the direction of the major principal horizontal stress.

Some stress related roof conditions should be anticipated based on the measured stress directions of N68°E/N22°W and the historical evidence for this orientation shown in Figure 5. Several recommendations are provided for the development of the "new" West Mains.

- ◆ Under no circumstance should crosscuts be driven at a ninety (90°) degree angle to the entries since they would be on a North-South orientation.
- ◆ Crosscuts should be angled a minimum of thirty (30°) degrees from an East-West orientation of the mains.

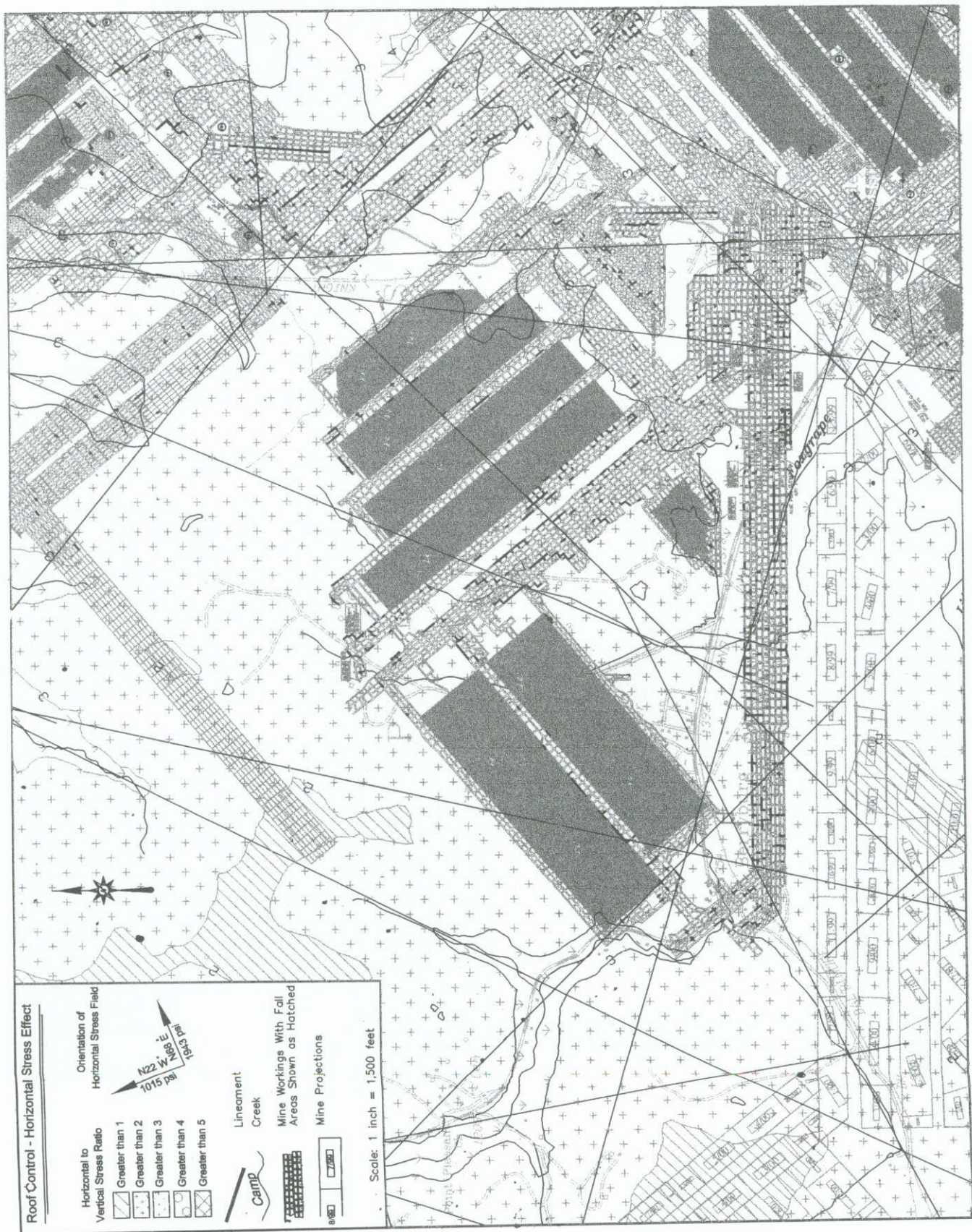


Figure 2. Horizontal to Vertical Stress Ratio

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Perferred Orientation of Roof Falls
Raw Data - 836 Roof Falls

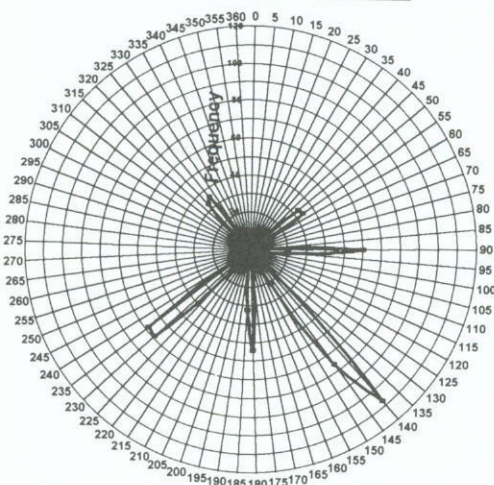


Figure 3. Rose Diagram of Roof Fall Orientation in the Mine Workings.

Percentage of Roof Falls
Normalized by Azimuth

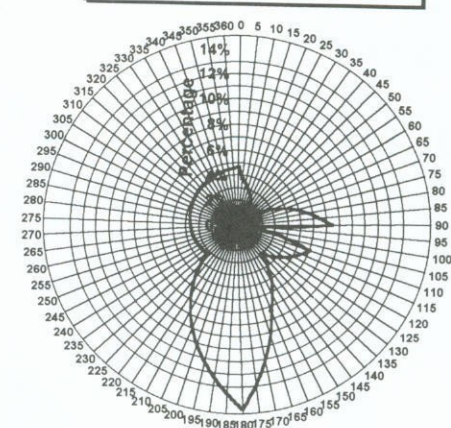


Figure 5. Roof Falls as A Percentage of Mine Development in Specific Azimuths.

Preferred Mining Orientation
All Entries and Crosscuts

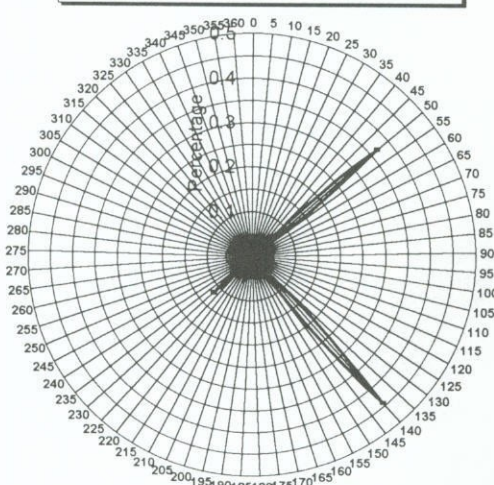


Figure 4. Orientation of Mining in the Mine.

Immediate Roof Strata and Anticipated Stability

Roof conditions in the Western reserve are partially controlled by the presence or absence of the Lower Kittanning Rider seam, an associated fireclay, laminated strata, and the strength of the individual units that comprise the immediate roof. The effect of the Rider seam and fireclay are obvious while the laminated immediate roof is more sensitive to the effect of high horizontal stresses. Geologic cross sections were constructed to define where the immediate roof strata are laminated and therefore susceptible to horizontal stress.

The result of this qualitative examination of the immediate roof strata is illustrated on a roof hazard map shown in Figure 6. Areas of poor roof conditions are denoted by laminated roof strata, Rider coal, and fireclay as determined from the core logs and descriptions. Reviewing the roof hazard map several points are evident. First, there is a strong correlation between presence of laminated immediate roof and roof falls. Laminated roof is the most significant cause of immediate roof control problems in comparison to mining orientation and to the σ_H/σ_V stress ratio. Second, a significant portion of the Lower Kittanning seam is overlain by laminated immediate roof strata, the rider seam, or its associated fireclay. In the mine workings most of the coreholes are categorized as having laminated immediate roof strata. In the Western reserve this is also the case. However, the rider seam and associated fireclay also adversely affect the immediate roof quality.

- ◆ The crosscut spacing in the "new" West mains should be increased to reduce the areal extent of exposed roof.

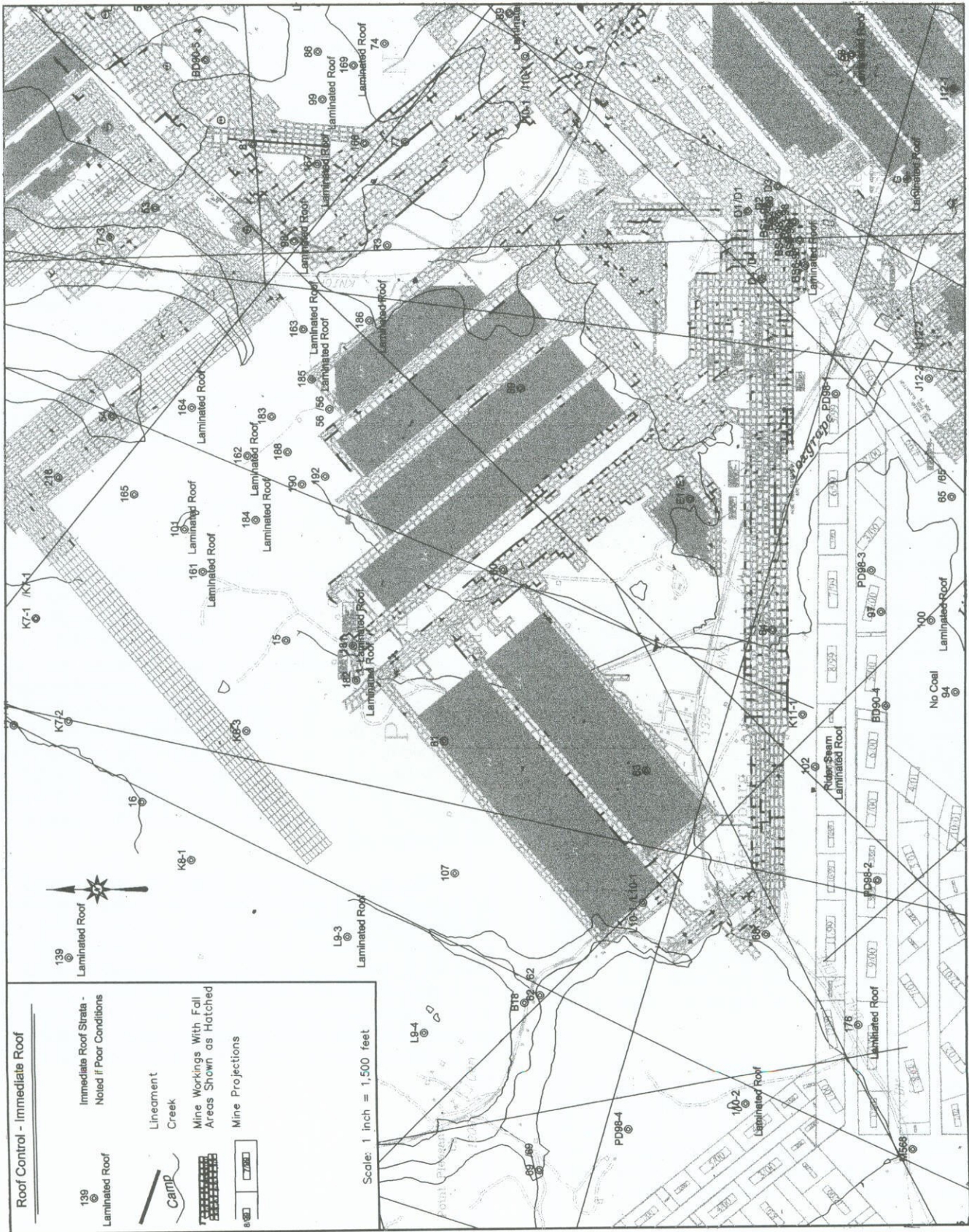


Figure 6. Distribution of Immediate Roof Lithologies.

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CONCLUSIONS AND RECOMMENDATIONS

A quantitative index of roof stability was developed using fixed end beam safety factors based upon a standard eighteen (18') foot mine opening, the tensile strength, and density of the immediate roof strata. Roof stability was evaluated assuming each lithologic unit acts independently of the overlying and underlying beds. Roof beam safety factors were calculated for the thicknesses of each stratigraphic unit. In Table 2, the fixed end safety factors were calculated for each immediate roof lithology at each of the four (4) PD98 series boreholes drilled along the proposed mains and in the Western reserve area. Using fixed end beam safety factors of 4.00 and 8.00 as criteria for short term and long term stability, the relative stability of the immediate roof can be determined from the combination of Table 2 and stratigraphic descriptions of the core. In Table 2, the main roof where present is delineated by a thick line while the immediate roof is between the thick and thin lines.

It should be noted for the shales and sandy shales, these beams presume uniformity with no laminations, fossil planes, or other discontinuities present that would reduce the beam thickness. For tensile failure, the recommended safety factor ranges between 4.00 and 8.00 (Obert and Duvall, 1967). A threshold safety factor of 8.00 was selected based on the above safety factor range and optimistic assumption that each lithologic unit is a single homogenous beam.

In Table 2 those strata with safety factors greater than 8.00 are shaded in gray to highlight the stronger zones. Roof problems can be anticipated in strata with safety factors less than 4.00. This criterion is also useful in determining which strata need to be bolted into a single composite beam. In addition to the safety factor for an eighteen (18') foot wide entry, the maximum permissible roof span is calculated at safety factors of 1.00, 4.00, and 8.00. Where the short term span (safety factor ≤ 4.00) is less than eighteen (18') foot, the lithological unit is not considered self supporting. Depending upon its location in the immediate roof, these units may either become "draw rock" or indicate units that should not be used as anchorage horizons and must be bolted with adjacent units into a composite beam. Based upon a thorough examination of the core from holes PD98-01, PD98-02, PD98-03, PD98-04 and the beam safety factor calculations, it is evident that eighteen (18') foot wide entries and crosscuts should be used in driving the proposed West mains.

Active roof support was recommended in terms of a torque-tension bolt anchored with a two (2') foot minimum of resin. The torque should be moderate (150 ft.-lbs. to 200 ft.-lbs.) with large bearing plates. Higher torques and normal bearing plates have the potential of pulling the plate into the first roof unit and cracking the intervening units between the anchorage horizon and bearing plate.

The projected mining conditions in the Western reserve were examined based upon historical conditions in the active and abandoned mine workings, analyzing the effect of mine orientation, the horizontal stress to vertical stress ratio, the nature (laminated versus massive) and lithologic composition of the immediate roof strata, examination of recently drilled coreholes (PD98-01, PD-98-02, PD98-03, and PD98-04), and roof beam safety factor calculations. The historically poor roof conditions in certain areas of the mine typically result from a combination of the following factors:

- ◆ where entries or crosscuts are perpendicular to the major principal horizontal stress direction of N68°E as measured by the U.S. Bureau of Mines, or are mined in a North-South orientation
- ◆ mining near lineament intersections, particularly under low overburden where these regional fracture systems can dilate due to reduced confining pressure
- ◆ where the ratio of horizontal stress to vertical stress exceeds 2.00 and
- ◆ where the immediate roof strata are either laminated as described in drillers logs, occurs as thin lithologic units, or where the Lower Kittanning Rider seam and its associated fireclay are within the immediate roof.

Of these items, laminated immediate roof strata has the largest effect in terms of a correlation with roof falls since the majority of the roof strata are categorized as laminated. The East-West orientation of the proposed West mains is constrained by previous mining and a "no coal" zone to the South. Historical precedent shows that although roof falls have occurred in the East-West orientation and roof problems can be expected, the North-South alignment should be avoided.

The ratio of horizontal stress to vertical stress decreases in the Western reserve as the overburden depth increases. Consequently, horizontal stresses will not be as significant in the West as they were in the Eastern portion of the mine. Minor roof problems are anticipated in areas where the stress ratio exceeds 2.00 and severe ground control problems will occur when the ratio exceeds 4.00. Because of the increased overburden depth, lineaments should not be as significant in the West as compared with the East.

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Table 2

Roof Stability Safety Factors - Fixed End Beam

Hole No.	Lithology	Interval		Beam Thickness (feet)	Safety Factor for a 18 Foot Roof Span	Maximum Roof Span for a S.F. = 1.00	Maximum Roof Span for a S.F. = 4.00	Maximum Roof Span for a S.F. = 8.00
		Top (feet)	Bottom (feet)					
PD98-01	Sh. w/LS	442.40	442.90	0.50	0.78	15.93	7.97	5.63
	Sh. w/SS Strks.	442.90	448.00	5.10	12.12	62.67	31.33	22.16
	SS w/Sh.	448.00	448.40	0.40	-	-	-	-
	Sh. w/SS Strks.	448.40	448.90	0.50	1.43	21.53	10.77	7.61
	Sh. w/SS Strks.	448.90	450.50	1.60	4.58	38.52	19.26	13.62
	Sh., layered	450.50	452.35	1.85	7.03	47.72	23.86	16.87
	Sh., layered	452.35	452.50	0.15	0.57	13.59	6.79	4.80
	L. Kittanning	-	-	-	-	-	-	-
PD98-02	Sh.	761.00	784.50	23.50	90.51	171.25	85.63	60.55
	Coal	784.50	785.00	0.50	-	-	-	-
	Sh. w/SS Strks.	785.00	790.00	5.00	21.93	84.29	42.14	29.80
	Sh.	790.00	792.30	2.30	7.74	50.07	25.04	17.70
	L. Kittanning	-	-	-	-	-	-	-
PD98-03	Sndy Sh.	756.80	774.10	17.30	51.55	129.23	64.62	45.69
	Shale w/Coal Strks.	774.10	775.10	1.00	3.60	34.15	17.08	12.07
	Shaly Limestone	775.10	777.30	2.20	3.45	33.42	16.71	11.82
	Shaly Claystone	777.30	782.10	4.80	6.32	45.24	22.62	16.00
	Carb. Sh.	782.10	782.50	0.40	1.44	21.60	10.80	7.64
	Sh.	782.50	787.40	4.90	10.75	59.02	29.51	20.87
	Carb. Sh.	787.40	787.50	0.10	0.36	10.80	5.40	3.82
	L. Kittanning	-	-	-	-	-	-	-
PD98-04	Claystone	764.30	781.20	16.90	40.17	114.08	57.04	40.33
	SS w/Sh. Strks.	781.20	786.20	5.00	14.90	69.48	34.74	24.56
	Sndy Sh. w/SS Strks.	786.20	790.20	4.00	24.96	89.93	44.96	31.79
	Carb. Sh.	790.20	791.50	1.30	6.65	46.42	23.21	16.41
	Bone	791.50	791.70	0.20	-	-	-	-
	L. Kittanning	-	-	-	-	-	-	-

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The Development and Use of Risk Assessment Techniques to Assess Roadway Stability in the Parkgate Seam at Thoresby Colliery

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ABSTRACT

A rockbolted roadway risk assessment procedure has been developed specifically for the Parkgate seam at Thoresby Colliery by Rock Mechanics Technology and colliery engineers. The purpose of this assessment is to identify any areas of significant risk in relation to roof instability due to potential failure or overloading of the rockbolt support system. This allows appropriate additional support to be installed in sections of roadway considered to be at increased risk.

The assessment of risk in rockbolted coal mine roadways, in terms of fall of material, has to be based on an understanding of the rock mass behaviour at that site. The development of an appropriate risk assessment system is therefore site specific. A consistent approach to developing such a system can, however, be applied by adopting the same general logic to each individual site.

The use of risk assessment to categorise areas with increased risk of roof instability has been undertaken for four longwall panels at Thoresby. The procedures used for these assessments are discussed and the operational benefits examined.

The approach successfully used at Thoresby has been developed into a routine tool to assist mine personnel in assessing areas of potential roof instability in rockbolted roadways and as a means of communicating the position of these areas to both supervisors and workmen. The system provides a rational and objective interpretation of the current underground conditions based upon a logical, consistent and structured approach.

INTRODUCTION

The widespread adoption of rockbolting as mine roadway support in UK coal mines has been associated with an excellent safety record. The main reason for this is the systematic use of roof movement monitoring to indicate the

level of risk of roof falls and the need for further support. To maintain and improve on existing levels of safety and to meet the requirements of modern Health and Safety legislation procedures for the assessment of risk in mine roadways are also required.

The assessment of risk in mine roadways, in terms of fall of material, has to be based on an clear understanding of the rock mass behaviour at that site. The development of an appropriate risk assessment system is therefore site specific. However, a consistent approach to developing such a system can be applied by adopting the same general logic to each site.

A rockbolted roadway risk assessment procedure has been developed specifically for the Parkgate seam at Thoresby Colliery by Rock Mechanics Technology Ltd. (RMT) in collaboration with colliery engineers. The purpose of this assessment is to identify any areas of significant risk in relation to roof instability due to potential failure or overloading of the rockbolt support system. This allows appropriate additional support to be installed in sections of roadway considered to be at increased risk. This paper describes the technique developed to assess the risk of support failure. The use of risk assessment to categorise areas with increased risk of roof instability has been undertaken for four longwall panels at Thoresby. The procedures used for these assessments are discussed and the operational benefits examined.

ROCKBOLTING AND THE MONITORING OF ROOF BEHAVIOUR IN UK COAL MINES

Traditionally, support of UK coal mine tunnels was undertaken using 'H' section colliery steel arches set at maximum spacings of 1.2 m, as prescribed by mining legislation, with many variations in terms of opening size, support profile and section. The predominant mining system utilised advancing longwalls, which were often labour intensive and relatively high cost. In 1984-85 a decision was made to change to retreat mining in order to improve



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productivity. This, in turn, required that rapid drivage and retreat rates were achieved. The successful introduction and use of rockbolted support became essential for this to be realised. With the introduction of rockbolt support significant difficulties had to be overcome particularly in terms of uncertainty over design and the 'cultural change' in adopting a 'new' support system.

Rockbolting was initially applied as supplementary support to arches. Prior to the advent of systems of properly measuring rockbolt system performance, supplementary rockbolting was often ineffective. In the worst cases, substitution of lighter section colliery arches, together with a poor system of rockbolting resulted in roof falls. This damaged confidence in rockbolting as a support system. Progress was only made once a design by measurement or 'observational' approach was developed and a standardised high strength bolting system adopted. Successful trials were undertaken during 1987-89. Central to the design process, and to ensure safe conditions, roof movement action levels were adopted as indicators of the need for additional support. This is based upon the concept that horizontal stresses maintained in the immediate roof by the installation of effective reinforcement results in roof stability. A reduction in the level of roof stability occurs if these stresses reduce as a consequence of the development of roof shear and dilation. Extensive experience has shown this concept to be correct, although indicative levels of roof movement are site dependant.

Site suitability for rockbolting is determined by site measurements including geological core information, rockbolt bond strength measurements, using short encapsulation pull tests, and assessment of rock stress conditions. The rockbolt support design adopted is subject to confirmation and modification depending upon monitoring results obtained from sonic extensometers and strain gauged rockbolts (1).

Detailed sonic extensometer measurements allow roof movement 'action levels' to be derived for a site. These action levels are the amounts of roof displacement within and above the bolted height that can be safely allowed before action is required to prevent a fall. Typical action levels used in UK coal mines are 25 mm either within the bolted height or in the monitored height above the bolts. These action levels are by no means universal, being affected by stress conditions and local geology. Once action levels have been reached, remedial action is undertaken to secure the roof. Roof movement is routinely monitored using safety monitoring devices, known as dual height tell tales, which provide a visual indication of roof displacement. They are positioned at regular intervals along rockbolted mine roadways, normally every 20 m, and indicate movement within and above the bolted height.

Systematic procedures for inspection, recording and reporting of readings from monitoring form an essential part of mine safety procedures. The approach has been embodied within an Industry Guidance document (2), drawn

up by the UK Health and Safety Executive.

In recent years, in order to further improve on existing levels of safety and to meet the requirements of modern Health and Safety legislation, procedures for the assessment of risk have been actively introduced into a wide range of mine operations (3) including strata control.

THE NEED FOR RISK ASSESSMENT AT THORESBY

Thoresby colliery is owned by RJB Mining (UK) Ltd and situated in the Nottinghamshire Coalfield. The mine currently works the Parkgate seam at a depth of 830 m using profitable single longwall units producing 1.8 million tonnes of saleable coal per annum.

On the 9th May 1995 a fall of ground occurred from the immediate roof on the roofbolted conveyor road of PG125's retreating longwall. No one was injured and within twenty four hours, following roadway repairs, production resumed. However, there had clearly been an unacceptable potential for injury and the colliery were determined that similar situations would not be allowed to recur. The investigation that followed identified a number of factors that led to the roof fall. Had these factors been identified and understood prior to the roof fall then corrective action could have been taken and the incident prevented.

Following the incident in PG125's mine management saw a need for a structured pro-active approach that would allow areas at increased risk of a roof fall, for whatever reason, to be identified. Corrective action could then be investigated and applied and the reasons for this action communicated to all management, supervisors and workmen associated with the roadway. Mining engineers at Thoresby became aware of the risk assessment work being developed at RMT and so collaboration was quickly established to allow for its development and practical application at Thoresby.

Discussions took place to develop the risk assessment model to suit the potential risk factors applicable to the geotechnical environment at Thoresby Colliery. As PG125's longwall was near the end of its life, work concentrated on subsequent longwall units and the first survey was undertaken in September 1996, using the procedures outlined in this paper. The initial survey was conducted jointly by colliery personnel and RMT engineers with a view to the colliery eventually being able to conduct its own surveys with the data being subsequently analysed by RMT.

The adoption of this technique at Thoresby Colliery was particularly timely from two other points of view. Firstly the colliery management team were actively pursuing a culture change in management style by adopting a people orientated safety conscious approach to all activities using both safety audits and risk assessments to spearhead this new approach. The concept of a geotechnical risk assessment once explained was thus readily accepted by the supervisors and workforce. Secondly the colliery was starting to extract coal

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reserves from an area of the mine affected by geological variations, which included sandstone channel features and faulting, in addition to interaction effects from overlying old workings. The risk assessment survey focused attention onto the risks associated with these factors.

DEVELOPMENT OF THE RISK ASSESSMENT SYSTEM

The assessment of risk at any site, in terms of fall of material, has to be based on an understanding of the rock mass behaviour at that site. The development of an appropriate risk assessment system is therefore site specific.

A general logic for support risk assessment has been developed by RMT as part of a range of risk assessment methods related to general and specific failure mechanisms. In each case a consistent approach to developing such a system has been applied by adopting the same general logic to each site. Figure 1 shows the general model on which the risk assessments are based. This model illustrates the steps required in the development and application of a support risk assessment.

In the context of this paper, risk factors are detectable or observable features which, if present in or around the roadway, indicate the potential for, or contribute towards, the risk of a fall of material. The most obvious risk factor is roof movement. As movement increases so does the risk of roof instability. Some risk factors may be readily observable features such as faults, strata water, excessive movement/roof shear or roofbolt collar loading. Others factors may only be detected by carrying out investigations, such as the variation in roof/floor lithology and material strength which could be determined from cores. Factors such as interaction or the presence of major geological features (e.g. sandstone roof channels etc.) may need to be determined by reference to records or plans.

FACTORS CONTRIBUTING TO THE POTENTIAL FAILURE OR OVERLOAD OF ROOFBOLTS IN THE PARKGATE SEAM

Any fall of ground is considered unacceptable in terms of both safety to mine personnel and economically due to the potential disruption in mining operations. The Parkgate seam has been worked at many mines in the United Kingdom. Although the number of roof falls which have occurred in roadways supported with rockbolts in the UK has been low, small scale localised falls from within the roofbolted height have occurred in the Parkgate seam.

Examination of relevant case histories indicates that the falls of ground generally occurred due to the failure of the rockbolts over part of the roadway. In each case failure of the rockbolts was considered to be due to a number of similar contributing factors. These could be summarised as follows :-

a. Roof shear associated with :-

- i) potential shear horizons within the immediate roof associated with changes in roof lithology.
- ii) changes in roof profile due to overbreak. An uneven immediate roof profile offers minimal lateral confinement to the lower beds below a potential shear horizon. Shear restraint is therefore, provided almost totally by the rockbolt crossing the shear surface.
- iii) stress increase due to front abutment effects inducing additional shear displacements.

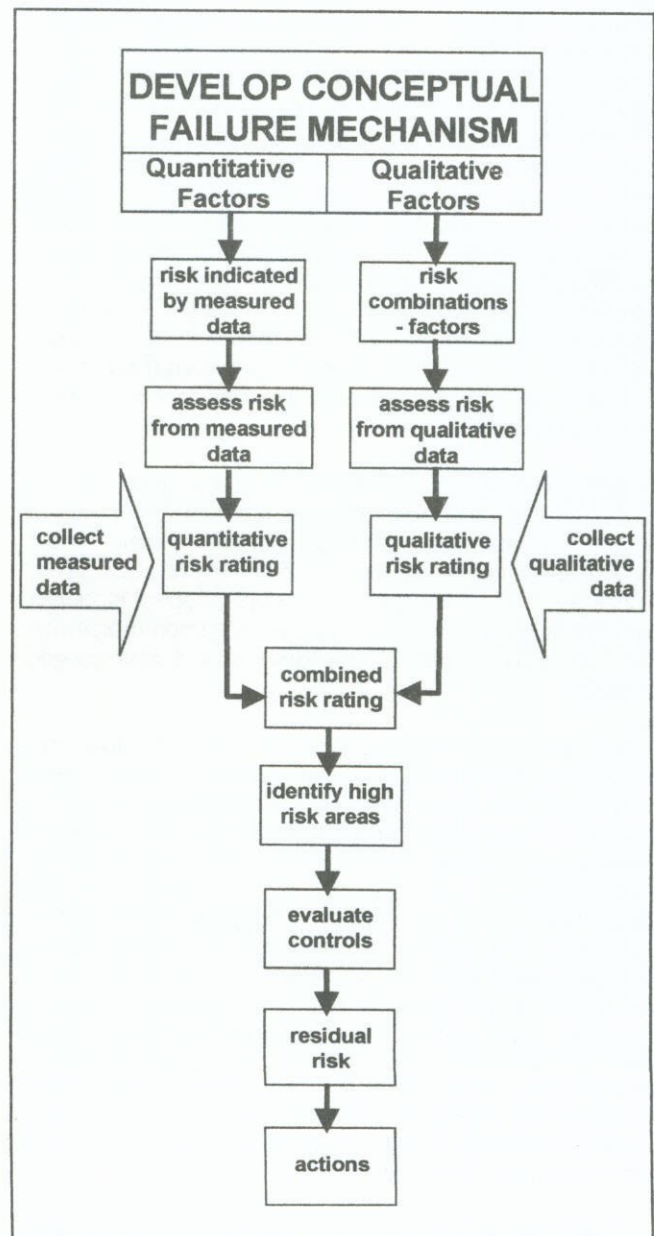


Figure 1. General risk assessment model.

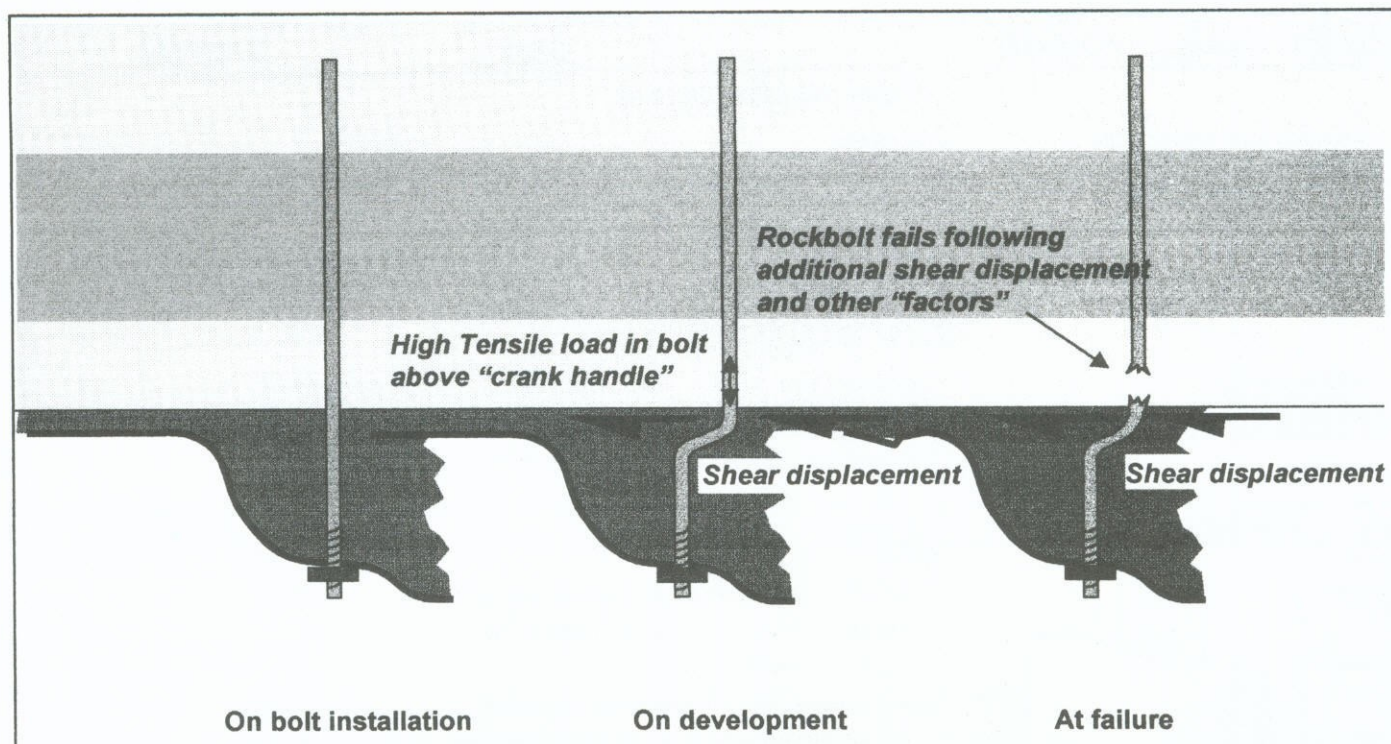


Figure 2. Mode of rockbolt loading due to high shear displacement in immediate roof

- b. Corrosion of the bolts due to exposure to highly corrosive strata water.
- c. Inappropriate remedial reinforcement in response to visual deformation of the lower roof.

A sketch illustrating the potential for increased loading of the roofbolts in such situations is shown in Figure 2. The implications from these conclusions are that there is a potential risk of other falls occurring where similar conditions exist, and, therefore, any areas at risk need to be identified and if necessary, appropriate remedial action taken.

All rockbolted roadways in UK coal mines are routinely monitored using dual height tell tales. In certain situations warning of small scale falls may not be given by routine tell-tale monitoring when used in isolation and therefore a system has been devised to complement the tell tale monitoring system with observations made during underground surveys of the roadways.

SITE INVESTIGATIONS

The assessment of risk is based on observations and measurements obtained from underground surveys and information regarding the mine layout and known geological and geotechnical features.

A simple booking sheet has been developed for the

purpose of recording observations made underground. The survey sheet makes reference to a number of key roadway parameters which in combination are considered to be contributing factors in the potential for a localised failure of a rockbolt reinforcement system.

The booking sheet, shown in Figure 3, has been divided into a number of sections. The first is used to give the roadway position at the mid survey point which is usually based on a dual height tell-tale location. This ensures that the survey is undertaken at regular intervals of not greater than 20 m and can be cross referenced with both the tell-tale monitoring data and a location plan. The second column has a roadway cross section template which allows a quick summary sketch of the salient points (e.g. overbreak, shear zones and roof / rib displacements). The third column is for comments made with regard the condition of the road at the survey point. The final column is divided into nine rows which allow the observer to quickly record the presence of key parameters using a series of codes. (It should be noted that all nine parameters were not used in the analysis of risk at Thoresby but were recorded during the surveys for completeness. This would allow their presence to be evaluated for future inclusion to the procedure if considered necessary).

A brief description of each parameter is given below :-

1. Immediate roof deformation (as recorded by the "A device" of the dual height tell-tale). The amount of

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COLLIERY: Thoresby ROADWAY :		DATE :		SHEET No :	
POTENTIAL RISK FACTORS					
1. IMMEDIATE ROOF DEFORMATION (based on A - tell tale movement) L = low, < 50% of action level M = med, 50 - 100% of action level H = high, > action level		2. ROOF WATER N = none, dry P = water present in the past Y = yes, water present		3. ROOF PROFILE F = flat U = uneven	
4. STRESS CHANGE F = front abutment D = dinting I = interaction with adjacent longwall		5. REMEDIAL ACTION N = None SB = Spotbolted (eg. SB5 = 5 SB/m run) FB = Flexible bolted (eg. FB5 = 5 FB/m run) CB = Cablebolted (eg. CB32 = 3:2 pattern) (denote D = on development, A = after development)		6. SHEAR POTENTIAL (related to roof lithology) H = high (eg. sandstone channels) L = low (eg. normal lithology)	
7. DRIVAGE DIRECTION G = good i.e. with stress B = bad i.e. across stress I = intermediate (30 - 60° to max. horizontal stress)		8. VISIBLE ROOF SHEAR N = none or minor (slight buckling of immediate roof strata) M = moderate (increased N, overbreak, minor roof/strap shortening, few flat bolt plates) S = significant (increased M, guttering, tension breaks, many flat bolt plates)		9. VERTICAL STRESS L = low (under goaf) M = moderate (depth of cover) H = high (interaction, small pillars)	

ZONE	ROADWAY SKETCH (looking inbye)	COMMENTS	POTENTIAL RISK FACTORS
TT No.25		EXAMPLE	1 H
POSITION = 510 m		- ROOF WET, DRIPPERS BIASED TO LHS, ROOFBOLT PLATES CORRODED.	2 Y
A= 30 mm		- OVERBREAK INTO ROOF AND FLOOR HEAVE BIASED TO RHS.	3 U
B= 4 mm		- FLATTENED BOLT PLATES ON LHS.	4 F
C= n/a		- SANDSTONE PARTING EVIDENT 1.6 M ABOVE SEAM ON RHS.	5 SB4 (A)
		- 4 SPOTBOLTS/M (SB4) INSTALLED THROUGHOUT ZONE 2 MONTHS AFTER DEVELOPMENT	6 H
			7 B
			8 M
			9 L

Figure 3. Example of underground survey booking sheet.

roof dilation present within the bolted horizon is related to pre-determined colliery action levels. Increased roofbolt load is expected as the level of immediate roof deformation increases.

2. The presence of roof strata water. The sandstone measures close to the Parkgate seam are often associated with corrosive mine water. Exposure of the roofbolt steel to the strata water, through fractures formed in the resin surrounding the bolt as a result of roof movement, may lead to significant corrosion of the roofbolt steel. Strata water may also lead to degradation of the strata in the immediate roof.
3. Roof profile. An uneven immediate roof profile offers minimal lateral confinement to the lower beds below a potential shear horizon. Increased shear forces and loading could therefore be potentially generated in this situation, as illustrated in Figure 2.
4. Stress change. Increased roof shear which results in increased bolt loading is often associated with a significant stress change such as the front abutment generated during extraction.
5. Remedial or additional reinforcement. Appropriately timed additional reinforcement reduces the level of risk of potential roof failure. The timing of any

remedial action is an important factor. The condition of additional reinforcement placed on drivage could be significantly different to that which has been subsequently placed after roof deformation and/or corrosion has occurred.

6. Shear plane potential (related to roof lithology). The characteristic roof lithology of the Parkgate seam comprises immediate mudstones overlain by progressively stronger siltstones. However, sandstone channel features are sometimes present which result in the normal roof lithology being replaced by laminated sandstone units within bolted height. The presence of distinct bedding or a parting plane, such as the base of a competent sandstone bed, could result in significant bending of the roofbolts in the vicinity of the shear surface should the lower roof move laterally below the upper beds. In situations where the roof is uneven or significant roof shear is evident high loads are likely to be generated in the bolts and the potential for broken roofbolts exists.
7. Drivage direction with respect to the maximum horizontal stress direction. The orientation of a roadway with the maximum horizontal stress can have a significant effect on the immediate roof behaviour especially when the normal Parkgate roof geology is disturbed or affected by sandstone channel

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features.

8. Observed roof shear. This observation is critical since excessive shear can result in significant localised immediate roof deformation which is potentially not detectable with the routine tell tale monitoring.
9. Vertical stress regime. The Parkgate has been extensively overworked in the Top Hard seam leaving a number of small remnant pillars which can, in conjunction with other parameters cause floor heave and additional rib deformation. In certain conditions, this can result in additional loading and shear in the roof.

The findings of the underground survey are analysed using the process described below.

DEVELOPMENT OF RISK ASSESSMENT TECHNIQUES TO ASSESS IMMEDIATE ROOF STABILITY

The "Roadway Risk Assessment" approach is divided into a number of stages :-

- | | |
|---------|---|
| Stage 1 | <p>Assessment of routine monitoring.</p> <p>This was considered a suitable starting point since tell tale results are already recorded on a systematic basis in all UK coal mines.</p> |
| Stage 2 | <p>Potential risk rating.</p> <p>This assessment identifies key variables present at the site which in combination, and in areas of moderate roof deformation and shear, increase the risk of rockbolt failure. The results are collated to give a Potential Risk Rating (PRR).</p> |
| Stage 3 | <p>Roadway zone risk rating.</p> <p>This quantifies both measured roof deformation and observed shear using a simple scoring system of "low" "medium" and "high" which results in a Roadway Risk Rating (RRR).</p> |
| Stage 4 | <p>Assessment of appropriate reinforcement.</p> <p>This stage assesses the level of risk, the appropriate level of support required to maintain roadway condition and the suitability of any remedial reinforcement or support already installed.</p> |

Figure 4 shows a flow diagram of this process, the results of which lead to two alternative outcomes :-

- a) No additional remedial action is required, therefore,

continue with routine monitoring, or

- b) Appropriate remedial reinforcement system is required to ensure long term stability.

In the latter, the appropriate action required is dependant on the measured and perceived risks and whether any

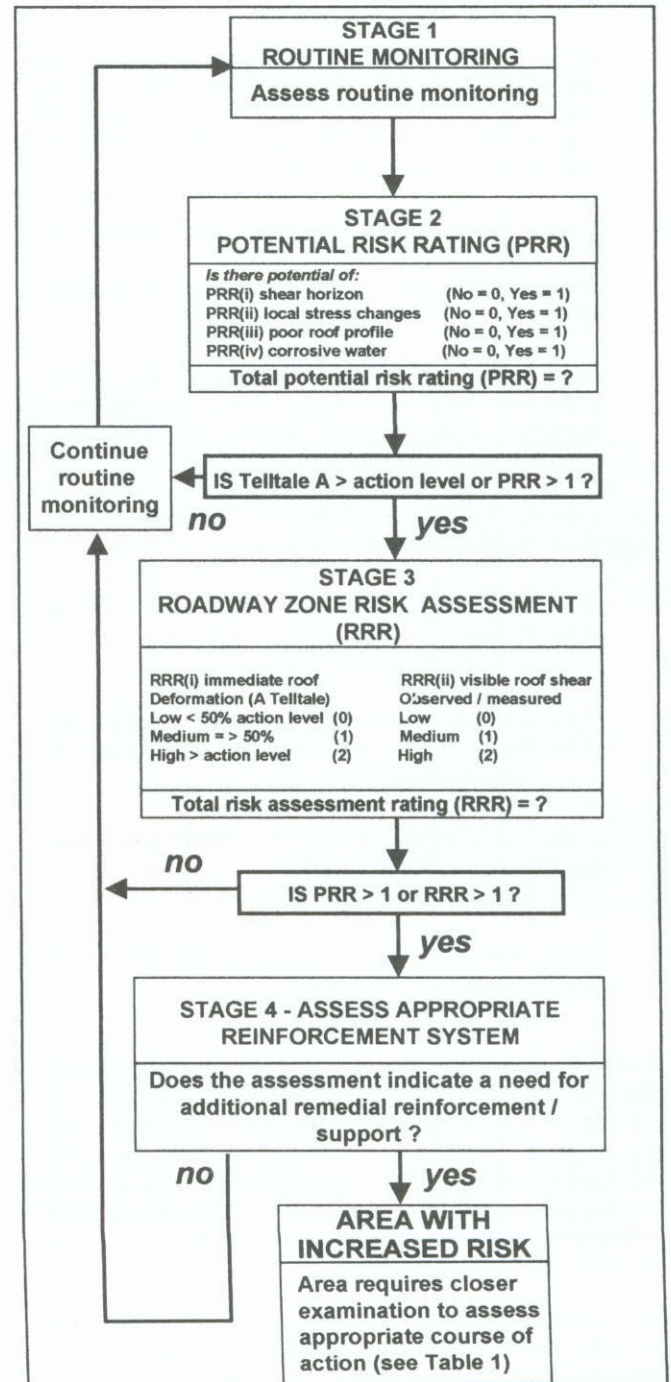


Figure 4. Risk assessment flow chart

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remedial action has already been taken. If additional reinforcement or support has been placed this must in turn be assessed for likely effectiveness particularly in terms of the timing and type of additional reinforcement.

Appropriate remedial actions have been specified for four situations which may be identified by the risk assessment monitoring to ensure continued stability. These are, in descending order of severity :-

- i) Areas which have had additional reinforcement placed and have not stabilised. These require closer examination, by qualified personnel, to assess the appropriate course of action and are potentially high risk areas if no response is made.
- ii) Areas suspected of, or potentially having, failed bolts.
- iii) Areas suspected of, or potentially having, overloaded bolts.
- iv) Those areas that, at present, do not require additional reinforcement but should be flagged as potential areas at risk should additional roof deformation or shear occur.

In the case of the Parkgate seam at Thoresby situations (ii) and (iii) have been sub divided as follows :-

- a) Moderate roof movement and shear with no corrosive water.
 - risk of overloaded bolts
- b) High roof movement and shear or moderate roof movement and shear with corrosive water present.
 - risk of failed bolts
- c) Areas having greater than two potential risk factors not including corrosive water.
 - potential risk of overloaded bolts
- d) Areas having greater than two potential risk factors which include corrosive water.
 - potential risk of failed bolts

If the criteria of (a) and (b) are met then these areas are considered to have a risk of overloaded or failed bolts respectively and require additional reinforcement. Alternatively, if conditions (c) and (d) are met these are considered to be areas with a potential risk of overloaded and failed bolts respectively and further investigations must be undertaken to check for this and to assess the appropriate remedial reinforcement system.

As a result of the survey the risk assessment zones are individually rated and recommended actions proposed. These are summarised in Table I.

In addition to the above risk assessment rating an assessment is also made of any additional reinforcement that

has been installed in each zone. This assessment compares the existing additional reinforcement with the minimum level of reinforcement which is considered appropriate and also takes into account whether it was installed before or after roof deformation had occurred. If the additional reinforcement was installed prior to significant deformation (e.g. during roadway development) then it has been subjected to a similar level of deformation and corrosion as the standard support system and therefore cannot be classed as remedial. This assessment is necessary to confirm that the installed support is appropriate for the prevailing conditions and that the roadway is behaving in a stable manner. In the case of continued instability, these areas are identified as HIGH RISK areas which urgently require closer examination by trained personnel to assess the appropriate course of action.

Table I Table of risk classification and recommended minimum actions

RISK RATING	LEVEL OF RISK	MINIMUM ACTIONS
Risk of Bolt Failure - Areas where there is a risk of bolt failure.	HIGH	Rockbolting engineer to inspect site and assess required remedial support. High density spotbolting required in affected areas, minimum 3 bolts/m run. Re-assess conditions on a regular basis to confirm stability.
Risk of Bolt Overloading - Areas where there is a risk of overloaded bolts.	HIGH	Spotbolt affected areas, minimum 3 bolts/m run. Re-assess conditions on a regular basis to confirm stability.
Potential Risk of Bolt Failure or Bolt Overloading - Areas which require additional investigative work to assess if the bolts are either overloaded or have failed.	MEDIUM /HIGH	Undertake investigative work or assume HIGH risk and install additional reinforcement. Investigation work would consist of in-situ testing or overcoring of selected bolts to assess their condition and level of loading.
Flagged Area - Areas which are potentially at risk if additional roof deformation and/or shear occurs.	MEDIUM	Continue with normal routine monitoring. Regularly re-assess to identify any change in condition.
Continue Routine Monitoring - Areas which presently have a low risk rating with regard to both bolt overloading and failure.	LOW	Continue with normal routine monitoring. Regularly re-assess to identify any change in condition.

Based on the results obtained from underground surveys, and an overview of other factors which may influence stability, the above risk assessment procedure was used to categorise the potential for rockbolt failure due to roof shear for a number of longwall gateroads at Thoresby Colliery.



APPLICATION OF THE RISK ASSESSMENT TECHNIQUE AT THORESBY COLLIERY

Following the first full risk assessment survey and the analysis of the results a follow up visit was made to the district to assess the outcome of the risk analysis from the perspective of the colliery management team. The results, in part, accurately reflected the teams thoughts with regard to the security of the roadway and placed a structured emphasis on zones of risk from a multiplicity of risk factors that in reflection had not been genuinely appreciated to be of significance.

Following the secondary site visit a number of actions were initiated resulting in the installation of additional reinforcement in the form of additional bolts, long tendon anchors or even passive supports such as cribs and wooden props. Other areas which were "flagged" as moderate risk and worthy of particular attention were closely monitored for any deterioration and possible inclusion in future reinforcement programmes.

The final stage in the process, having recognised the value of the risk assessment technique, was to communicate the outcome effectively to the supervisors and workmen associated with roadways. To do this a layout plan of the district was produced including all relevant geological and geotechnical information in an easily understandable format. The risk assessment zones were appropriately colour coded and referred to by a key. Finally the positions of all monitoring devices were added together with the positions of all remedial actions taken in the risk areas. Figure 5 shows a much simplified example of such a plan. Risk assessment plans were produced at the size an advertising poster, approximately 1.0 m wide and 0.5 m high, and encapsulated in protective plastic before being displayed in the colliery offices and underground in the roadways themselves.

Feedback on the effectiveness of the posters was sought from the supervisors and workforce and improvements in presentation resulted from their suggestions and advice. In this way ownership of the roadways and their associated problems was quickly accepted by the supervisors who were able to focus attention to high, low or medium risk areas as appropriate. This in turn led to the supervisors themselves, taking the correct pro-active remedial measures if required long before a roadway stability problem developed into more serious proportions.

Another additional benefit occurred when the mine planning engineer correlated risk plans for adjacent longwall panels. The patterns which emerged were extrapolated to neighbouring longwalls allowing operational risk to longwall production, related to zones with a high risk rating, to be incorporated into the colliery business plan. Appropriate pro-active reinforcement of risk areas resulted in increased confidence in achieving planned longwall retreat rates and allowed the planning schedule for replacement longwalls to be reviewed and planned more precisely.

In terms of cost, lost production of one shift due to roadway instability during the life of a face would cost the colliery significantly more than the cost incurred of conducting the risk assessment. In addition, Thoresby's moral and legal obligation to take all reasonable measures to provide a safe working environment have been further enhanced and the potential cost of injuries to workmen has been greatly reduced.

Since the roof fall on PG125's district all subsequent longwall gate roadways have been risk assessed using the technique described. Remedial actions have been undertaken wherever the need has been identified both prior to and during production. Longwalls have retreated through zones potentially more hazardous than PG125's without any type of roof fall or roadway closure problems. There have been no recorded injuries at all from falls of ground, as there have been no falls of ground. Production has been continuous, with the budgeted output consistently achieved week in, week out adding to the markets confidence in Thoresby Colliery as a reliable supplier.

CONCLUSIONS

The approach successfully used at Thoresby has been developed into a routine tool to assist mine personnel in assessing areas of potential roof instability in rockbolted roadways and as a means of communicating the position of these areas to both supervisors and workmen.

The system provides a rational and objective interpretation of the current underground conditions based upon a logical, consistent and structured approach. In addition, it is considered that the use of regular re-assessments could be used to identify significant changes in roadway condition and allow a pro-active approach to ensuring roof stability.

The colliery now undertakes roadway stability risk assessments routinely for all production units. This has allowed Thoresby to:-

- a. invest in some of the best longwall equipment available with confidence,
- b. operate longwall faces which realise their full potential,
- c. understand and categorise roadway conditions better and communicate problem areas effectively to the workforce,
- d. safely mine reserves profitably in a geotechnically challenging environment.

The strategy has been easy to implement in the form described and has realised a number of unexpected benefits.

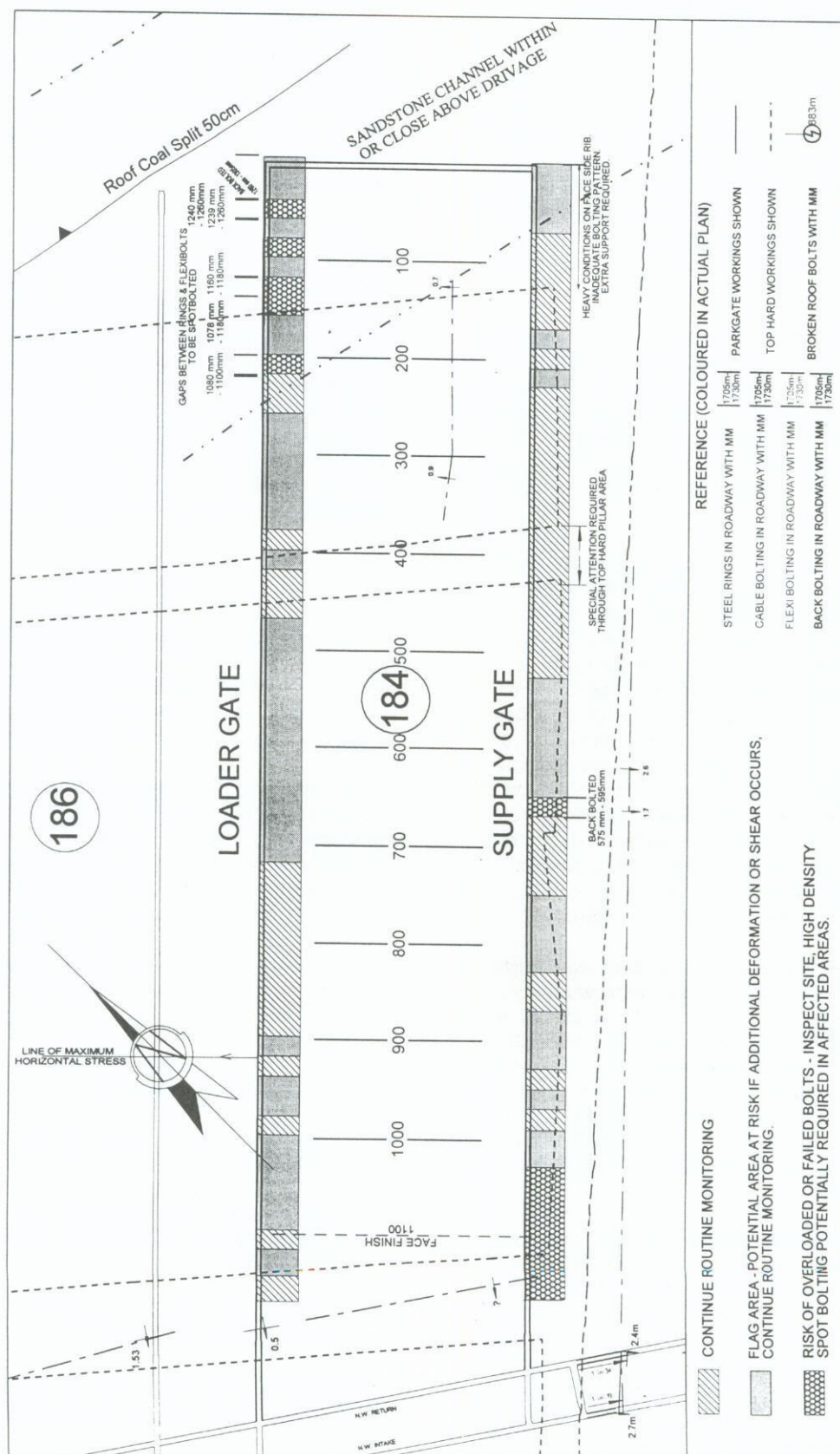


Figure 5. Simplified risk assessment plan for PG184's, Parkgate seam, February 1999.



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ACKNOWLEDGEMENTS

The views expressed within this paper are those of the authors and not necessarily those of RJB Mining (UK) Ltd.

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Time-dependent Analysis of Underground Opening Stability

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ABSTRACT

Excessive deformation occurred in the underground openings of Jinchuan Nickel Mine, the largest nonferrous underground mine in China. Developed in weak rock formations at great depth, the underground openings are under great in-situ stresses with the horizontal stress being twice as high as the vertical one. Based on in-mine observations and in-situ testing results at the mine, time-dependent finite element simulations with coupled creep and plasticity constitutive models were conducted in this study to analyze the stability of underground openings. In the study, the Burgers creep model and the Drucker-Prager plastic model were coupled to simulate underground drifts and cut-and-fill stopes. Two-dimensional finite element models were developed with ABAQUS - a comprehensive finite element simulation package. The study investigated the behavior of underground drifts at great depth with high horizontal stress and the behavior of cut-and-fill stopes with various stope heights and fill conditions.

The numerical simulations revealed important characteristics of opening deformations as a function of time after excavation and location relative to the wall of opening. The results had good agreement with field monitoring data. They provided useful information to aid mine engineers in the design of mine openings, the selection of opening supports and the determination of support installation procedures.

INTRODUCTION

In deep underground mines with mine openings subject to great stresses, the time-dependent deformation of rocks frequently plays a predominant role in determining the stability of the openings. This is particularly true in soft rock formations. In order to have a complete and accurate analysis of the stability of underground openings, the time-dependent deformation behaviors of rock materials must be taken into account.

In this research, studies were conducted at Jinchuan Nickel Mine, the largest underground nonferrous mine in China. The

mineral deposits of the mine are located at a depth ranging from 500 m to 1000 m. High in-situ stresses exist in the mine area and the horizontal stress is almost twice as high as the vertical one. At the depth of 500 m, the measured horizontal in-situ stress is 24.5 MPa, and the vertical stress is 13.7 MPa. The high horizontal in-situ stress frequently induces significant creep deformation, which may lead to complete failure of the drift. In order to maintain the stability of the drift, the opening support methods and timing must take into account of the time-dependent deformational behaviors of the rock mass.

In addition, the cut-and-fill mining method is used at Jinchuan Mine. No pillar is left in the mined area. Therefore very large mined and filled stopes are created over time. The dimension of the stope is currently at about 100 m wide, 50 m to 100 m high, and 2000 m long. In the future, the height and length of stopes will continue to increase. The stability of such a large stope is of deep concern, although it is filled.

In this study, time-dependent finite element simulations with coupled creep and plasticity constitutive models were conducted based on in-mine observations and in-situ stress testing results at the mine. The Burgers creep model and the Drucker-Prager plastic model were coupled to simulate underground drifts and cut-and-fill stopes. Two-dimensional finite element models were developed with ABAQUS - a comprehensive finite element simulation package. The study investigated the behavior of underground drifts at great depth with high horizontal stress and the behavior of cut-and-fill stopes with various stope heights and fill conditions.

FINITE ELEMENT MODELS

In this study, the comprehensive finite element package, ABAQUS running on CRAY supercomputer, was employed. The program includes well-developed non-linear material models, very flexible user interface and a strong post processing program with graphic and image capabilities.



Material Models

Burgers model is applied in this study to simulate the time-dependent deformation of rock materials. The physical representation of Burgers model is composed of four elements: a pair of spring and dashpot in parallel, and a pair of spring and dashpot in series, as shown in Fig 1. The stress-strain-time relation is given by Equation (1):

$$\eta_1 \epsilon'' + k_1 \epsilon' = \frac{\eta_1}{k_2} \sigma'' + \left[1 + \frac{k_1}{k_2} + \frac{\eta_1}{\eta_2} \right] \sigma' + \frac{k_1}{\eta_2} \sigma \quad (1)$$

where: ϵ =strain

σ =stress

k_1, k_2 =coefficients of elasticity

η_1, η_2 =coefficients of viscosity

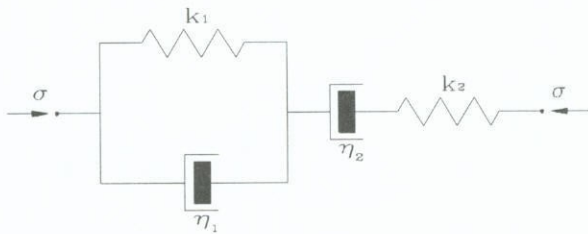


Figure 1. Burgers Model

The yielding and the subsequent plastic deformation of rock material is governed by the Drucker-Prager model, which has been extensively used to simulate rock material behaviors. The Burgers model is coupled with the Drucker-Prager model in the finite element simulations using ABAQUS to study the combined effects of creeping and plastic deformation.

Geometric Models of Mine Openings

To analyze the problems encountered at Jinchuan Mine, four different simulation models are developed based on the site conditions at the mine as described below.

Model of Drift Most of the long-term mining service drifts at the mine are located in rock surrounding the orebody. Service life of drifts ranges from 2 to 5 years. Some may reach 10 years. Drifts in the orebody are kept for a very short period of time, usually 1-3 months. The stability problems mainly occur in the long-term mining service drifts located in the rock. The analysis in this study, therefore, focuses on the drift in the rock. Since the length of the drift is much longer than the width and height of the drift, a 2-dimension simulation model is developed. The simulation model is shown in Figure 2, in which the drift is represented by a 8 m x 5 m (width x height) opening.

Models of Cut-and-fill Stope The mining area of the mine is about 100 m wide and 5000 m long along the strike of the orebody, and 50 to 150 m high. Since the dimension along the strike direction is much larger than the along width and height, a cross section perpendicular to the strike direction is taken to develop 2-dimension models. The model is shown in Figure 4. Since the stope height varies from 50 m to 150 m, three models of stope are established with a height of 50 m, 100 m, and 150 m, respectively.

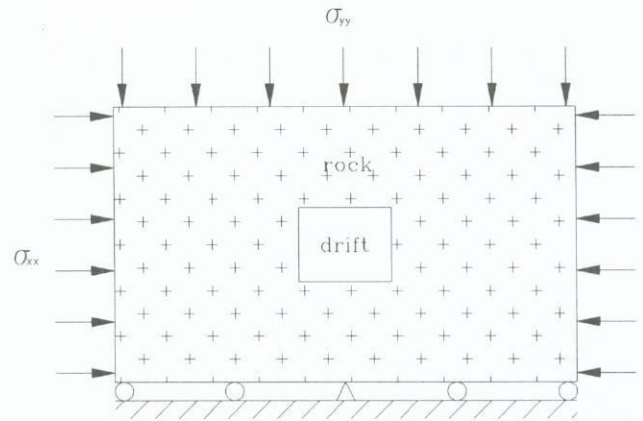


Figure 2. Simulation Model of Drift

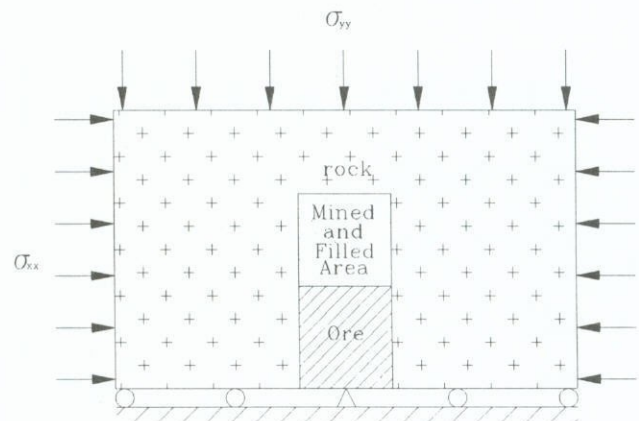


Figure 3. Simulation Model of Cut-and-fill Mining Stope

Models of Fill Conditions Although the design requirement is that the mined stope should be fully filled, it is not always fulfilled in practice. Several reasons lead to this undesirable situation:

- ▶ Limitation of filling techniques;
- ▶ Water drainage from the fill materials; and
- ▶ Shrinkage and compaction of the fill materials

In addition to the fully filled models mentioned above, another three models of various filled conditions are established. They are models of stopes with fill materials contacting 30%, 50%, and 70% of the roof area respectively, as shown in Figure 4.

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Models of Stopes With Adjacent Service Drifts At Jinchuan Mine, the mining service drifts are located adjacent to the stope. The distance between the drift and the wall of the stope varies from 10 m to 50 m. To simulate this situation, three models are established, with the service drift 12 m, 28 m and 52 m from the wall of the stope respectively, as shown in Figure 5. The dimension of the drift is 6 m x 5 m (width x height).

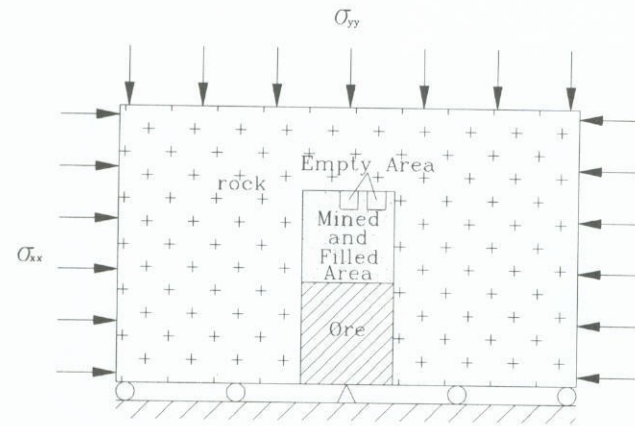


Figure 4. Simulation Model of Cut-and-fill Mining Stope with Different Fill Conditions

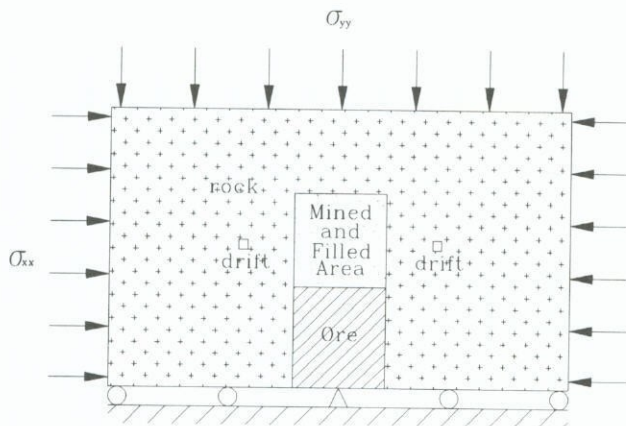


Figure 5. Simulation Model of Stope with Service Drifts

FINITE ELEMENT SIMULATIONS AND RESULTS

Model Calibration and Verification

In order to fine-tune the finite element models for more in-depth study, in-mine monitoring of roof displacement in the drifts and the stopes were utilized to calibrate the model and to verify the accuracy of the results. The comparisons of roof displacement between the field data and the simulation results are shown in Figures 6 and 7. The graphs illustrate good agreements between the two sets of data.

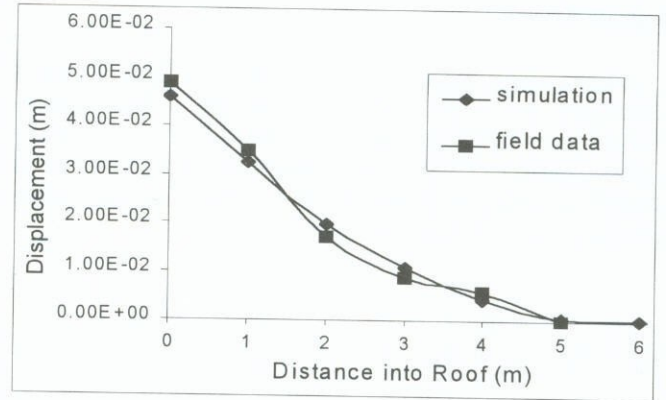


Figure 6. Comparison of Displacement in the Roof of Drift (at t = 60 days)

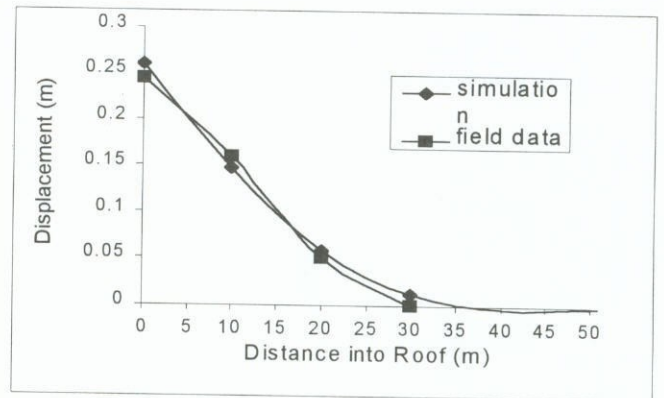


Figure 7. Comparison of Displacement in the Roof of Stope (at t = 60 days)

Stability Analysis of Mine Drift

The drift of the mine is of rectangular shape with a length by height dimension of 8x5 m². The drift is located in a depth of 500 m in underground. The rock material properties are shown in Table 1.

Table 1 Properties of Rock Material

Young's modulus of the rock (E)	3.0 x 10 ⁹ Pa
Poisson's ratio of the rock (μ)	0.25
Density of the rock (ρ)	2700 kg/m ³
Yield stress of the rock (σ_c)	1.34 x 10 ⁸ Pa
Drucker-Prager frictional angle (β)	55°
Drucker-Prager material constant (k)	1.0
Burgers model material constant (k_1)	3.0 x 10 ⁹ Pa
Burgers model material constant (k_2)	3.0 x 10 ⁹ Pa
Burgers model material constant (η_1)	3.4 x 10 ¹⁵ (Pa-s)
Burgers model material constant (η_2)	2.6 x 10 ¹⁵ (Pa-s)

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The simulation results are shown in Figures 8 to 11. The vertical and horizontal stresses as a function of time and distance into the wall of the drift are shown in Figures 8 and 9. The displacements as a function of time and distance into the roof of drift are shown in Figures 10 and 11.

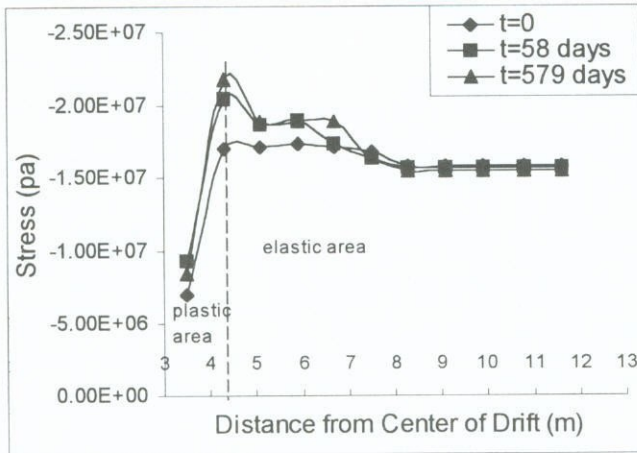


Figure 8. Vertical Stress in the Wall of Drift

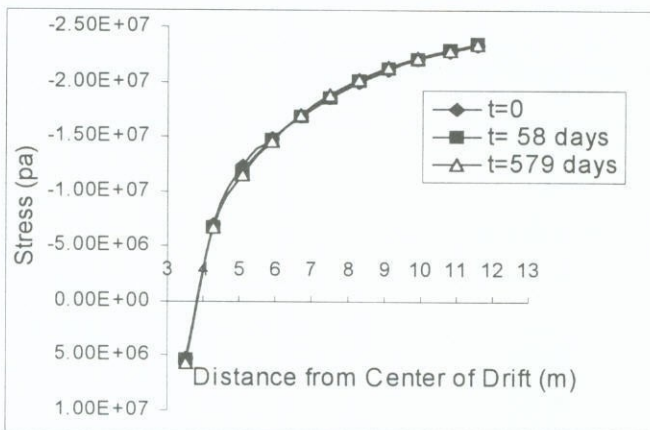


Figure 9. Horizontal Stress in the Wall of Drift

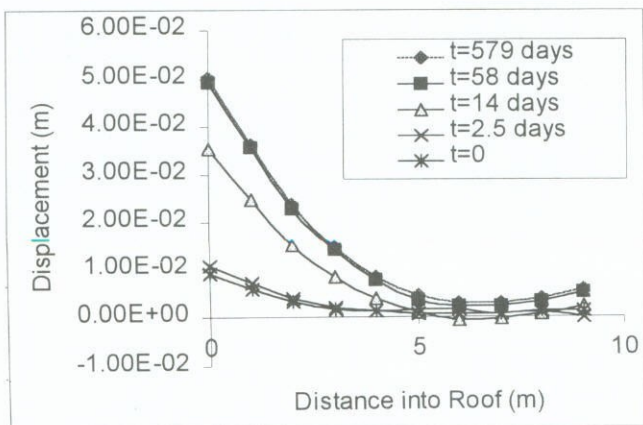


Figure 10. Displacement in the Roof of Drift

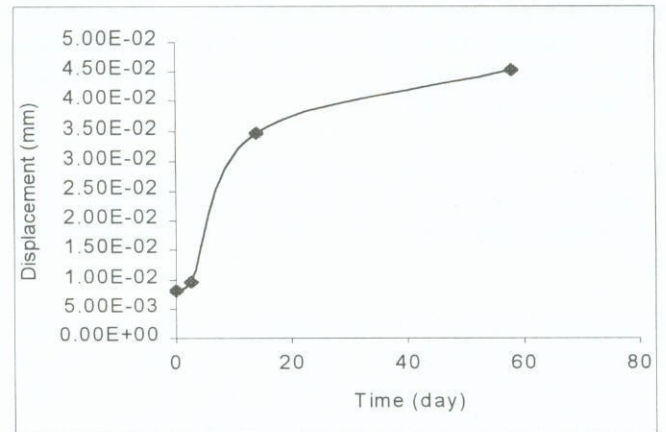


Figure 11. Displacement as a Function of Time in the Roof of Drift

From the results of finite element modeling, the following conclusions can be drawn:

- In the wall of the drift, the plastic region is about 1.5 m. The peak vertical stress is 1.40-1.52 times of the in-situ vertical stress over the time period of 579 days. The location of peak stress is at 1 m into the wall of drift.
- The horizontal tensile stress occurs at the middle height of the drift wall. This region extends about 0.5 m deep. The tensile stress is the main reason to cause damage of the drift.
- The simulated displacement in the roof of the drift agrees well with the results of in-situ monitoring (Sun and Chen, 1998). The maximum displacement appears on the surface of the wall and roof. It decreases sharply as distance into the wall or roof increases. At about 6 m into the wall and roof, the displacement reaches its minimum and keeps approximately constant thereafter.
- The displacement has a significant increase in 15 to 20 days and more than 90% of the displacement takes place within 60 days.

Stability Analysis of Cut-and-fill Stopes

Since mining depth varies from place to place, three models of stopes are simulated. Stope 1 has a height of 50 m, Stope 2 100 m, and Stope 3 150 m. The three stopes were considered to have the same width of 100 m. The properties of rock and fill material for the three stopes are considered to be the same as well. The rock is a mixed igneous rock formation. The fill material is fine sand mixed with cement. The properties of rock and fill material are shown in Table 2.

Fully-filled Stopes Typical simulation results are shown in Figures 12 and 13, which show the vertical stress and displacement in Stope 1. Simulation Results and comparisons among Stopes 1, 2, and 3 are shown in Table 3. From these results, the following conclusions are drawn:



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- No tensile stress exists in the wall of Stope due to the pressure of fill material. Both vertical stress and the horizontal stress decrease slightly over time in the rock, and increase slightly over time in filled region.
- With the increase of the stope height, the size of the plastic region increases, from 30 m for a stope height=50 m to 45 m for a stope height=100 m and to 55 m for a stope height=150 m.
- With the increase of the stope height, peak stress in the wall decreases, and distribution region of the peak stress is wider, from 0 for a stope height=50 m to 30 m for a stope height=100 m and to 40 m for a stope height=150 m.
- With the increase of the stope height, the stress in the filled region decreases slightly.
- With the increase of the stope height, the displacement of the floor in the stope increases slightly.

Effects of Various Fill Conditions Four different fill conditions in the stope are considered. They are:

- 1) Model 1, fill material contacting 100% of the roof area;
- 2) Model 2, fill material contacting 70% of the roof area;
- 3) Model 3, fill material contacting 50% of the roof area; and
- 4) Model 4, fill material contacting 30% of the roof area.

Table 2. Properties of Rock and Fill Material for Stopes

Young's modulus of the rock (E)	3.0×10^9 Pa
Poisson's ratio of rock (μ)	0.25
Density of rock (ρ)	2700 kg/m ³
Yield stress of rock (σ_c)	1.34×10^8 Pa
Young's modulus of fill material (E)	5.0×10^7 Pa
Poisson's ratio of fill material (μ)	0.25
Density of fill material (ρ)	2100 kg/m ³
Uniaxial strength of fill material (σ_c)	3.0×10^6 Pa
Drucker-Prager frictional angle (β)	55°
Drucker-Prager material constant (k)	1.0
Burgers model material constant (k_1)	3.0×10^9 Pa
Burgers model material constant (k_2)	3.0×10^9 Pa
Burgers model material constant (η_1)	3.4×10^{15} pa-sec
Burgers model material constant (η_2)	2.6×10^{15} pa-sec

Table 3. Comparison of Simulation Results among Stopes 1, 2, and 3

Stope	1	2	3
Stope height	50 m	100 m	150 m
Plastic Region	30 m	45 m	55 m
Factor of peak stress over in-situ stress	1.75 (t=0) 1.7 (t=579 days)	1.45 (t=0) 1.4 (t=579 days)	1.4 (t=0) 1.37 (t=579 days)
Peak stress in the wall of stope	2.27×10^7 Pa (t=0) 2.2×10^7 Pa (t=579 days)	1.93×10^7 Pa (t=0) 1.9×10^7 Pa (t=579 days)	1.82×10^7 Pa (t=0) 1.78×10^7 Pa (t=579 days)
Width of peak stress region	0	30 m	40 m
Stress in the filled region	1.21×10^6 Pa (t=0) 1.53×10^6 Pa (t=579 days)	8.23×10^5 Pa (t=0) 1.07×10^6 Pa (t=579 days)	7.57×10^5 Pa (t=0) 9.49×10^5 Pa (t=579 days)
Tensile stress occurrence	No	No	No
Percentage of the creep deformation in the total deformation in stope wall	10%	46%	90%
Heaving on the floor of stope	0.1 m (t=0) 0.26 m (t=379 days)	0.14 m (t=0) 0.38 m (t=379 days)	0.15 m (t=0) 0.46 m (t=379 days)
Displacement on the roof of stope	0.77 m (t=0) 0.9 m (t=379 days)	0.77 m (t=0) 0.95 m (t=379 days)	0.7 m (t=0) 0.96 m (t=379 days)

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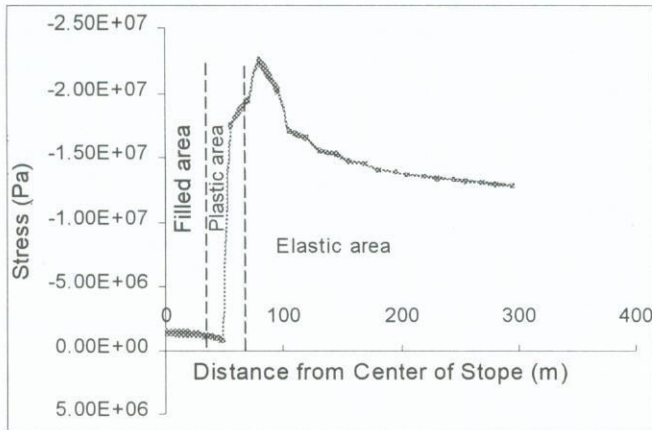


Figure 12. Vertical Stress in Stope 1

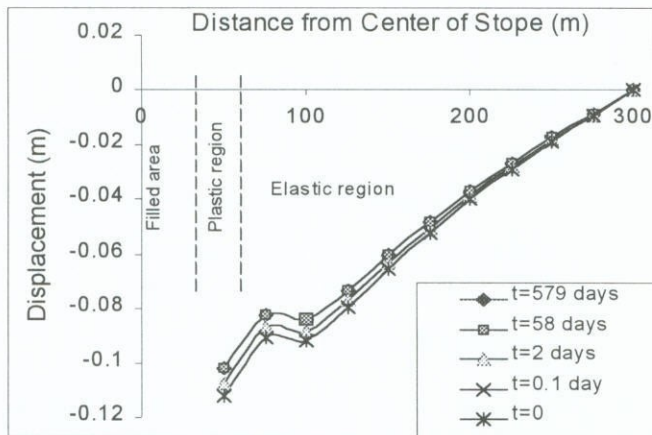


Figure 13. Vertical Displacement in Stope 1

The properties of rock and fill material are the same as for the fully-filled stope and the stope height is 50 m for all four fill conditions. Figures 14 to 16 show the simulation results for various fill conditions in the stope. Figure 14 displays peak vertical stress in the wall of stope under various fill conditions, Figure 15 vertical floor heave and Figure 16 vertical displacement on the roof.

From the simulation results, the following conclusions are drawn:

- Although the peak vertical stress in the wall of the stope drops a little when the contact area between fill material and roof decreases, the change is insignificant (Figure 14).
- The vertical stress in the filled region decreases when contact area decreases. The magnitude of vertical stress drops by about 18% from Model 1 (100% contact with roof) to Model 4 (30% contact with roof).
- The vertical floor heave increases when the contact area decreases (Figure 15). The magnitude of floor heave increases by 8.3% from Model 1 (100% contact with roof) to Model 4 (30% contact with roof).

- The vertical displacement on the roof increases when contact area decreases (Figure 16). The magnitude increases by 5.7% from Model 1 (100% contact with roof) to Model 4 (30% contact with roof).

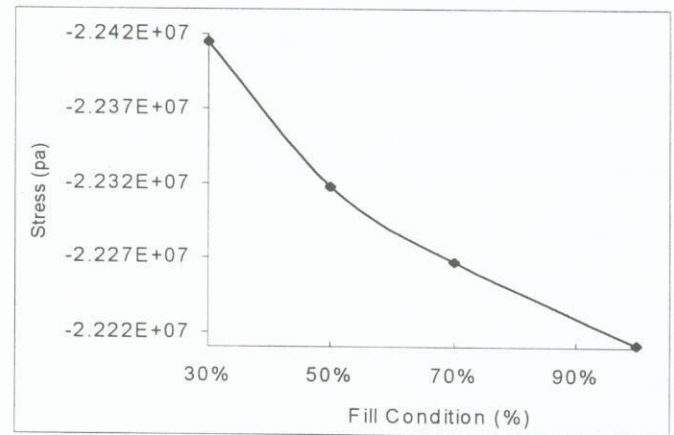


Figure 14. Peak Vertical Stress in the Wall of Stope with Various Fill Conditions

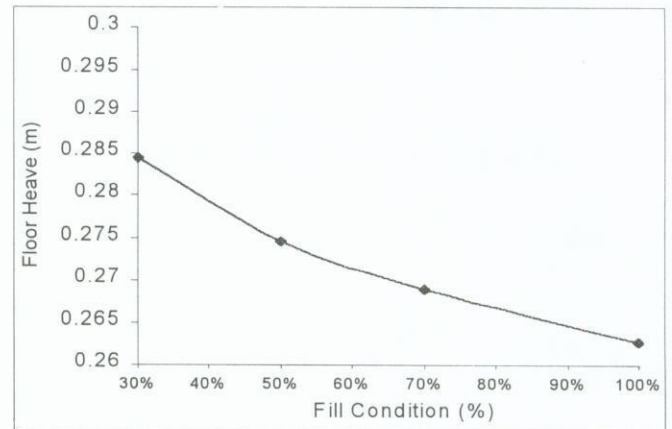


Figure 15. Vertical Floor Heave in the Stope with Various Fill Conditions

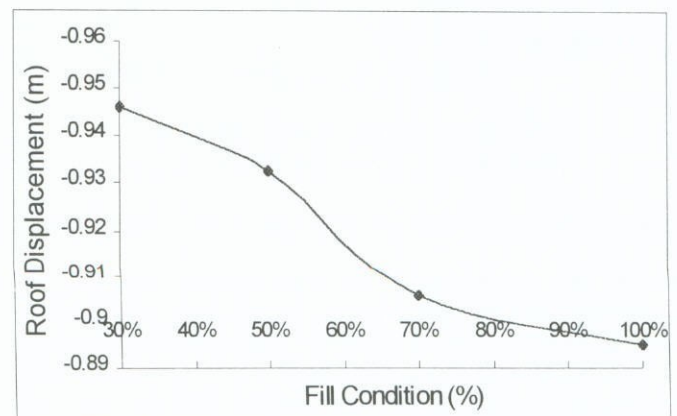


Figure 16. Vertical Roof Displacement in the Stope with Various Fill Conditions

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Influence of Nearby Drifts to the Stope At Jinchuan Mine, the mining service drifts are located outside the stope. The distance between the drift and the wall of the stope ranges from 10 m to 50 m. In order to simulate this situation, three models are established:

- 1) Model A, drift located 12 m from the wall of stope;
- 2) Model B, drift located 28 m from the wall of stope; and
- 3) Model C, drift located 52 m from the wall of stope.

The dimension of the drift is 6 m x 5 m (width x height). The simulation results are shown in Figures 17 to 19.

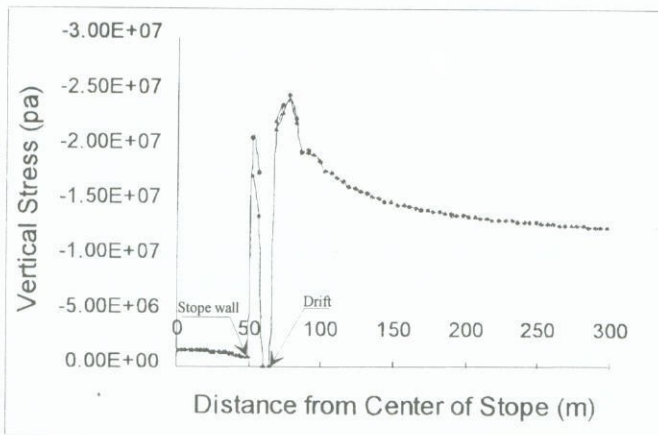


Figure 17. Vertical Stress in Model A
(With Drift 12 m from the Wall of Stope)

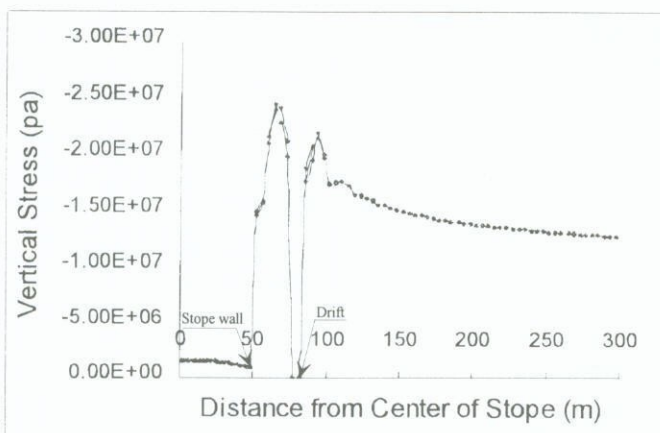


Figure 18. Vertical Stress in Model B
(With Drift 28 m from the Wall of Stope)

From the simulation results, the following conclusions are drawn:

- Since the dimensions of the stope and the drift are very different, the stope being about 10 times the size of the drift, the drift has very limited influence to the stability of the stope. The distribution of stress around the boundary of the stope with adjacent service drifts is only slightly different from that without service drifts.
- The stope has significant influence on the drift. In Model A (Figure 17), not only the drift is located in the plastic area, but

also a very high peak stress occurs. The peak stress of Model A is 12.2% more than that of Model C. In Model B (Fig. 18), the drift is located in the peak stress area. It also has a high peak stress, 10.4% more than that of Model C. In Model C (Fig. 19), the influence of stope to the drifts is very insignificant. The magnitude of the peak stress is approximately the same as the stope without service drifts. The only difference is that without service drifts, high vertical stress over 20 MPa occurs only in a very narrow region, while with service drifts, high vertical stress over 20 MPa occurs in an approximately 20 m-wide area.

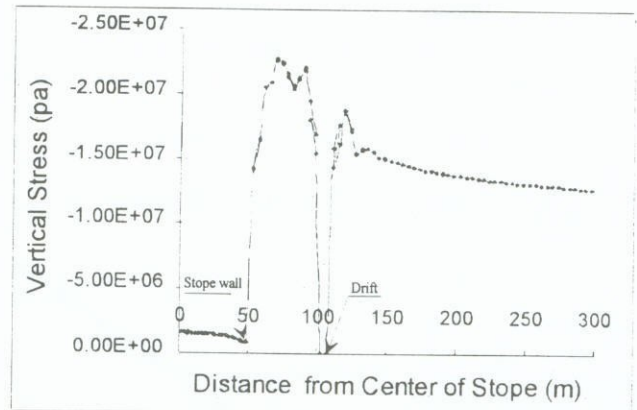


Figure 19. Vertical Stress in Model C
(With Drift 52 m from the Wall of Stope)

SUMMARY AND CONCLUSIONS

The findings of the finite element simulations of the underground drift and the cut-and-fill stopes are summarized below and recommendations for mine ground control are also provided.

- In the wall of the drift, the plastic region is about 1.5 m. The peak vertical stress varies from 1.4 to 1.52 times of the in-situ vertical stress from time=0 to time=579 days. However, more than 90% of vertical stress increase takes place in 60 days.
- The tensile stress region is about 0.5 m deep into the wall of the drift, with a height of the region about 1.5 m. This is the main reason that the drifts are always damaged at both sides of the wall.
- Simulated displacement in the roof of the drift has good agreement with the results of field monitoring. The time-dependent deformation or creep is more than 80% of total deformation.
- Displacement increases rapidly in the first 15-20 days. More than 90% of total displacement takes place in 60 days.
- The decrease of vertical stress in the wall of the stope is very insignificant. From time=0 to time=579 days, the change is only 2%.
- The vertical stress in the center of filled region increases for



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about 25% to 30% from time=0 to time=579 days. This result indicates that some parts of stress in the wall of the stope transferred into the filled region.

- No tensile stress appears on the wall of the stope. This indicates that the fill material plays a very important role in enhancing the stability.
- With an increase of the stope height, the size of the plastic region increases. In the stope of height=50 m, the plastic region is 30 m. In the stope of height=100 m, the plastic region is 45 m. In the stope of height=150 m, the plastic region is 55 m.
- With an increase of the stope height, the distribution region of the peak stress is wider, from a point for stope height=50 m to 30 m for stope height=100 m and to 40 m for stope height=150 m.
- The vertical stress in the filled region decreases by 18% from Model 1 (with fill material contacting 100% of roof) to Model 4 (30% contact with roof).
- The vertical displacement on the floor increases by 8.3% from Model 1 (with fill material contacting 100% of roof) to Model 4 (30% contact with roof).
- The vertical displacement on the roof increases 5.7% from Model 1 (with fill material contacting 100% of roof) to Model 4 (30% contact with roof).
- The service drifts adjacent to stope have very limited influence to the stability of the stope.
- The stope has significant influence on the drift. The impact of stope on drift is sensitive to the location of drift relative to the stope.

Based on the results of finite element simulations, the following recommendations are provided:

- Since the tensile stress appears on the wall of the drift and causing most of the damage, temporary supports should be employed immediately when the drift is excavated. The temporary support should be flexible, and provide sufficient resistance.
- A permanent support should be applied in 60 days. The support should be stiffer than the temporary one.
- In the mined stope, the fill materials should contact at least 70% of the roof area.
- The mining service drifts should be located more than 50 m away from the stope if possible.

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Tekflex as a Spray-on Screen Replacement in an Underground Hard Rock Mine

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ABSTRACT

Falconbridge Ltd., a major base metal producer in the Sudbury Basin of Ontario, Canada, identified a need to find a safe and cost effective replacement for welded-wire mesh used to support loose ground during underground nickel mining. Safer and lower operating costs are a must to maintain mine viability in lower base metal price environments. Stringent material strength parameters were put in place to determine if a spray-on liner was acceptable for testing and application trials. Falconbridge Ltd. took a methodical approach to the concept of a spray-on liner as a screen replacement proving the strength characteristics of the liner as a safe, cost effective, and suitable method of supporting ground. Materials testing was undertaken at Laurentian University to further understand the capacity of the material. Laboratory and field tests indicated excellent adhesion and good tensile strength. Further, it maintains significant tensile strength even after elongation of over 25%. Compact, hydraulic mixing and spraying equipment have been developed and successfully utilized in the Fraser Mine of Falconbridge's Sudbury Operations. Currently the Tekflex material has been approved as a wire-mesh replacement and is undergoing full-scale production trials in selected locations at Fraser Mine.

BACKGROUND

Falconbridge Limited in Sudbury, Ontario, Canada is one the world's top base metal producers. Our core business in the Sudbury Basin is nickel and copper production from 6 producing mines. Our Raglan mine in northern Quebec produces high grade nickel and copper, which is refined in Sudbury. Falconbridge has global mining interests, including an open pit copper mine in Chile producing 100,000 tonnes per day.

The support design is based on standard rock mechanics principles and are part of mine standards that are in place at all operations. Support is installed currently with the application of mesh screen in mind. Typically our 6 x 11 ft screen will be supported with mechanical bolts using domed washers and spherical seats on a 3-2-3 pattern. For openings wider than 5 m, 2.6 m or 8 ft support is installed. For development under 5 m in width, 1.8 m or 6 ft. support is installed. Resin grouted rebar is installed as additional support depending on the ground conditions, for the given conditions. Grouted cable bolts are typically in wide spans within stope and where adverse structural conditions require additional support be installed. With the introduction of the spray-on liner, TekFlek we are now able to bolt to support the ground following a 1.1 m² pattern. Split sets can be used in stope walls and in fill abutments. Super Swellex are used periodically as conditions warrant.

In the late 1980's it was recognized that there was a need to change the bolting and screening process with the objective to improve safety and reduce exposure. During the late 80's and early to mid 90's many initiatives to mechanize this process were constantly limited by the ability to handle screen. In 1998, Falconbridge decided to expand the work initiated by Canadian Mining Industry Research Organization on the concept of a spray-on liner as a screen replacement. Two key factors affecting this project at Fraser Mine were the materials handling issue and ground support optimization through better use of our mechanized bolters.

The relatively small cage capacity limits the volumes of shotcrete that can be transported underground to fulfill support requirements. Shotcrete as a support system is well understood and used at other Falconbridge mines, but its use is difficult to justify at Fraser Mine because of the amounts of materials required, and the

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mobilization distances involved. Hence the interest in adopting a spray-on liner as a screen replacement while also reducing the use of the cage for materials handling. As an example, the area covered by a pallet of liner product, versus the area covered by a pallet of screen is about 3:1.

As part of the ground support optimization review we recognized a potential saving if we reduced the number of bolts installed, and better utilized our mechanical bolters. A thorough review of our support is underway to decide on a primary support for Fraser mine. We are currently reviewing all support methods and will decide on one preferred support method such as mechanical bolts or resin grouted rebar.

CONCEPT DEVELOPMENT

Falconbridge identified a need to apply a liner that met our stringent environmental workplace requirements. A "wish list" was created of properties that we felt were required. We recognized a need to not expose our underground workforce to any additional safety or health risks. Cost was the next most significant item on this list. The geomechanical properties required in a liner were also reviewed. Several potential liner products were reviewed as part of the initial evaluation, provided they fulfilled our requirements. Each product was tested for suitability and ease of application, and preliminary evaluation and laboratory testing were initiated.

FOSROC Inc. of Georgetown, Kentucky was first to develop a product, called Tekflex that met our initial requirements. Development of several potentially suitable products was reviewed and several versions of Tekflex were tested. Currently, Tekflex is a two part, (liquid and powder) polymer modified cementitious material that is mixed together to form a single sprayable product. Tekflex was developed to be as user friendly as possible. Several other materials for spray-on liners were eliminated from consideration due to health and safety concerns for the workers. With Tekflex, only splashes on the skin and dust inhalation pose any risk or mild concern. Standard protective equipment of gloves, safety glasses and a dust mask is recommended when using Tekflex. Underground environmental sampling and testing by Falconbridge's environmental department for the joint health and safety committee confirmed the lack of harmful materials.

Tekflex is sprayed with a highly mobile shotcrete boom arm to cover the back to a thickness of about 4 mm, and the walls to a thickness of 3 mm. Uniform coverage and build up of material is easily completed after the area has been mechanically scaled and bolted using a mechanical bolting jumbo. The spray

arm is mounted on a scissors lift truck modified for the purpose of spraying the liner for the trial period. Fosroc assisted in spray equipment development to meet the needs of the carrier selected and application requirements. A mobile unit has been ordered specifically for the spraying of membranes at Fraser Mine.

Underground and laboratory testing on the current version of Tekflex were undertaken to develop confidence in the material properties, and the concept of a liner support system. Adhesion and tensile strengths of the membrane are considered to be key elements in the support process. Membrane elongation and resistance to tearing under normal conditions are also required.

Laurentian University's Geomechanics Research Centre in Sudbury, Ontario was chosen as an independent source to evaluate and perform the initial testing, and to report on the results. Laboratory tensile and adhesion testing were completed to develop a profile of the materials capacity over specific time windows. We wanted to send crews back under the liner after 8 hours, or sooner to start the mining cycle again. The Tekflex material can be engineered to achieve tensile strengths of 1 MPa after 8 hours.

APPLICATION PROCESS

Drilling and loading of development rounds to a mine standard have been established as the best way to continuously produce consistent ground conditions. Perimeter control blasting techniques are considered mandatory for producing good quality ground conditions.

Once a heading is mucked out, the area is mechanically scaled to bring down large loose material. The round is then bolted with the mechanical bolter. At this time the ground is considered supported. Ground control recommendations for the area are strictly adhered to. The bolted round is then prepared for liner application by the mobile spray unit. High pressure water scaling and manual check scaling are completed from the spray unit in preparation for liner application. Once this process has been completed, the liner materials are batched and readied for application. A calculated quantity of the liner material is sprayed onto the surface, which includes extra material to account for an assumed 35% surface roughness factor. A total of 200 litres is sprayed in a 4.6 m wide by 3.5 m high advance heading, and on the walls to a height of 2.5 m from the floor. This is the area currently covered in the Fraser Mine bolting and screening standard. A total of 300 litres is sprayed in a cut and fill environment where the breasting width is 11 m, with 3.5 m advance. In the future, the support cycle

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will be reassessed to see if it is possible to scale first, followed by spraying the liner, then bolting through the liner to support the ground.

GEOTECHNICAL EVALUATION AND MONITORING

Geotechnical evaluations were established to review Rock Mass Rating (RMR), support optimization, surface roughness, and quality of drilling and blasting. Rock Mass Rating was performed in several areas to recheck our current values. As part of this evaluation, an auditing process was undertaken to check the amount of loose rock contained by the screen on the back and walls. This survey was undertaken in development and active stoping to develop a relationship between the volume and load of loose in the screen, and the percentage of material contained by the screen per area audited. This information will be later used to check the capacity of the screen. A liner audit sheet was made for use underground to rate random locations to evaluate the performance of the liner in varying ground conditions (Figure 1). Fraser mine has ground conditions that vary from good to poor, with minimal structure in the massive granites to highly structured ground in the blocky norite. Our stress levels vary from normal to high stress levels in the area of the liner trials. The liner is being tried in all of these conditions to truly evaluate the range and limits of the application suitability.

Although numerous materials tests have been performed in the laboratory and underground, the question of the validity of these tests still exists. Tensile strength is seen to be one of the key elements in the overall performance of the liner support. Certainly adhesion and elongation play key roles as well. Standard material test procedures, however valid, do not truly represent the liner support function as required in the field. Discussions on how to representatively evaluate the concept of the liner supporting loose ground have been productive and numerous test methods have been suggested. If the liner is subjected to any load, the liner will start to deform and elongate, reaching its plastic limits when tensile cracking occurs. This has been observed underground, as well as in the recent underground testing. The plastic or rupture limit of the material is surpassed when the loose material starts to delaminate and fall from the liner. The current capacity of the liner has been estimated from underground pull testing at ages of 8 and 24 hours. The liner capacity at 8 hours is 1 MPa and is 3 MPa after 28 days.

Laboratory elongation tests indicate that Tekflex can elongate between 12 and 30% before rupturing. Large scale laboratory testing is underway to validate the liner-loading concept. Testing is performed in a 1.1 m² steel frame filled with compacted loose rock material to

fill a 0.3 m high frame. Tekflex is sprayed on the rock surface and allowed to cure. The loading frame is placed under a large hydraulic press and loaded at a controlled rate to simulate creep failure. Review of the load versus deformation characteristics of the liner facilitates estimation of Tekflex's elongation capability. Instrumentation is installed under the membrane to monitor the vertical deformation of the center and the perimeter of the membrane. This method is expected to become our standard way of evaluating membranes in a passive and dynamic stress situation. Figures 2-6 photo document the above process. Figure 7 graphically displays the peak load and center point deformation at failure. Note, the consistent load and deformation behavior after full curing, approximately three days. This test method was developed to simulate "Worse Case" for the TekFlex to be applied in. The test frame size of 1.1 m² represents the span of our bolting standard. The frame depth of 0.3 m to 1 ft represents the typical depth of the loose generated overtime after blasting and scaling. The test frame is filled with lightly consolidated material, covered by slabs of waste granite, which represent the worse case for our support system to be expected to work within. Granite slabs cover the gravel material, and are interlocked to form a flat surface on which to spray. The material is sprayed to an average thickness of 6 mm and allowed to cure for the times required for the specific test. We would certainly not be applying TekFlex in conditions this severe if they were encountered underground. In this case support system using shotcrete would be in place.

In the test trial area at Fraser Mine, eight headings are currently being sprayed to evaluate the liner. Three headings are in cut and fill stoping, and five are in permanent development. Initially all headings were screened after the liner was sprayed to evaluate if any loose would develop in the screen, and to build confidence in this material as a potential screen replacement.

Drilling and blasting quality is constantly checked to develop a relationship between best drilling and blasting practices and what has been accepted as standard drilling and blasting quality. Better liner performance occurs with better drilling, blasting, scaling, and surface preparation. We have found Tekflex to be quite resistant to blast damage, and it continues to hold loose despite damage from blasting.

If all of the quality checks are in place, there is now a high degree of confidence in the use of spray-on liners. The quality of work from our crews was seen to significantly improve. Audits of the drilling and blasting at Fraser mine are showing improvements in perimeter control, drill hole alignment, and a reduction in bootlegs. We are seeing considerable improvements in the quality of the support installed. As an example,

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Fraser Mine Spray-on Liner Quality Audit:

Date: _____ Audit By: _____

Heading No. _____ Drift Rd. _____ Breast Rd. _____ Panel _____ Slot _____

Sprayed Round No. _____ No. of Pails Sprayed: _____ Drift = 10 p 200L / Breast = 15 p 300L

Drilling: Best Practices ☐ Yes ☐ No ☐ Destress? ☐

Blasting: Best Practices ☐ Yes ☐ No ☐ Tracing? ☐

Bolting:

Spacing (m)		Mechanical		Rebar		Cables, spacing	
Bolt loss from blasting:				No. of bolts at face.			
Concerns?				Span (m)			
Bk Bolt Count		LHWall		RHWall		Bolting Density (m)	

Ground Conditions and Liner Quality:

Back:

Scaling Method		Mechanical		High Pr. Water		Check Scale		G r d
Scaling Quality		Good		Fair		Poor		
Sounding		Solid		Drummy		Muffled		
Roughness		Blocky		Slabby		Irregular / Both		
Surface Prep.		Dry		Wet		Damp or Dusty		L l n e r
Visual Quality:		Good		Fair		Poor		
Thickness:		+4mm		2-4mm		0-2mm		
Coverage %		>90%		70-90%		50-70%		
Mat'l Loss		Uniform Cvg.		Slumping		Dripping		
Spray Stand off from Face (m)				0.3m or less		Tight to face		

Wall RHS

Scaling Method		Mechanical		High Pr. Water		Check Scale		G r d
Scaling Quality		Good		Fair		Poor		
Sounding		Solid		Drummy		Muffled		
Roughness		Blocky		Slabby		Irregular / Both		
Surface Prep.		Dry		Wet		Damp or Dusty		L l n e r
Visual Quality:		Good		Fair		Poor		
Thickness:		3mm +		2-3mm		0-2mm		
Coverage %		>90%		70-90%		50-70%		
Mat'l Loss		Uniform Cvg.		Slumping		Dripping		
Spray Stand off from Face (m)				1.5m Std.		Less than 1.5m		

Wall LHS

Scaling Method		Mechanical		High Pr. Water		Check Scale		G r d
Scaling Quality		Good		Fair		Poor		
Sounding		Solid		Drummy		Muffled		
Roughness		Blocky		Slabby		Irregular / Both		
Surface Prep.		Dry		Wet		Damp or Dusty		L l n e r
Visual Quality:		Good		Fair		Poor		
Thickness:		3mm +		2-3mm		0-2mm		
Coverage %		>90%		70-90%		50-70%		
Mat'l Loss		Uniform Cvg.		Slumping		Dripping		
Spray Stand off from Face (m)				1.5m Std.		Less than 1.5m		

Comments: _____

Thickness Measurements Back: _____

LH Wall: _____

RH Wall: _____

Figure 1. Liner audit sheet.

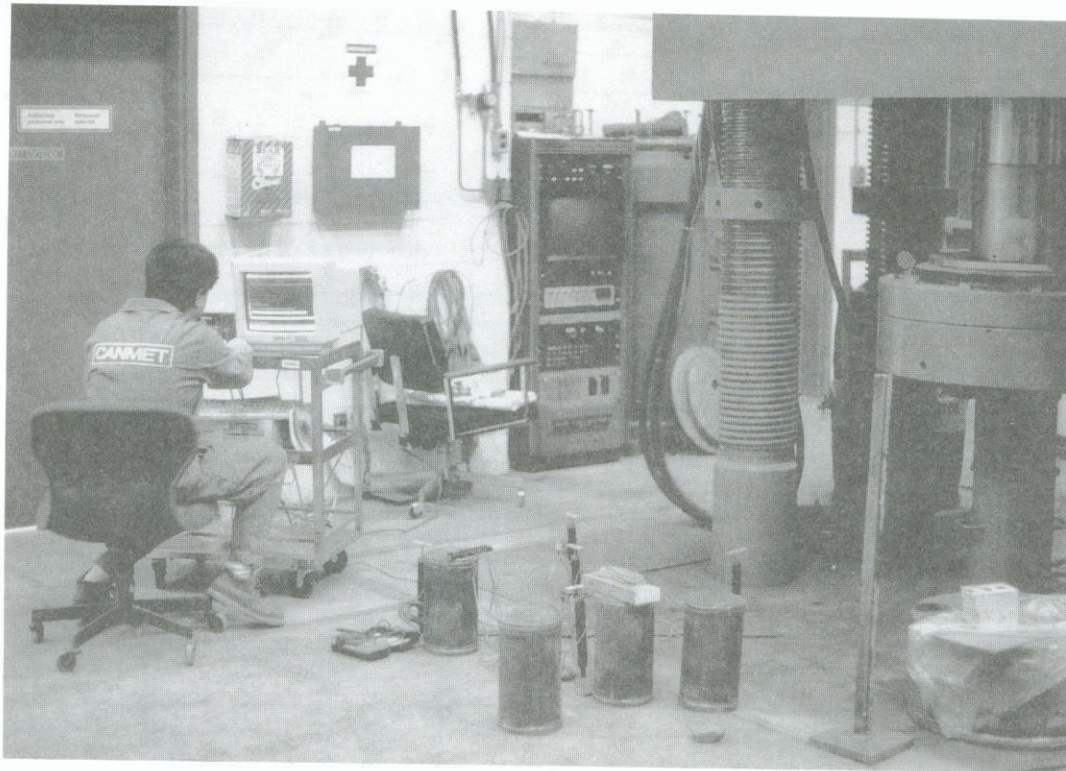


Figure 2. CANMET load frame and control equipment.



Figure 3. Box frame after loading with loose rock and spray coating with Tekflex.

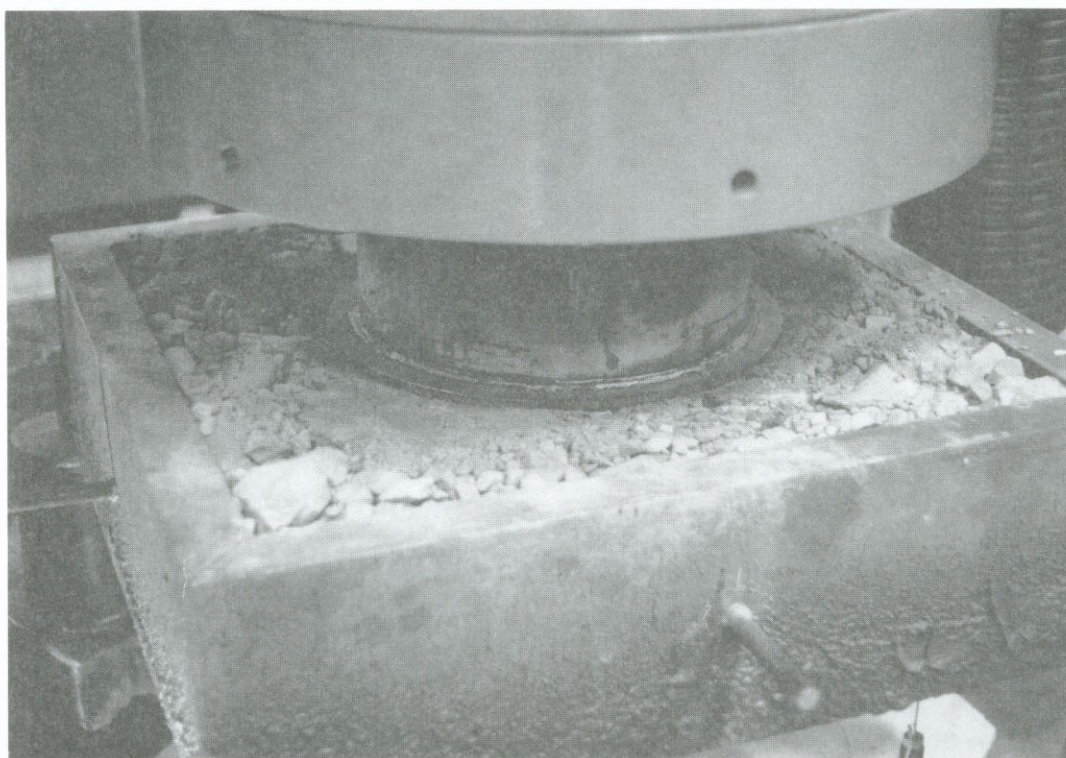


Figure 4. Inverted box frame ready for hydraulic loading.

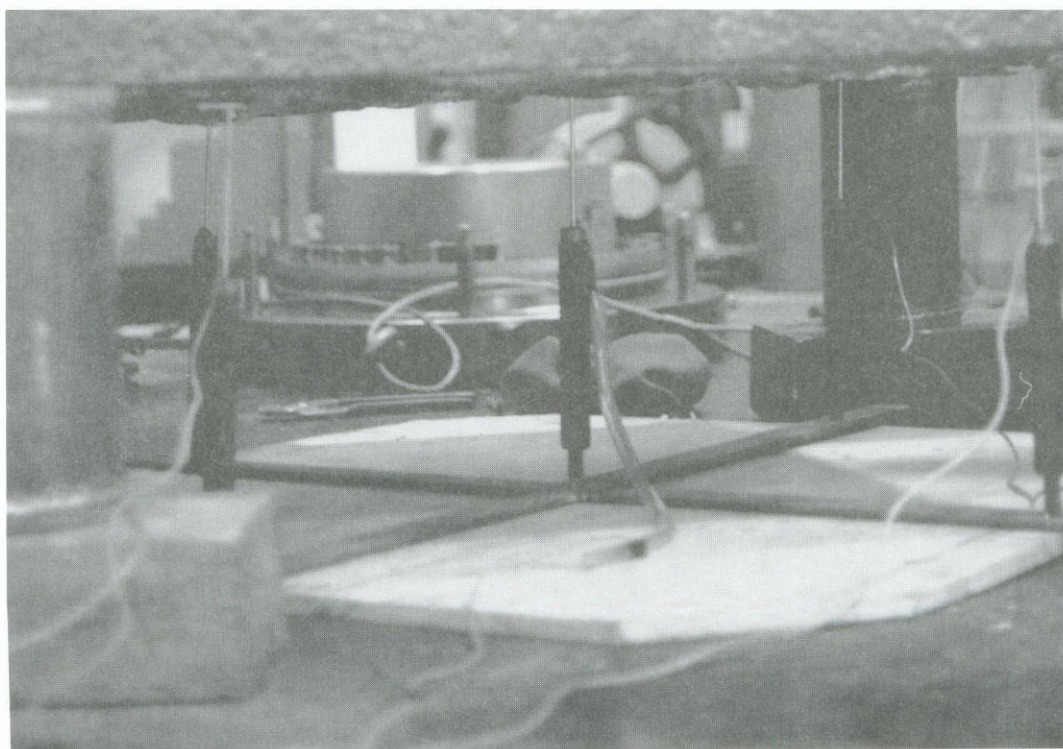


Figure 5. Side view of box frame and deformation monitoring equipment.

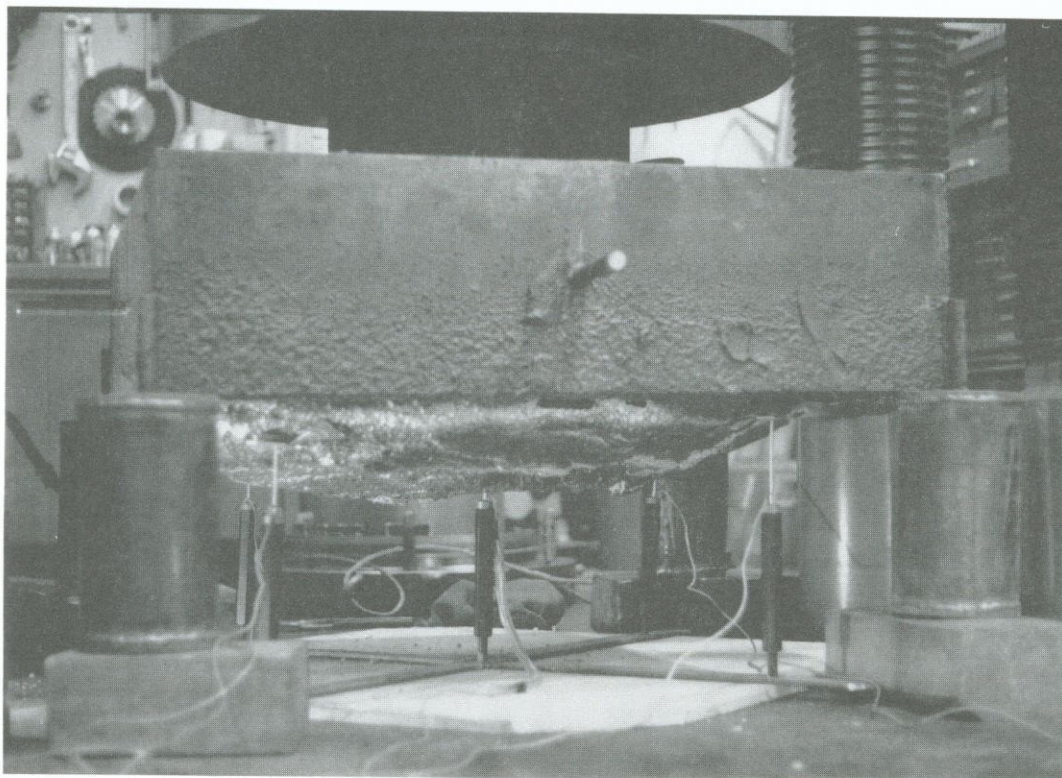


Figure 6. Side view of specimen at peak load.

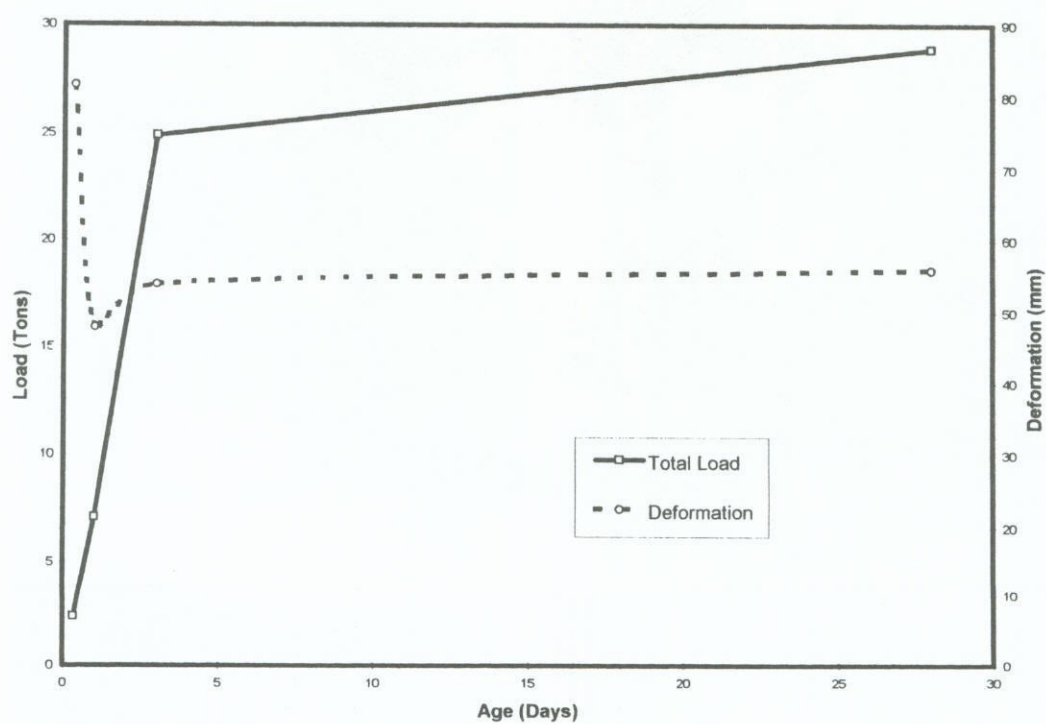


Figure 7. Peak load and maximum center deformation vs. specimen age.



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SUMMARY

we are now installing support to "support the ground" instead of installing support to hold screen in place. Our current practice for installing screen is to install eight bolts per sheet of welded-wire mesh (1.8 x 3.4 m² or 6' x 11' sheets) on a 3-2-3 pattern. The standard bolting density required for our conditions is one bolt per 1.1 m². However, in many cases this practice causes us to install more bolts than are required for the ground conditions. We are hopeful we will be able to open our bolting pattern to a maximum of one bolt per 1.5 m² should the concept be proven. Audits of the support installation are showing that we are obtaining the 1.1 m² spacing, with seeing ground conditions improving as a whole.

As part of the quality assessment, we are rating rounds based on overall quality, and developing a numerical rating that will be used to check the amounts of the liner used versus surface roughness due to drilling and blasting practices. There is a direct relationship between the quantity and quality of the liner application and the surface area and roughness. Several rounds in highly structured and blocky ground were not sprayed with Tekflex because surface roughness calculations indicated that excessive quantities of liner material were required per unit surface area. These rounds were bolted and screened conventionally to better fit our operating costs. Currently we have sprayed in excess of 50 rounds with exclusive use of bolts and Tekflex as a screen replacement.

Falconbridge Ltd. took a methodical approach to the concept of a spray-on liner as a screen replacement proving the strength characteristics of the liner as a safe, cost effective, and suitable method of supporting ground. Materials testing was undertaken at Laurentian University to further understand the capacity of the material. Laboratory and field tests indicated excellent adhesion and good tensile strength. Further, it maintains significant tensile strength even after elongation of over 25%. Compact, hydraulic mixing and spraying equipment have been developed and successfully utilized in the Fraser Mine of Falconbridge's Sudbury Operations. Currently the Tekflex material has been approved as a wire-mesh replacement and is undergoing full-scale production trials in selected locations at Fraser Mine.

Tekflex has demonstrated to be a superior product in relation to blast damage resistance. We are realizing less maintenance than we have had in the past with screen. The long-term stability of this product is still under consideration.

As part of our commitment to improved safety Falconbridge will be seriously reviewing the concept of using spray-on liners in dynamic stress environments. Our Onaping Depth deep mining project is in the feasibility stage and support system viability is being studied. If we can develop and use a highly deformable, high tensile strength membrane with superior adhesion, the support options available to Falconbridge Limited will increase as we continue to push the limits of technology and to mine safely at greater depths.



Tunnel Deformation Monitoring "Action Levels" in Coal Mines for Support/Reinforcement Design

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ABSTRACT

Mine tunnel support using rockbolts was introduced in UK coal mines over the period 1987- 1992 and is now used in over 70% of all mine tunnels. In every case the reinforcement design has been based on the observational approach. After the initial reinforcement design has been established appropriate action levels are defined for predetermined amounts of movement indicating the onset of instability. If the action levels are exceeded appropriate remedial action is taken usually in the form of additional support or reinforcement to maintain stability.

This paper outlines the observational design method as used in UK coal mine tunnels. It discusses the low cost instrumentation which has been developed to monitor roof deformation, dual height telltales, how action levels are defined based on the geotechnical environment and the types of remedial measures typically taken.

An alternative approach for dealing with areas of increased deformation involves reacting to early roof movement trends and modifying the support installed at the face of the development accordingly. Installation of additional support at the face of the heading improves ground control in poor geotechnical conditions and has the advantage of potentially reducing later additional support costs. A case study is described to illustrate how 'early' action levels can be incorporated into a roof support plan enabling a flexible roof support strategy. Such a system will be site specific and continued analysis of monitored information will be required to confirm the action levels or to adjust them according to any change in geotechnical environment. More accurate roof deformation monitoring systems, such as the Remote Reading Telltale system recently developed by Rock Mechanics Technology Ltd. (RMT) and currently undergoing Mines Safety and Health Administration (MSHA) approval, are ideally suited to this application. The Remote Reading Telltale system provides accurate mine wide monitoring of roof condition on a surface PC.

INTRODUCTION

Rockbolting has become an established means of support in UK coal mines over the last ten years. For many years prior to 1989, no rockbolted roadways had been driven due to fears over safety and reliability. The safe adoption of rockbolting in the relatively weak and highly stressed UK geotechnical environment was achieved through British Coal's policy to adopt the best technology available. The main technical factors adopted, collectively termed Advanced Technology, AT, rockbolting are:¹

- (i) a reliable design methodology, based on an observational approach of design by measurement,
- (ii) a routine monitoring system to ensure timely installation of remedial support wherever required,
- (iii) an understanding of rock stress and a means to measure it in-situ,
- (iv) high strength and stiffness consumables, and,
- (v) a unified approach incorporated into a Code of Practice, now the DMCIAC (Deep Mines Coal Industry Advisory Committee) Guidance on the Use of Rockbolts to Support Roadways in Coal Mines.²

This paper focuses on the routine monitoring system, an integral part of the observational design approach for rockbolted roadways which is used to ensure that stability is maintained throughout the working life of the roadway.

The routine monitoring device, the telltale, is essentially a wire extensometer with two anchor points located above the roadway roof. They are configured such that the two reflective indicators provide separate visual indications of movement within and above the rockbolted height. They provide a visual warning of excessive movement and allow an accurate reading in millimetres. These are installed at regular intervals and read frequently. In order to derive maximum benefit from telltale results, a sound understanding is required of roadway deformation characteristics. Over the last ten years numerous roadway deformation characteristics have been identified in UK coal mines. This has led to the



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determination of site specific action levels whereby telltale data is appropriately used to indicate the onset of instability and any requirement for additional support.

The concept of maintaining roadway stability using a novel approach for interpreting telltale results has been investigated. The approach involves reacting to early roof movement as measured by the telltales and modifying the reinforcement/support installed at the face of the heading accordingly. It is envisaged that this approach could reduce the cost of additional reinforcement repairs outbye. This is achieved by additionally reinforcing the ground, which, under the basic support design, would later go on to behave in an unacceptable manner. This concept relies on the basic assumption that the roadway movement recorded by the telltales at the face of the heading within the first two weeks or less can reliably indicate the long term behaviour of that particular section of roadway. A case study is discussed and recommendations made regarding the feasibility of such a design approach.

SUPPORT OF UK COAL MINE TUNNELS

Introduction of Rockbolting to UK Coal Mines

Rockbolting was first introduced into UK mines in 1946, making use of mechanical point anchored bolts which were used in subsequent trials up until the 1970's. The trials proved to be unsuccessful with the systems proving unsuitable for many UK conditions and some major collapses resulted due to inadequate equipment and a lack of understanding of the mechanics involved.

In the mid 1980's systematic investigations of American and Australian bolting systems and techniques were undertaken. This resulted from a policy decision to improve UK coal mine productivity by shifting from advancing to retreat longwall mining, which world wide experience recognised as being the most efficient method of mining underground deep coal. This required that rapid driveage and retreat rates be achieved and the successful introduction and use of rockbolted support became essential. The rockbolting trials undertaken in the 1980's found the Australian bolting systems, which utilised rockbolts fully bonded with polyester resin, to be the most effective for the UK conditions.³ The introduction of these rockbolting techniques improved heading advance rates from 20-25m per week to over 100m per week and the improved support offered by rockbolting also allowed material costs to be halved and materials transport costs to be reduced. There was also a marked reduction in accident rates due to falls of ground over this period, which, at least in part, can be attributed to the monitoring systems adopted.⁴

UK Mining Conditions

Compared to countries such as USA, Australia and South Africa, mining conditions in the older coal mining areas of Europe are more challenging with greater working depths,

typically around 800m, weaker roof conditions which often comprise low strength mudstones, and interaction effects from multi-seam mining.

In the UK, as in many other parts of the world, it has been found that the stress field is a major factor in dictating mining conditions. The vertical stress is typically proportional to depth of cover with a non uniform horizontal stress. The ratio between the maximum and minimum horizontal stress components can be as great as 2:1 with the major horizontal stress in the UK being NNW-SSE.⁵ This has a marked effect not only on roadway deformation characteristics but also stress concentration effects around longwall faces.⁶

Standard AT Rockbolt Support System

The rockbolting system used in UK coal mines utilises consumables which produce a high strength, high stiffness system. With only one specification for these rockbolting materials there can be no confusion with respect to which materials can be installed. The consumable items are produced to strictly monitored quality standards, set out in the British Standard for strata reinforcement support system components used in coal mines.⁷

The 'Advanced Technology' or 'AT' system specifies 22mm diameter bolts with a special rib profile and a yield strength of 25 tonnes. The bolts are installed into holes with a nominal diameter of 28mm which results in a small annulus to maximise bond strength. A special high strength polyester resin is used to fully encapsulate the rockbolts and this is supplied in capsules for simple, rapid installation.

OBSERVATIONAL DESIGN

Observational Design Approach

The approach required to design the single entry roadways which serve the UK's longwall retreat panels needs to accommodate the potentially variable geotechnical conditions that can be encountered. The only reliable design procedure for these conditions is one based on detailed measurement of strata movement and bolt system loading at each site. This design by measurement methodology has the same underlying principals as the observational approach used for New Austrian Tunnelling Method, NATM, tunnels. Recent publications have described this observational approach as applied to geotechnical design.⁸ The observational design process and it's application to rockbolting are shown in Figure 1.

Code of Practice/DMCIAC Guidance on the Use of Rockbolts to Support Roadways in UK Coal Mines

In order for rockbolting to be introduced successfully and safely into UK coal mines interested parties had to be persuaded that the system was safe and responsibly applied. These interested parties were Her Majesty's Mines Inspectorate (HMI), Mine Management, Trade Unions and

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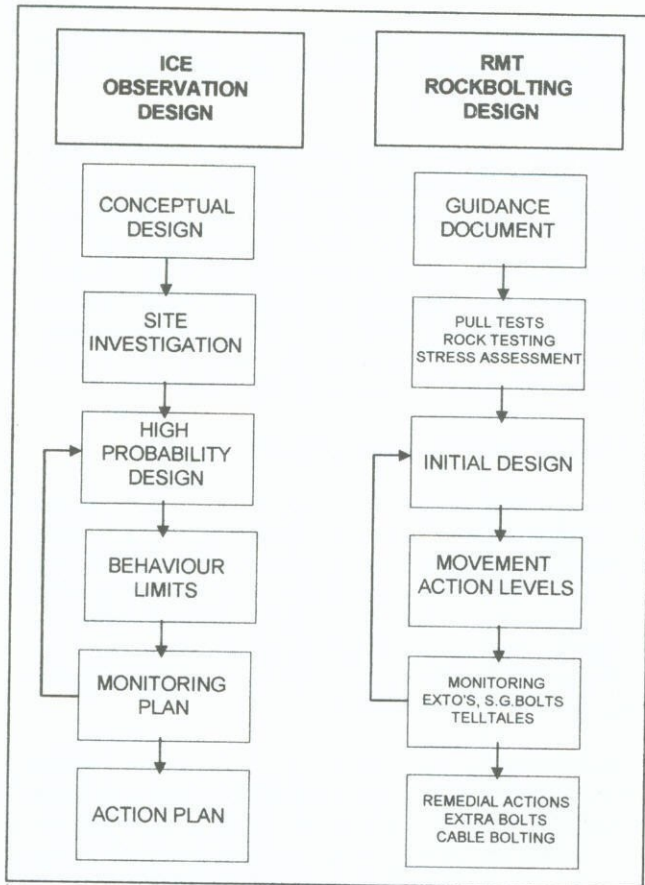


Figure 1. The Observational Approach and its Application to UK Coal Mine Rockbolt Design (ICE Institution of Civil Engineers⁸)

the Workforce. As rockbolting was seen to be a potentially important step forward in order to improve productivity and safety any early failures would have curtailed its introduction into UK coal mines. It was therefore important to have set guidelines for its introduction, especially in light of the fact that the British mining legislation, last revised in 1966, makes no reference to the use of rockbolts.⁹

When rockbolting was first introduced the safety of the bolting system at each mine site was first visibly demonstrated by using the rockbolts in conjunction with reducing levels of steel standing support. The decision to proceed to rockbolts as the main support was subject to suitable monitoring results and acceptance between the HMI, Management and Workforce. Therefore to ensure the same practices were applied at each site in July 1990 British Coal, in collaboration with the HMI, issued 'The Code of Practice for the Support of Roadways by Rockbolts'.¹⁰ This was updated in 1993 and subsequent to the Privatisation of the industry replaced by the DMCIAC Guidance document on the Use of Rockbolts to Support Roadways in Coal Mines.²

Observational Design Approach Applied to UK Coal Mine Rockbolted Roadways

Normal practice in the UK is to commence bolting at a new site using an initial rockbolt design in conjunction with steel supports whilst measurements are taken and the design finalised. This initial design is based on geotechnical assessment of the site, experience (taking into account the expected conditions) and the use of the roadway, for example gate road or longer term trunk roadway.

An initial geotechnical assessment of a site to be rockbolted will assess numerous factors including the geology, load transfer characteristics (bond strength) of the support system and the in-situ stress field. The roof geology is investigated by obtaining a core which is logged and tested to locate any weak strata where shear failure may preferentially occur. The bond strength between the rockbolt system and the strata is measured using short encapsulation pull tests at a number of horizons above the roof.

Detailed rock movement measurements are taken using a specially designed sonic extensometer in a 7.5m/23ft long vertical hole into which 20 magnetic anchors (measurement points) are placed. The relative movement of these anchors indicates where the rock is moving. The other essential information is provided by the measurement of load in the bolts. The technique used provides a detailed and precise understanding of the bolt load profile which is used to improve the design of the system. Bolts instrumented using strain gauges are installed instead of the standard bolts during drivage and are used to measure axial and bending strains in considerable detail. Build up of load with time can be plotted and contour mapping of load and strain will readily indicate high strain zones within the rockbolted horizon. A typical design monitoring station is shown in Figure 2.

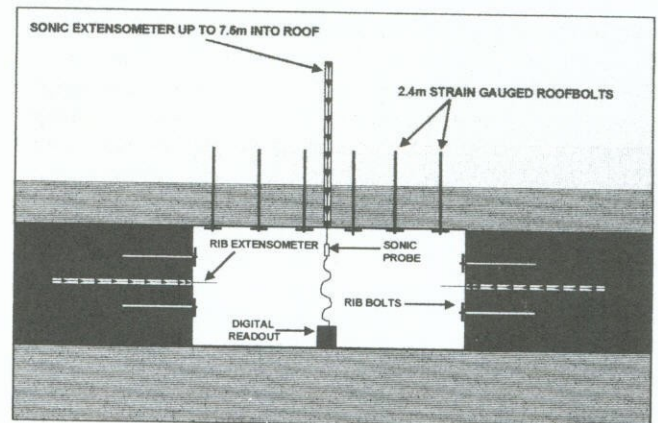


Figure 2. Typical Design Monitoring Station

The information from the geotechnical assessment and the design monitoring station is used to either confirm or improve the initial rockbolt design. Where the design is modified a further design monitoring station is used to verify this modified design. Once the design has been established routine monitoring continues to confirm the design as drivage progresses.

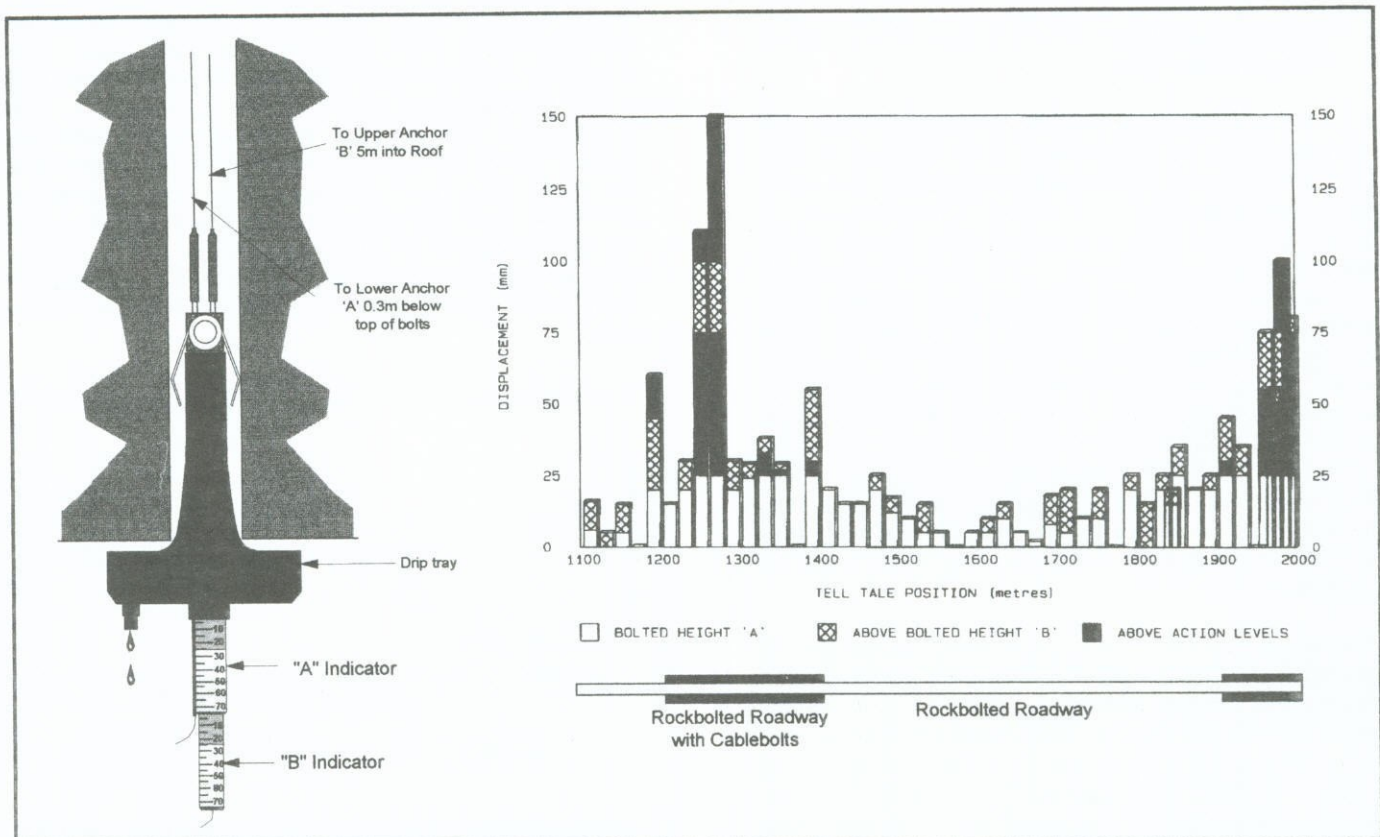


Figure 3. Dual Height Telltale and Typical Results

ROUTINE MONITORING AND THE USE OF DEFORMATION ACTION LEVELS FOR DESIGN MODIFICATION

Routine Monitoring

Once the reinforcement design has been established routine monitoring devices are installed in order to confirm the support design. Within the UK coal mining industry these measuring devices take the form of the dual height telltale which is installed at regular intervals to measure roof movement. For every rockbolt site roof movement 'action levels' are derived which refer to the amounts of roof movement within and above the rockbolted height that can be safely allowed before action is required to prevent a roof fall during either roadway development or face retreat. If action levels are reached action is undertaken to secure the roof by the installation of remedial support or reinforcement.

Dual Height Telltale

The dual height telltale is an extremely effective measuring device that was developed in-house by British Coal at the time of the introduction of rockbolting into UK coal mines in the late 1980's. They are essentially wire extensometers and are designed to be installed quickly by the heading team at the face of the reinforced roadway at regular intervals (20m/60 ft in the UK). Should roof movement occur the

dual height telltales will give a visual warning of the magnitude of strata movements along the rockbolted roadway. Figure 3 shows a dual height telltale. It consists of two anchor points, one within the rockbolted height, normally 0.3 m/1ft below the top of the roofbolt, and one above the bolted height, normally 5m/15ft into the roof. The device is capable of indicating a total of 150mm/6in of strata movement, 75mm/3in from each anchor point. The indicators have coloured bands which are 25mm/1in wide and coloured in a similar manner to a traffic light. The first 25mm/1in band is coloured green, the second 25mm/1in-50mm/2in band is yellow and the band indicating greater than 50mm/2in of movement is red. Movement of the top indicator, 'A', to its reference, bottom of the telltale mouth anchor body, represents the total strata movement within the rock bolted height. Movement of the bottom indicator, 'B', relative to its reference, bottom of indicator 'A', is equivalent to the strata movement above the rockbolted height. The total strata expansion is given by summing the movements recorded on the 'A' and 'B' indicators. Systematic procedures for installation, inspection, recording and reporting of readings from the telltales form an essential part of the routine monitoring plan. Minimum procedures adopted in UK coal mines are given in the DMCIAC Guidance document.²

Determination of Action Levels

Appropriate action levels need to be identified for each

rockbolted site. This is because upon excavation each different geotechnical environment will have its own distinct deformation characteristics.

In the UK coal mining industry the nature of the rock deformation can be determined from analysis of data recorded by the sonic extensometers installed as part of the design monitoring system. The sonic extensometers are used to determine when the load bearing capacity of the strata is reduced or 'softened'. The strain-displacement plots recorded by the sonic extensometers can be used to determine the type and location of rock softening in the roadway roof. These locations will be dependant upon the strength and nature of the strata about the seam and the magnitude and geometry of the stress field about the excavation. The distance into the roof to which 10 mm/m or 1% vertical strain has been exceeded is termed the Height of Softening, HOS. Based on the HOS concept numerous types of roadway failure characteristic have been observed in UK coal mine roadways since the introduction of rockbolting.¹¹

The term HOS therefore refers to the distance into the roof strata where concentrated in-situ stresses have caused significant strata failure leading to a reduced load bearing capacity. It is this reduced load bearing capacity that leads to roadway instability and if sufficiently reduced a fall of ground can occur. In order to maintain stability the HOS should be maintained within the reinforcement height. The maximum roof movement at which the HOS progresses above the reinforced height can be used to define the action levels.

The usual action levels set for telltales at new sites follow the recommended action levels set in the DMCAC Guidance document², these being A > 25mm and B > 25mm. These levels approximate to a strain level of 10 mm/m or 1% within the rockbolted height, 2.4m/8ft (measured to 2.1m/7ft m by the A telltale), and in the monitored zone above the rockbolts, 2.1m/7ft - 5.0m/15ft (measured by the B telltale) if the roof movement was evenly distributed. However because of the following variables the general action levels are not always applicable:

- (i) roof movement is not normally evenly distributed,
- (ii) in-situ stress levels in the roof vary,
- (iii) different deformation characteristics occur due to variable geology.

This means that site specific action levels may be required. For example in Figure 4 the HOS profiles are shown for four different types of roadway deformation characteristic observed in the UK. In the case of 1 and 2 good conditions are maintained with low movements, for case 2 the rockbolted support should be adequately reinforcing the softening within the first 1m of roof which occurred after 6mm of movement. However in case 3 after just 16mm of total movement (A + B on the telltale), softening is occurring above the rockbolted height and remedial reinforcement is required. The sonic extensometer at this site indicated that softening occurred after just 6mm of movement above the bolts and 10mm within the reinforced height. Case 4, progressive softening up into the roof,

follows a similar trend. The HOS progresses above the reinforced height after 18mm of total movement. The progressive development of softening in this roof is illustrated in the extensometer results shown in Figure 5. The extensometer indicates that at the time roof softening occurred above the bolted height only 5mm of movement was measured above the bolted height. Consequently in both these cases the action level for the B telltale would need to be reduced to within the order of 10mm. These examples therefore show how the monitoring information needs to be interpreted to determine site specific action levels.

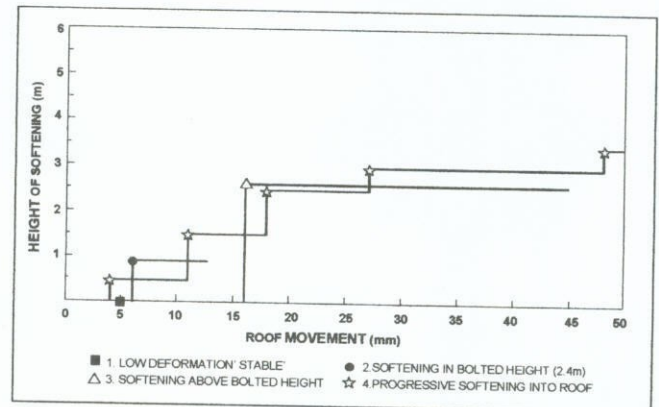


Figure 4. Height of Softening / Displacement Characteristics for Different Rockbolted Roof Behaviours in the UK

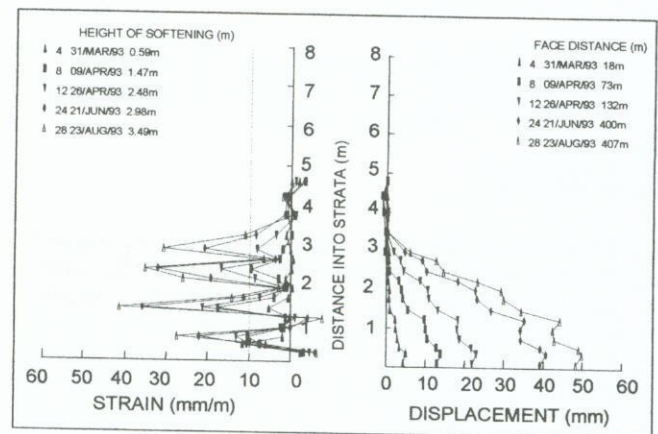


Figure 5. Sonic Extensometer Plot Illustrating Progressive Deformation into the Roof

Remedial Action

A range of additional support measures can be implemented in rockbolted roadways if the roof movement action levels are exceeded.¹² Although these include the setting of free standing support such as wooden cribs or steel legs, the installation of additional reinforcement has been found to be generally the most effective means of restricting further roof movement and maintaining access clearance.



For movement developing within the rockbolted height, installation of additional rockbolts can be the most appropriate action. For movement above the rockbolted height, the installation of birdcaged cablebolts has proved successful.¹³ Birdcaged cablebolts are partially rewound steel strands installed in a high strength grout, usually to a height of 8m. The combination of high capacity cables, 60 tonne, and high stiffness associated with the birdcaged configuration provides effective reinforcement to the roof. Additional special dual height telltales are then installed to detect any further movement above the cablebolted height.

Normally the requirement for additional action is restricted to small sections of rockbolted roadways where local geological features such as faults or lithology variation result in increased roof movement, Figure 3. However, in some cases this requirement may become extensive and indicates that modification to the original bolting design is needed. Depending upon circumstances this may include additional rockbolts or cablebolts installed systematically as part of the reinforcement pattern or at a predetermined distance behind the face of the heading. The recent development of 'flexible' bolts, consisting of steel strand finished with a surface profile to give high bond strength and fully encapsulated using polyester resin, allows longer length reinforcement to be installed at the face of the heading. These flexiblebolts, up to 5m long, are proving effective in restricting movement above the rockbolts thereby avoiding a later requirement for cablebolting.

CASE STUDY : NOVEL APPROACH FOR USING EARLY TELLTALE MOVEMENTS FOR REINFORCEMENT MODIFICATION AT THE FACE OF THE HEADING

Need for a Novel Approach

At the mine where this study was undertaken the level of overall stability for the rockbolted developments was generally high with conditions continuing to remain stable for successful face retreat. There were, however, areas within a section of the mine which exhibited increased roof deformation during development as a result of an increase in the thickness of a weak mudstone above the seam. These areas required additional support outbye in the form of flexible and cablebolts.

The installation of this additional flexiblebolt and cablebolt reinforcement outbye of the face was expensive and sometimes interfered with the normal development operations and hence reduced advance rates. As a result an alternative approach was proposed for dealing with the areas of increased deformation. The approach involved reacting to early roof movement as measured by the routine monitoring telltales and modifying the support installed at the face of the development accordingly. In modifying the support installed at the face improved roof control was anticipated and it was envisaged that ground which would otherwise have behaved in an unacceptable manner with the basic support design would remain within acceptable

movement limits with the modified support pattern. It was accepted that whilst this approach could reduce the cost of expensive outbye repairs, it was unlikely to be eliminated entirely due to the reaction time in identifying these areas as they were entered.

To utilise successfully early roof movements as part of a support strategy for gate roadways driven in the area where the mudstone thickness was expected to vary, definition of the appropriate action levels was required. These early telltale action levels could then be used to modify the installed support at the face of the heading to suit the prevailing conditions. This type of support strategy is summarised as a flow chart in Figure 6.

Two sets of action levels were needed. Set 'one' to introduce additional support such as flexiblebolts, set 'two' to stop setting additional support. Existing colliery action levels could thereafter be used to assess the long term requirement for further remedial support, for example cablebolts.

Determination of Early Telltale Movement Action Levels for Installation of Additional Reinforcement at the Heading Face

The early warning action levels were determined through a detailed analysis of telltale movements recorded in four roadways all of which had been subjected to a variation in roof behaviour as a result of the lateral change in mudstone thickness above the seam.

The approach adopted was to analyse the behaviour of the telltales over three months following installation to determine if roof movement initiated early in the life of the roadway could provide a good indication to its future behaviour. The movements were analysed in terms of total dilation and movement of the individual 'A' and 'B' devices.

Graphical analysis of the data (time verses displacement plots for each telltale) indicated that it was feasible to predict future roof behaviour from early telltale movement. A statistical analysis was then carried out to determine the level of confidence of using the early telltale movements to indicate the subsequent behaviour of the roof. The approach was based on the readings taken 6-13 days after telltale installation and two scenarios were considered based on the limiting criterion of either:-

- (i) upper roof B movement of 5mm or more being measured, or
- (ii) a total movement (A+B) of 10mm or more.

Based on these threshold values of roof movement an assessment was undertaken to determine if the second telltale measurement could indicate the future roof behaviour and if the action levels of A = 25mm and B = 10mm were likely to be exceeded. This analysis was obviously likely to contain a number of errors based on the variability of elapsed time between telltale installation and the second readings and also the observer's potential reading error due to the small dilations being considered.

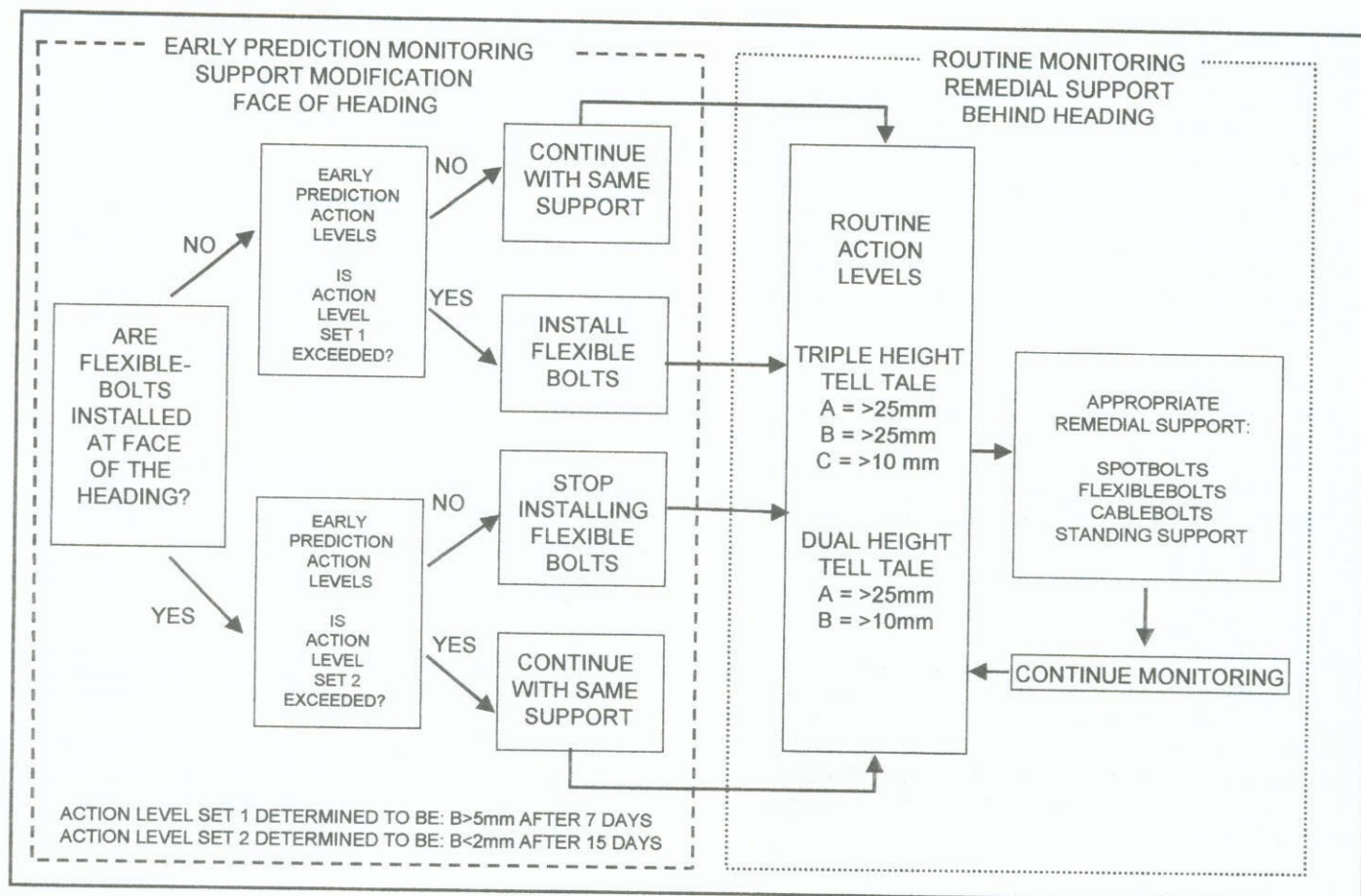


Figure 6. Flow Diagram for Proposed Support Strategy

The statistical analysis was conducted for each of the four roadways. The results for the gate road which was in closest proximity to the proposed trial gate road is shown in Table 1. These telltale results covered a distance of approximately 2800m and passed through a region of thick roof mudstones.

Table 1. Statistical Analysis to Determine the Likelihood of Telltales Exceeding or Remaining within Routine Action Levels when using the Early Action Level of B = 5 mm

No. of TT showing 5mm or more on "B" (after 6-13 days) = 18		No. of TT recording <5mm on "B" (after 6-13 days) = 144	
No. of TT which subsequently exceeded action levels (A=25mm, B=10mm, after 3 months) = 15	No. of TT which remained within action levels (A=25mm, B=10mm, after 3 months) = 3	No. of TT which subsequently exceeded action levels (A=25mm, B=10mm, after 3 months) = 16	No. of TT which remained within action levels (A=25mm, B=10mm, after 3 months) = 128
% share of sample sub set		% share of sample sub set	
83%	17%	11%	89%

Using 5mm on the B telltale as a threshold resulted in the successful prediction outcome for 143 of the 162 telltales (88%). It appeared that the long term behaviour could have been successfully predicted by assessing the early roof behaviour with a fair degree of confidence.

Similar results were found for the other three roadways and also when using a total of 10mm of telltale movement as the potential action level.

In summary, it was considered that the use of early telltale movements to assess the likely long term behaviour of a roadway was feasible and could have major cost advantages. It was however also considered that in order to determine reliable threshold criteria on which to base the assessment additional data on the early behaviour of telltales and analysis would be required.

Determination of Early Telltale Movement Action Levels to Stop Installing Additional Reinforcement at the Face of the Heading

Subsequent to the above work relating to determining action levels for installation of additional support at the face of the heading more detailed monitoring information was collected from telltales installed in two trunk roadways.



These two developments were driven in an area where the mudstone had thickened above the seam consequently generating increased roof deformation. As a result the roadways were supported with AT rockbolts supplemented by 3.5m long flexiblebolts installed at the face of the headings. The early telltale information from these roadways was also examined as a possible indicator of long term roof behaviour and consequently an indicator of action levels when the flexiblebolts were no longer required at the face of the heading.

The detailed early telltale readings were obtained from telltales which were read on a daily basis for approximately the first 2 weeks after installation. These were then related to the telltale readings taken some 8-10 months later to assess which telltales subsequently went on to exceed the action levels for the various anchor heights and the total movement.

In order to determine the level of early movement that would be appropriate to indicate the long term behaviour of a roadway which uses both rock and flexiblebolts, correlation between the long term roof movement measured and the telltale readings taken at 5, 10 and 15 days following telltale installation were examined. The analysis incorporated results from both dual and triple height telltales which are installed in flexiblebolted headings. These enable the location of the roof displacement to be monitored relative to the flexiblebolted height, A = rockbolted height, B = flexiblebolted height and C = above reinforcement height to 5m. For the analysis the B and C readings from the triple height telltales were assessed individually and also combined to give a B+C reading which is equivalent to the B reading on a dual height telltale. The results of the correlation analysis are summarised in Table 2.

The results of the correlation analysis indicated considerable variation in early roof behaviour, especially at 5 and 10 days. This results in poor correlation and a high degree of uncertainty with regard to the long term behaviour. Improved correlation was however observed for a number of combinations at 15 days.

Based on the correlation results presented in Table 2 if deformation above the AT bolts is greater than 5mm at 15 days then the movement is likely to continue to above the action level of a dual height telltale. Similarly if on a dual height telltale B = 4mm or less at 15 days then no action is needed and the roof movement is likely to stabilise below action levels.

The above data comes from an area of the mine where there is a greater than normal thickness of roof mudstone. In this case, assuming the same level of reinforcement, the variation in roof movement will reflect the behaviour of the roof mudstone and the ability of the reinforcement system to modify the response and reduce the level of roof movement in the long term. This data set does not contain examples of a flexiblebolted roof where the roof geology is considered normal. Therefore it does not contain the vital

Table 2. Correlation Analysis for Determination of Action Levels to Stop Setting Additional Support, Flexiblebolts, at the Face of the Heading

	5 days	10 days	15 days
long term A with initial A	poor correlation	improved correlation <7mm	improved correlation, if <9mm A is likely to remain below 25mm
long term B with initial B	no correlation	poor correlation	improved correlation, if <7mm B is likely to remain below 10mm
long term C with initial C	no correlation	poor correlation	improved correlation
long term B+C with initial A	no correlation	no correlation	no correlation
long term A+B+C with initial A	no correlation	no correlation	no correlation
long term A+B+C with initial B	no correlation	poor correlation	improved correlation, if <3mm total is likely to remain below 25mm
long term A+B+C with initial C	no correlation	no correlation	no correlation
long term B+C with initial B+C	no correlation	poor correlation	improved correlation, if <4mm B+C is likely to remain below 10mm
long term A+B+C with initial B+C	no correlation	poor correlation	not good correlation
long term A+B+C with initial A+B+C	no correlation	improved correlation	improved correlation, if <10mm total is likely to remain below 30mm
long term B+C with initial B	no correlation	no correlation	improved correlation, if <3mm B+C is likely to remain below 13mm

information required in order to confidently recommend levels of roof movement for dispensing with flexiblebolts. These levels will obviously be low and subject to confirmation from further analysis of appropriate data.

Roof Support Strategy

The results discussed in the two sections above have been used to develop a roof support strategy for gate roads driven in the area where the mudstone thickness locally increases.

Based on the results obtained from the four roadways driven supported using the conventional design, that is no flexiblebolts, the following early movement action level "set one" (to introduce additional support) was proposed for use with dual height telltales:

if movement above the AT bolts is 5mm or more within 7 days of telltale installation then additional movement is likely and flexiblebolt reinforcement should be introduced.

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The data in the second analysis indicated the effectiveness of the flexiblebolts, but based on the information available the expected behaviour without flexiblebolts is not known. So although the data in the second analysis did not directly indicate an appropriate early movement level to dispense with flexiblebolts, it must obviously be lower than that at which they are introduced. The following early movement action level was therefore postulated when flexiblebolts are being used in addition to the standard bolting pattern:

if movement above the AT roofbolts is 2mm or less for a telltale installed for 15 days (and the telltales between this telltale and the face of the heading also indicate 2mm or less) then the flexiblebolt reinforcement could be dispensed with. To confirm roof behaviour following a reduction in reinforcement density the roof should be closely monitored. Consideration of any favourable change in roof geology would also assist in making a decision to stop installing flexiblebolts.

This information is presented in the flow diagram as shown in Figure 6.

Limitations of the Telltale Data Used

The information analysed in both the two case studies had limitations which need consideration when establishing early action levels and should ideally be avoided to achieve a high degree of confidence.

The primary factors affecting the initial roof behaviour for the first data set were inaccuracies as a result of infrequent reading of the telltales in the first week and inconsistencies in the installed support on development as a result of local design modifications in response to roof conditions.

The primary factors affecting the initial roof behaviour in the second data set were changes in the roof horizon within the thickened mudstone resulting from attempts to place more competent strata within the bolted horizon, interaction effects from extraction of the seam above and the development sequence of the two trunk roadways.

The flow diagram, shown in Figure 6, is considered to be suitable in principle. However, to increase confidence levels on the threshold movements it is considered that more detailed information should be obtained to: a) define the early movement levels for installing flexiblebolts for time periods shorter than 7 days, and b), define movement levels for when to stop installing flexiblebolts. Although the above data has been used to indicate such action levels the collation and analysis of more detailed information would allow the levels to be refined and determined with a greater degree of confidence.

Appropriate data could be obtained by either intensive manual reading of telltales or by the use of a remote reading system. In terms of accuracy and frequency of readings remote reading appears to offer the better alternative.

REMOTE READING TELLTALE SYSTEM

As well as the dual height telltale, RMT Ltd. manufacture a number of dual height telltale hybrids in order to suit special circumstances.[†] These include the triple height telltale and electrical versions. Also an intrinsically safe mine wide remote reading system for telltales is undergoing field demonstration in the UK. It allows up to 400 telltales to be read by a surface PC. It utilises low cost reliable transponders, with a high fault tolerance, that can be installed and connected by the heading team and still retain their visual readability. It employs an innovative yet simple communications technology similar to that used in the new m-Comm mines rescue communications system. The system is currently undergoing MSHA approval for the USA.

The use of the remote reading telltale system in relation to the above strategy would allow:

- (i) reading the telltales to sub 1 millimetre accuracy,
- (ii) reading and logging of movements on a more regular basis, up to hourly reading of each telltale, rates of movement to be accurately assessed, (rates of movement may provide a better guide to future movement or supplement the absolute movement and hence provide a more accurate predictive tool),
- (iv) allow action levels to be programmed to provide audible/visual warnings of ground movement and recommended remedial actions.

CONCLUSIONS

Rockbolt support in UK coal mines is installed as part of an observational design methodology. All reinforced roadways are routinely monitored using dual height telltales and action levels defined in order to indicate the onset of instability and the need for remedial support.

The action levels set are site specific, and will be based on analysis of monitoring information to determine the roadways deformation characteristics. The action taken is usually in the form of additional reinforcement installed outbye. Although this system generally works well it can disrupt normal heading operations.

An alternative approach involves reacting to early roof movement as measured by the routine monitoring telltales and modifying the support installed at the face of the development accordingly. In modifying the support installed at the face improved roof control is anticipated and it is envisaged that ground which would otherwise have behaved in a unacceptable manner with the basic support design would remain within acceptable movement limits with the modified support pattern.

To utilise successfully early roof movements as part of a support strategy, definition of the appropriate action levels is required. These early telltale action levels could then be used to modify the installed support at the face of the



heading to suit the prevailing conditions. Two sets of action levels are needed. Set 'one' to introduce additional support such as flexiblebolts, set 'two' to stop setting additional support. This design modification method is therefore based on the assumption that the early movements on telltales at the face of the heading for a given support design are able to indicate the long term stability of the roadway.

Results from a study site to determine the use of early action levels showed that early roof movement could be used to indicate the long term stability of the roadway roof. Consequently it was considered feasible to set an action level which if exceeded within the first week of driveage indicated the need for additional support at the face of the heading.

The subsequent analysis to determine an action level whereby such additional support was no longer required at the face of the heading was also seen as feasible. However the time period over which the analysis was made needed to be at least 15 days.

The results were able to define a reinforcement strategy which was sound in principal. It was able to indicate when additional reinforcement was required at the face of the heading and when it could be dispensed with. However to increase confidence in the levels more detailed information would be required for analysis.

The more detailed information from the telltales could be gained from either intensive reading of the manual telltales or from the use of remote reading telltales. In terms of accuracy and frequency of readings the remote reading system would offer a better alternative. It would allow accurate reading to sub-one millimetre accuracy, read and log movements on a regular basis and allow rates of movement to be assessed. Rates of movement, alongside the absolute movement, could provide a more accurate predictive tool. A dual height remote reading telltale system is undergoing field demonstration in the UK and has been submitted for USA MSHA approval.

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Skin Failure of Roof and Rib in Underground Coal Mines

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ABSTRACT

Skin failures of roof and rib in underground coal mines continue to be a significant safety hazard for mine workers. Skin failures do not usually involve failure of the support systems but result from rock or coal spalling from between the support elements. For instance in 1997, over 800 miners were injured by roof and rib falls, of which 98 pct were the result of skin failures. Also, nearly 80 pct of the roof and rib failure injuries occurred at or near the working faces in development sections. The face area is a zone where the potential for skin failure accidents and injuries and roof and rib failures, in general, is high because of mining activity, ground readjustment due to changing stress conditions, and the higher exposure of mine workers. In addition, failures occur where the roof and rib are unsupported. This paper features a review of the roof and rib accident statistics resulting from skin failure, highlighting the incidences by average days lost, in-mine location, state, MSHA District, and worker activity at the time of injury. A detailed analysis of recent fatalities caused by skin failure is included. Also discussed are the causes of roof and rib skin failures, current methods and support materials for skin surface control, as well as a historical literature review of skin failures and control methods.

INTRODUCTION

Falls of roof and rib have traditionally been one of the leading causes of mine worker injuries and fatalities in underground coal mines. From 1993 through 1997, nearly 35 pct of all reported underground incidents resulted from falls of roof and rib. These falls of roof and rib resulted in almost 4,800 injuries, or over 15 pct of the total reported underground injuries. Also, skin failures, the failure of small blocks or slabs of roof and rib, have been a recognized problem in the coal mining industry for many years. Detailed analyses showed that in 1997 alone, approximately 98 pct of the roof and rib injuries were from skin failures. This suggests that as many as 4,700 injuries may have

resulted from skin failures of the roof and rib during this five-year period.

Reference to skin failures are found in the literature as far back as the late 1920's. Most of the early references discussed the effect of moisture and humidity on roof failures (1, 2). Other authors addressed ways to condition mine air such as water sprays and tempering entries to prevent roof deterioration (3, 4). Considerable work was presented on the effectiveness of various sealants to coat mine strata including coal tar (5), Ebonol (6), asphalt-based paints (7), sulfur based coating materials (8), cement and cement mixtures (9), shotcrete (10) and polymeric sealants (11). More recently, researchers have investigated the mechanisms of shale roof rock deterioration due to atmospheric moisture, explaining it seems to be a result of stresses from moisture-induced weakening and swelling strain, rather than slaking (12, 13). Finally, Merrill and Stateham (14) explained how loose roof and rib can be detected using infrared devices.

Although the above references dealt mostly with roof skin failure, rib skin failure has also received attention by the coal mining industry. The theory and practices regarding rib failure, especially in thick coal seams, was addressed by Smith (15) who suggested that fracturing begins at a stress level equivalent to one third to two thirds of the ultimate strength of the material. Peng (16), Dolinar and Tadolini (17), and Dolinar (18) discussed general coal rib stabilization and the effectiveness of wood dowels, resin bolts, and straps to provide pillar reinforcement. Martin, Carr and Hendon (19) provided information that demonstrated the superior performance of yieldable rib bolts to stabilize ribs when twin seam mining at Jim Walter Resources. Wykoff (20) and Horino, Duvall and Brady (21) investigated the use of wire rope to wrap pillars. Their research indicated that wire rope can significantly effect the compressive strength, and stability of pillars. In addition, many of the references on mine sealants mentioned the use of these for coating and sealing coal ribs.

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Many advances have been made in dealing with roof and rib failures, but unfortunately, the problems have not been eliminated. Continued research by government, academia, labor, and the mining industry is needed to address roof and rib skin failures and minimize the associated mine worker injuries. Research at the National Institute for Occupational Safety and Health (NIOSH) is continuing this effort by investigating the causes of skin failure and evaluating control techniques.

DESCRIPTION OF SKIN FAILURE

For the purposes of this manuscript and analyses, skin failure is defined as "the progressive loosening of slabs, blocks, or layers of rock from the surface of the mine roof or the spalling of pieces or slabs of coal from the coal pillar." Skin failure involves smaller pieces of rock or coal, rather than massive roof failures (above anchorage) or coal pillar failures (bumps and bursts). Skin failures can occur in both supported and unsupported mine strata. Figure 1 illustrates skin failure of unsupported mine roof while figure 2 is an example of skin failure of supported (bolted) mine roof where the failure occurs between the supports. Rib skin failures are illustrated in figure 3 (unsupported rib) and figure 4 (supported rib).

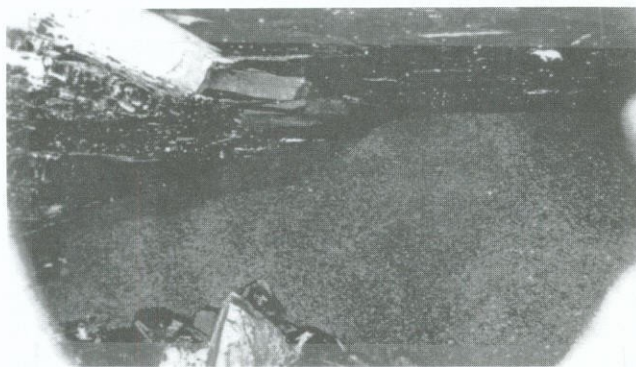


Figure 1. Skin failure of unsupported roof.

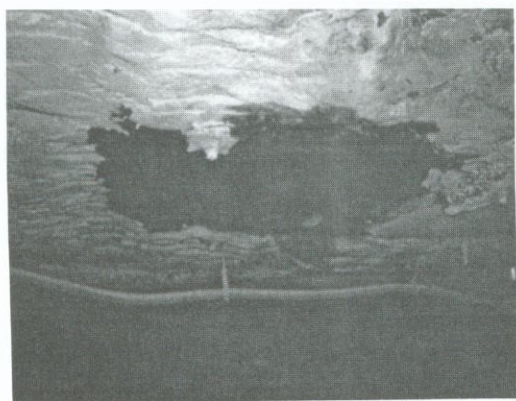


Figure 2. Skin failure in supported roof.

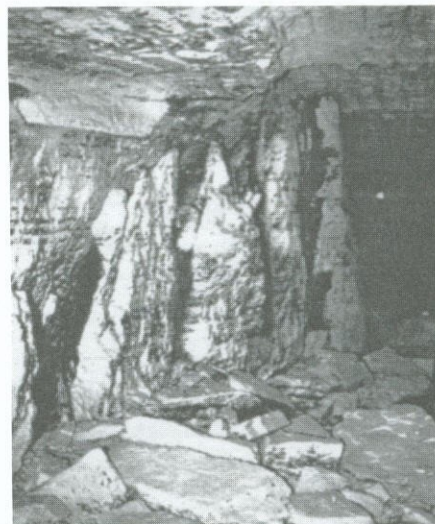


Figure 3. Skin failure of unsupported rib.



Figure 4. Skin failure of supported rib.

The mechanisms responsible for skin failures can vary considerably, the most common factors being competence of the strata and presence of geologic discontinuities. In many mines the roof is composed of draw rock (slate and shale), coal, bony material, and other highly stratified, thinly laminated strata. This strata is susceptible to progressively failing in thin layers because of bedding plane weaknesses. Some of the causes of bedding plane failures which result in skin failures include sag of the strata under its own weight, overburden pressure as depth increases, horizontal stress, and moisture or temperature sensitivity. If geologic discontinuities are present, the likelihood of skin failure increases because the discontinuities weaken and compromise the structural integrity of the rock. It would seem logical that the potential for roof to experience skin failure can

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be estimated using the Coal Mine Roof Rating (CMRR). CMRR estimates the structural competence of coal mine roof and considers bedding most important. It includes the factors that weaken bedded coal-measure rocks such as discontinuities, moisture, and rock strength (22). The lower the CMRR, the less competent is the roof and the more susceptible to skin failure.

Coal ribs can experience skin failure for many of the same reasons. Primarily, rib skin failure is associated with the early stages of the failure of insufficiently sized pillars or the effects of depth. A general observation about rib skin failures is that rib spalling tends to increase as mining height increases. A plot of rib skin failure injuries and seam height for 1997 indicated that the injury rate increased as the seam height increased, up to 8-ft-thick (2.4-m-thick) (fig. 5). For seam heights above 8 ft (2.4 m), no specific trend was evident, possibly because of insufficient data and the probability that more rib support is used in the thicker seams. Rib spalling may also increase as the depth increases, it can be affected by face cleat, and it can be self-perpetuating. As coal breaks from the solid pillar, pillar dimensions decrease and pillar pressures increase. The result is more fracturing and spalling, creating a cycle of spalling and loading that may continue unless control measures are implemented. Rib skin failure is also frequently associated with rock partings within the coal pillar or with draw rock located at the roof/rib interface. Rock partings or bands within the pillar create planes of weakness where differential movement (fig. 6) and failure can occur, leading to spalling (skin failure). When weak draw rock that is subject to failure during coal extraction is present, and is mined with the coal, the draw rock exposed in the coal pillars creates a zone of potential rib skin failure. This inherent weakness makes the draw rock susceptible to spalling from the rib as the coal pillars experience load.

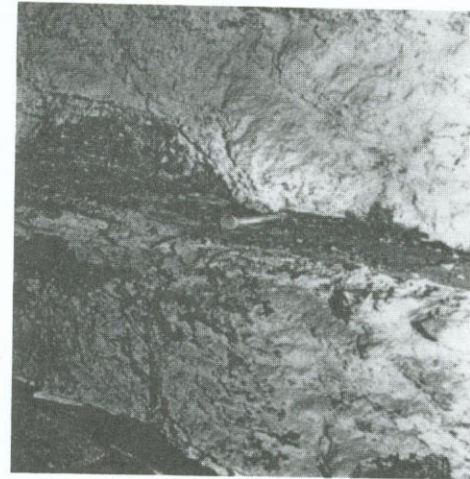


Figure 6. Example of differential movement along parting in coal rib.

SKIN FAILURE INCIDENT ANALYSIS

Two separate incident analyses were conducted, one addressing roof and rib fall fatalities from 1993 through 1997, and another addressing all reported roof and rib fall incidents for 1997. These analyses were designed to identify the fatalities and injuries resulting from skin failures, both roof and rib, and the massive failures, then draw some statistically based conclusions.

Roof and Rib Fall Fatalities

The underground coal mine fatalities caused by falls of roof and rib for the years 1993 through 1997 were separated between skin and massive failures. The criteria for classifying a fall as massive or skin were the thickness and areal extent of the fallen material. In general, roof falls with a thickness less than the bolt length were classified as skin failures (nearly all roof skin failures were less than 2-ft-thick (0.6-m-thick) while the rib skin failures were all less than 3.5-ft-thick (1.1-m-thick)). The assumption is that these falls could possibly have been prevented if a more effective support system had been used, the Automated Temporary Roof Support (ATRS) system and bolting station canopy provided more surface control, and if scaling of the roof and rib were practiced more frequently. Nearly all rib failures were classified as skin failures except those listed in the Mine Safety and Health Administration (MSHA) Fatality Report as being an outburst or bump. Table 1 summarizes the initial classification and illustrates that slightly more fatalities resulted from massive failures. It should also be noted that the massive failures occurred nearly evenly between advance mining (15 fatalities) and retreat mining (12 fatalities).

Next, a comparison of the skin failures was conducted. Table 2 illustrates that nearly twice the number of fatalities resulted from roof skin failures than rib skin failures. In addition, most of the roof skin failures occurred under

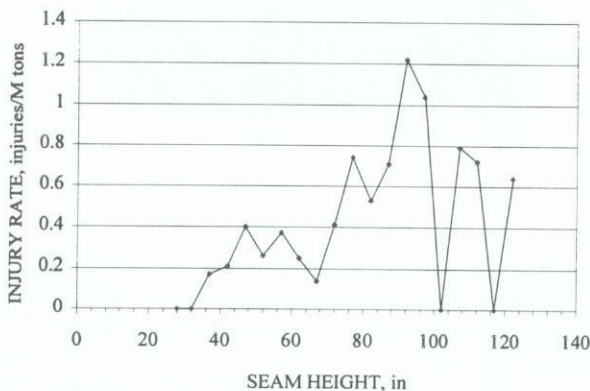


Figure 5. Seam height versus injury rate for rib skin failures, 1997.

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unsupported roof while the victim was in by permanent supports. The fatalities resulted from improper worker behavior (going in by supports). Additionally, the average thickness of the roof skin failures and rib skin failures was calculated. The rib skin failures were thicker (mean = 2.55 ft (0.78 m), std. dev. = 1.03 ft (0.31 m)) than the roof skin failures (mean = 0.88 ft (0.27 m), std. dev. = 0.70 ft (0.21 m)). For comparison, the massive failures on average were thicker (mean = 9.1 ft (2.8 m), std. dev. = 7.5 ft (2.3 m)).

Table 1. Roof and rib fatalities, 1993-1997.

Year	Total fatalities	Massive failures	Skin failures
1993	12	7	5
1994	8	4	4
1995	8	6	2
1996	13	4	9
1997	9	6	3
Total	50	27	23

Table 2. Comparison of fatalities from roof and rib skin failures.

Year	Rib skin fatalities	Roof skin fatalities	
		Supported roof	Unsupported roof
1993	1	0	4
1994	1	0	3
1995	0	0	2
1996	3	1	5
1997	3	0	0
Total	8	1	14

Finally, an attempt was made to correlate the tendencies for skin failures to occur within specific states or MSHA Districts. As shown in tables 3 and 4, the only trend is that the states and districts with the most underground mines experienced the most roof and rib fatalities.

Table 3. Roof and rib fatalities by state, 1993-1997.

State	Total fatalities	Massive failures	Skin failures	
			Roof	Rib
AL	1	0	0	1
CO	1	1	0	0
IL	2	1	0	1
KY	15	7	6	2
PA	7	4	1	2
TN	3	3	0	0
UT	1	1	0	0
VA	8	5	3	0
WV	11	5	4	2
WY	1	0	1	0
Total	50	27	15	8

Table 4. Roof and rib fatalities by MSHA District, 1993-1997.

MSHA District	Total fatalities	Massive failures	Skin failures	
			Roof	Rib
1	4	3	1	0
2	3	1	0	2
3	1	0	0	1
4	10	5	4	1
5	8	5	3	0
6	6	2	4	0
7	10	6	2	2
8	2	1	0	1
9	3	2	1	0
10	2	2	0	0
11	1	0	0	1
Total	50	27	15	8

Reported Roof and Rib Fall Injuries

To delineate the extent of the worker injuries resulting from skin failures, the MSHA Health and Safety Analysis Center (HSAC) Accident Database was examined for 1997. All injuries occurring in underground coal mines resulting from roof and rib failures were extracted and analyzed. This included degree of injury classes from one to six which were injuries ranging from no lost time or restricted activity to those that resulted in a fatality, but did not include reportable roof falls that occurred

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when no workers were present. In addition, the accident injury illness types extracted were fall of face, rib, or side, fall of roof, and machinery. The machinery incidents were included because some of these occurrences were actually the result of falling roof or rib. These misclassified incidents were sorted out by using the source of injury code with a criteria of caving of rock, coal, ore, and waste. This query eliminated the correctly classified machinery incidents. Table 5 lists a summary of the roof and rib injuries for 1997. It reveals that most of the injuries resulted from roof skin failures, but that the severity of the rib skin injuries was greater because the average days lost was higher. Finally, the table reveals that 570 mines did not report any roof or rib injuries in 1997.

Table 5. Number of injuries from roof and rib failures, 1997.

Failure type	No. of injuries	Pct. of injuries	Avg. days lost per injury	No. of mines ¹
None	0	0	0	570
Roof skin	669	83	27	266
Rib skin	128	15	190 ²	83
Massive	13	2	2,783 ³	6
Total	810	100	---	925

¹No. of mines reporting at least one injury of the failure type listed.

²Avg. includes 3 fatalities (18,000 days away), avg. without fatals is 43.

³Avg. includes 6 fatalities (36,000 days away), avg. without fatals is 25.

Next, another analysis determined the mining situations in which roof and rib skin injuries occurred. Table 6 indicates that 84 pct of the skin failure injuries occurred during development or retreat mining, the remaining 16 pct divided among longwall and other. An attempt was made to determine the location of skin failure injuries with respect to the state of roof support. The best estimate is that 383 of the 669 roof skin injuries occurred under permanent support, or 57 pct. It is possible that many of the roof skin failure injuries occurring where the roof was permanently supported could have been prevented through modified support designs. Another 233 (35 pct) roof skin injuries occurred under temporarily or unsupported roof. In addition, increasing the skin coverage of the ATRS and drill station canopies could help reduce the roof skin failure injuries occurring under temporarily supported roof. For the remaining 53 roof skin injuries, the state of support was unknown. Approximately 85 of the 128 rib skin injuries occurred where the roof was permanently supported, or 66 pct. The rib skin failure injuries occurring under permanently supported roof may be minimized by securing the ribs if necessary, or through scaling and increased awareness of rib conditions. Another 19 (15 pct) rib skin injuries occurred under temporarily or unsupported

roof, while the remaining 24 (19 pct) occurred where the state of the support was unknown.

Table 6. Roof and rib skin failure injuries classified by mining situation, 1997.

Mining situation	Roof skin failures		Rib skin failures	
	Injuries	Percent	Injuries	Percent
Development ¹	560	84	108	84
Longwall ²	38	6	11	9
Other ³	71	10	9	7
Total	669	100	128	100

¹Includes advance and retreat mining.

²Includes injuries occurring in the headgate and tailgate during panel mining.

³Includes injuries outby face and unknown origin.

Finally, the mine worker activities during roof and rib skin injuries was extracted from the database. The most common activities of workers injured by roof skin failures were drilling or bolting of the roof (44 pct), operating the continuous mining machine (CM) (10 pct), and general inside labor (8 pct). These three activities accounted for 62 pct of the injuries. No other worker activity was involved in more than 7 pct of the injuries. For injuries resulting from rib skin failures, the most common worker activities were drilling or bolting the roof (19 pct), operating the CM (17 pct), walking (12 pct), moving power cables (11 pct), general inside labor (11 pct), and maintenance and repair (9 pct), which in total accounted for 79 pct of the injuries. All other activities were involved in 7 pct or less of the rib skin injuries. Surprisingly, scaling of the roof or rib, which deals directly with skin failure and is thought to be a dangerous activity, comprised only 1 pct of the roof and rib skin failure injuries.

SKIN CONTROL METHODS

The review of skin control methods includes both a historical look at what has been done in the past as well as a description of current control techniques. The investigation reveals that many of the same methods are in use today as in the past.

Early Skin Control Efforts

Referring back to the literature review presented in the introduction, past control methods were directed primarily at preventing skin failures resulting from changes in temperature and humidity. These included various coating materials designed simply to seal the surface without providing additional strength or reinforcement, and attempts at conditioning the mine air prior to introduction into the mine workings. The air conditioning involved regulating the temperature and humidity

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to near ambient mine conditions to prevent failures due to expansion and contraction, and moisture variations. In the face area, past attempts at controlling roof and rib skin failure using artificial means were reflective of the support materials available. Mechanical bolts in combination with wood headers and planks, oversized plates, wire mesh, old hoist rope, wood dowels, and other simple support methods were commonly employed.

Current Skin Control Methods

Current control methods have built on the successes of the past techniques, employing the more sophisticated support materials currently in use. In addition, more thought has been given to matching the type of control to the failure mode. For instance because the mining industry has an improved understanding of the mechanism of strata failure, cement coatings utilizing steel or glass fibers are available to not only seal the strata, but also to add strength to resist failure. For control of roof skin failure, wood planks, steel straps and channel, and various meshes such as welded wire, chain link, or synthetic grid material are being used.

Rib support methods have changed as well, primarily in the use, type, and location of bolts. The emphasis is to match the deformability of the rib supports to that of the rib. Yieldable bolts as used at Jim Walters Resources (19), can stabilize the coal seam and ribs effectively by controlling displacements to reduce stress buildup.

A recent information request from MSHA District 3 revealed the following examples of roof skin control methods: (1) A mine using screens in one intake, one return, and the track entry, plus using a lot of gunite; (2) Another mine using 8 gauge steel, 5 by 16 ft (1.5 by 4.9 m) sections of "monster mats" installed on cycle; (3) A longwall mine required to use screening or spray where they have trouble holding up head coal in their gate roads; and (4) A mine that has a history of falls due to deteriorating top has miles of gunited track entry. Information obtained from MSHA District 4 revealed some additional skin control methods. These included using oversized bearing plates on pattern bolts, installing 2-ft-long (0.6-m-long) "bacon skins" (straps) with 3-ft-long (0.9-m-long) mechanical anchor bolts in between the pattern bolts, or covering the roof with synthetic grid material when roof skin failure is a problem. For rib skin failures at the face, some mines install 4- to 6-ft-long (1.2- to 1.8-m-long) planks with 18 or 36 in (45.7 or 91.4 cm) bolts. When sporadic rib failures occur outby the face area, mines mainly use timbers set close to the ribs to minimize the dangers to mine workers traveling nearby.

Typical rib skin control methods, taken from a roof control plan from a mine in Pennsylvania, includes the following: "Loose ribs are to be blocked, bolted, or taken down. Steel straps, planks, or header blocks with 4- to 6-ft-long (1.2- to 1.8-m-long) bolts may be used. Bolts are not to exceed 8-ft (2.4-m) intervals. In lieu of the above, such ribs may be supported by posts or cribs installed tightly near the rib."

SUMMARY AND CONCLUSIONS

Skin failure of roof and ribs injures many workers in underground coal mines. Statistics from 1997 indicate 98 pct of the injuries from roof and rib falls are due to skin failures, resulting in over 800 injuries, or approximately 15 pct of all underground coal mine injuries.

Skin failures, defined as the failure of small pieces of rock and coal, rather than massive roof falls or coal pillar failures, occur in both supported and unsupported roof and rib. It is possible that many of the injuries can be avoided by using improved methods of support and by limiting the need for mine workers to travel inby permanent supports to set temporary supports prior to installing permanent supports.

An analysis of roof and rib skin failure injuries revealed that far more injuries resulted from roof skin failures, but the rib skin failures caused more severe injuries. The analysis also revealed that roof and rib skin failures were three times more likely to result in an injury when they occurred on the working section than outby in other mine areas, simply because of greater worker activity in the face area.

The methods for support of roof and ribs to prevent skin failure are merely extensions of standard roof support methods. As dictated by the extent of skin failure problems, the on-cycle supporting methods are modified to provide additional surface coverage. Common skin control methods include oversized plates, header boards, wood planks, steel straps, meshing, and in rare instances, spray coatings. These control methods can control skin failures, unfortunately they are implemented reactively to control problems that are occurring rather than proactively to prevent future skin failure occurrences. The success of these controls can be enhanced by matching the characteristics of the support to the expected strata reactions to mining and modes of failure.

NIOSH research is continuing to address the causes and control of skin failure in underground coal mines. Emphasis will be to determine the mine, roof, and rib conditions most likely to result in skin failures then verify the most effective control techniques.

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Application of Polyurethane Injection for Rehabilitation of the Maingate of a Deep Single Entry Longwall Mine

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ABSTRACT

After 34km of roof bolted drivage the first fall of ground occurred at the Phalen Colliery in the maingate of the 8th East longwall panel. The fall occurred over the stage loader as the wall approached an area of poor roof geology. This area had developed a severe cutter since initial drivage and the longwall had yet to mine through 500m of even worse conditions. This paper describes the cause of the poor roof conditions, the use of polyurethane resin injection and cable bolting to re-stabilize the roof outby the fall, and the measurement of roof movement and performance subsequent to the continuation of mining under the rehabilitated roof.

INTRODUCTION

Phalen Colliery is located on the east shore of Cape Breton Island, on Canada's east coast. It is one of 2 submarine coal mines operated by the Cape Breton Development Corporation. The Sydney Coalfield consists of eleven seams ranging in thickness from 0.8m to 3.7m. The Phalen Colliery operates solely in the Phalen Seam which is comprised of coal ranging in rank from a high volatile sub-bituminous A to a bituminous A. Seam thickness ranges between 1.4m and 3.0m, the average being 2.5m. The seam outcrops on the shore and dips northward under the Atlantic Ocean at an average grade of 20%.

The Phalen Colliery utilizes the panel and pillar retreat longwall method, with longwall access via single entry gateroads (Figure 1). The gateroads are developed approximately along strike from the main access slopes. Until 1991, gateroad development was achieved by roadheader type machines, with passive steel sets installed for support. Relatively low driveage rates combined with deteriorating gate end conditions as the mine became deeper, led to a decision to change over to continuous miner driveage with roof bolting as the primary method of support (MacDonald et. al., 1992).

Trials and a phased introduction of roof bolting in the maingate of the 5 East Panel led to the adoption of a system which was based on experience in the United Kingdom and Australia (MacDonald et. al., 1994). The bolting pattern employed 7 torque-tension bolts (per meter advance) in a 4.8 meter wide roadway installed with air operated portable bolters. The bolts were 2.4m in length, Grade 75, .804 J Bar, fully encapsulated in a fast and slow set resin.

A total of 34 km of roadway have been driven using roof bolts as the primary method of support, with 6 longwall faces successfully retreated. All but 3 km of roadways employed continuous miners and air bolters. The exception being the 8 East maingate where a Voest Alpine ABM20 with 6 on-board hydraulic bolters provided the drivage and roof bolting requirements.

The use of single entry gateroads made it necessary to employ a relatively high support density, as it was crucial to minimize the risk of a roof fall. However, on Sept. 24, 1998, the first fall of ground (in a bolted roadway) occurred in the 8 East maingate. The events leading up to the fall, the probable causes of the fall, and the subsequent course of action for rehabilitation and remedial support are detailed in this paper. As well, results of the monitoring done during gateroad development, rehabilitation and subsequent longwall retreat are presented.

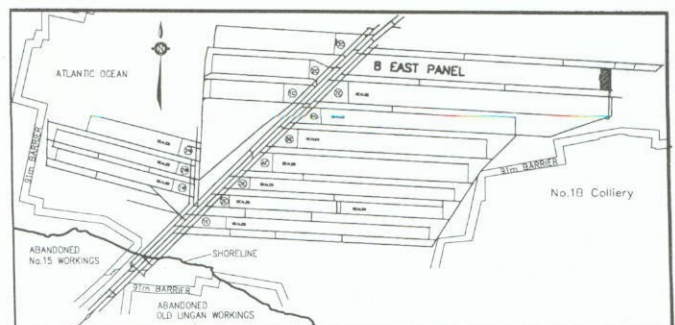


Figure 1. Mine Plan of Phalen Colliery.

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GATEROAD PARAMETERS

Drivage

The 8 East maingate was developed as a single entry, full face (4.9m (16') wide) drivage using a Voest Alpine Bolter/Miner (ABM 20). The gateroad was driven on a slight downhill grade (2%) and had a cross grade ranging from 12% outby to 21% inby. The ABM 20 was slightly oversized for the seam height and therefore some immediate roof had to be cut. The ABM 20 was also unable to maintain position on the full cross grade and therefore bottom was cut on the up dip side of the roadway to level the floor. As it is a full face machine, the ABM 20 was then forced to cut more roof on the down dip side than the up dip side.

Up dip side rib spall was difficult to control resulting in an average actual roadway width of about 5.5m (18') with some areas as wide as 6.7m (22').

Specific Geologic Setting

The immediate roof of the Phalen Colliery has been divided into 3 facies.

1. The Channel Facies (CF), which is comprised of multi storied sandstone beds up to 10m thick, formed by a river channel which eroded into the other facies. The RMR of this facies is about 75.
2. The Flood Plain Facies (FPF), which is comprised of interbedded muds, silts and sands with an average RMR of 45.
3. The Backswamp Facies (BSF), which consists of a basal seat earth, overlain by a thin (less than 15cm) high ash coal seam. The coal seam is draped by a high carbonaceous limestone. The RMR of this facies ranged from 15 - 25.

The area of the roof fall was in the transition zone between the Backswamp and Flood Plain Facies but was very close to full Flood Plain. The RMR was estimated to be 45 even though the roof still had a few characteristics of the Backswamp facies.

The area outby the roof fall was the full Backswamp Facies with extremely high prior roof movement and a very low RMR.

Support

The ABM 20 had 6 hydraulic bolters on board, 4 roof and 2 rib. The primary bolting pattern called for 7 full column resin roof bolts, 8' long, 7/8" diameter, grade 75 J-bar, to be installed across the (4.9m) 16' wide roadway with a 1m spacing between rows. The bolts were installed with two resin speeds to allow tension to be applied quickly to the lower 6' of the bolts. Fourteen gauge metal straps (mats) and plastic mesh was also installed on the roof. Operationally the bolters could not install all 7 of the roof bolts vertically. The pattern is

shown in figure 2. The up dip rib was fibreglass rib bolted with plastic mesh and the low side rib was steel rib bolted with wood headers. All rib bolts were 6' long with full resin encapsulation.

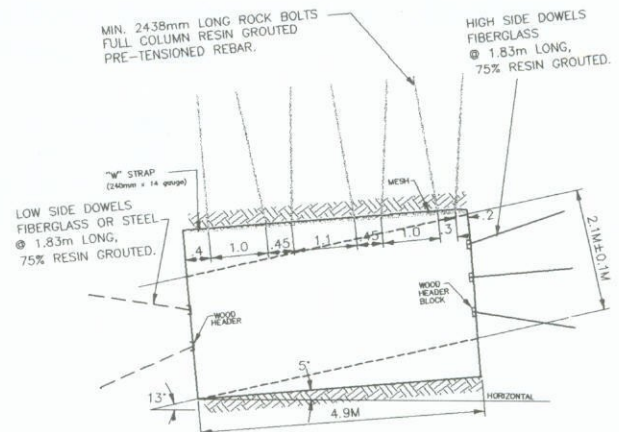


Figure 2. Primary Support in 8 East Maingate.

A program of roof monitoring was in place along 8 East maingate to assess roof stability. Results of this monitoring led to the installation of significant levels of secondary support.

In the area of the roof fall, solid bar trusses were installed at the face during primary bolting. In addition, center props were installed behind the miner on a 1m spacing. In the area of the Backswamp, the primary bolting density had been increased by shortening the row spacing to 0.7m. All of the following secondary support systems were installed at different times in different areas. Center cribs, center props (wood, yielding steel posts, yielding wood props, yielding steel legs), solid bar trusses, cable trusses, cement grouted cable bolts, resin grouted cable bolts and spot bolts.

Stress Environment

The direction and position of the 8 East Maingate made it prone to the following stress conditions.

The depth of cover (635m outby to 686m inby) is considered relatively deep by North American standards and therefore experienced the corresponding level of vertical stress. In addition, the roadway was driven under pillar edges of longwalls in the Harbour seam situated 138m (450') above the Phalen seam (Figure 3). Although this interburden thickness is considered the outside limit for interaction effects, areas of elevated stress have been routinely observed in the Phalen Colliery under pillar edges of the Harbour seam throughout the life of the mine.

Having 40m (131') of Atlantic Ocean as the top portion of the overburden is not believed to create an anomalous stress condition in the mine, although slight changes in pillar stress have been measured directly associated with tidal changes (CANMET, 1996).

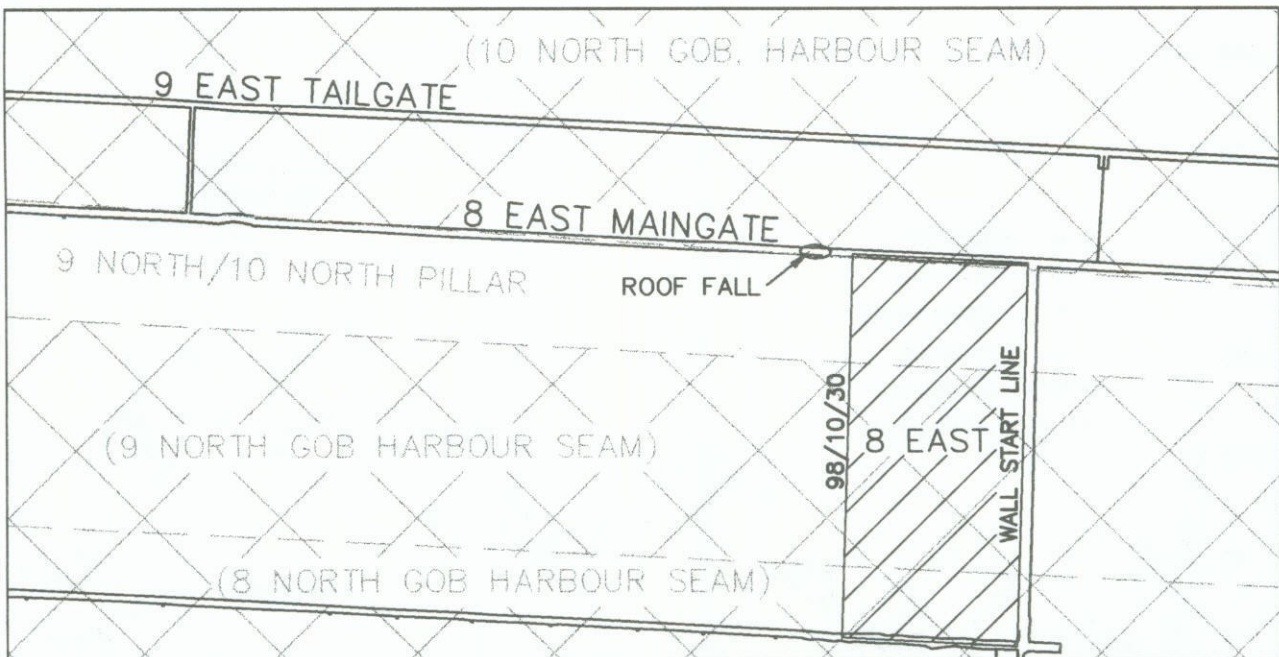


Figure 3. Partial Phalen Mine Plan with Harbour Seam Workings Overlay Showing Position of 8 East Maingate Relative to Overlying Pillar Edge.

The Sydney Coalfield has long experienced the effects of high horizontal stress (Payne, 1998). The direction of principal horizontal stress is observed to be $N60^{\circ}E$. The 8 East Maingate was driven at $S87^{\circ}E$, therefore the angle to the principal horizontal stress is 33° . A cutter often develops along the low side of the roadways, depending on immediate roof geology (Figure 4). The principal horizontal stress subsequently becomes elevated at the maingate end during longwall retreat as this corner leads into the horizontal stress field.



Figure 4. Cutter Roof Along Low Side Rib.

Time

Due to operational problems with the 2 longwalls mined prior to the 8 East longwall almost 3 years passed between the time of initial development of the 8 East maingate and the retreat of the longwall.

Rates of roof sag during development at the Phalen Colliery are dependent on geology and direction of the gateroad relative to the principal horizontal stress. In general, gateroads in the Channel Facies experience little to no roof movement, in the Flood Plain about 1mm/month and the Backswamp Facies can be 6-10 mm/month without any longwall abutment effect.

Time caused gradual deterioration as the stress field continued to act on the roof beam. The effect of creep failure (failure at lower stress due to time) added to the deterioration. In addition, with the development of a cutter and the exposure of roof strata to the mine air for 3 years, including the changing of seasons, humidity, temperature and ventilation circuits, there may have been some deterioration as a result.

A program of routine roof monitoring was in place in the 8 East maingate prior to the fall of ground. Mechanical sag stations were installed to depths of 8' and 16' on a minimum spacing of 20m. Figure 5 is a graph of the total roof movement along the entire 8 East maingate prior to longwall retreat. It demonstrates the large amount of roof movement in the area of the Backswamp Facies. As a result a significant program of roof rehabilitation (cable trussing) was undertaken to re-support the area of the Backswamp facies prior to wall retreat. Unfortunately the fall of ground occurred just inby the area being rehabilitated. The figure also shows the relative location of the fall and the area which was eventually cable bolted and

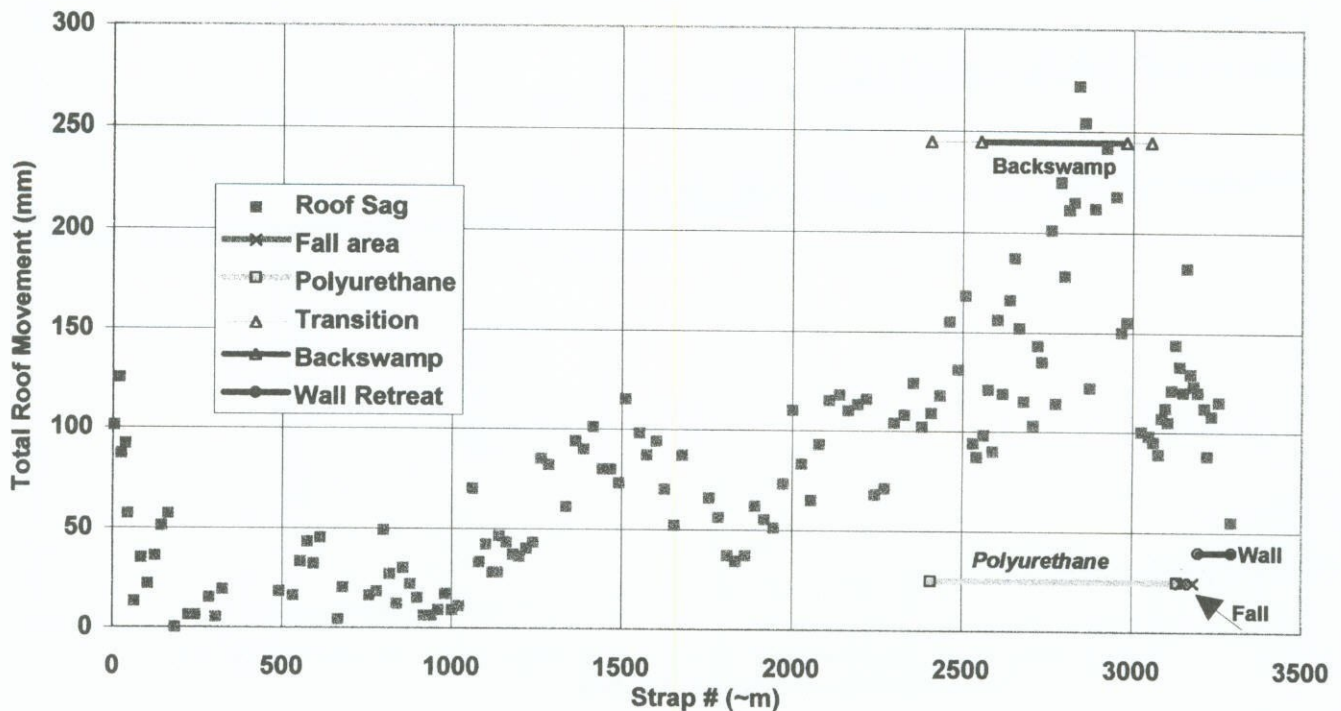


Figure 5. Total Roof Movement as Measured Along 8 East Maingate Prior to Wall Startup (Wall Position, Roof Fall, and Polyurethane Superimposed).

grouted with polyurethane.

In addition, several 7.6m (25') multi point (sonic) extensometers were installed along the maingate during development. Figure 6 shows the results of a 25' multi point extensometer installed during development near the area of the roof fall. It demonstrates the total amount of roof movement (approximately 100mm) and the location and magnitude of the separations in the roof after (within 6 months of) initial driveage. After this much roof movement the instrumentation hole sheared off and readings were no longer obtainable.

ROOF FALL

A fall of ground occurred after the 8 East longwall had retreated a distance of only 115 meters from the start line. The fall initiated approximately 15m from the face, and propagated 20m outby. The height of the fall ranged between 4-5m, while the width ranged between 6-6.5m. The down dip edge of the fall cavity (along the cutter) exhibited a relatively smooth surface, angling approximately 20° off vertical, while the up-dip perimeter was more jagged with individual layers sliding into the void. Roof lithology in the fall area was primarily thinly laminated siltstones, which overall could be described as poor with regard to quality of the rock mass. The majority of the broken rock mass fell at once, with progressive failure continuing over several days. The entry width in the fall area was in excess of 6m, as compared to the designed width of 4.9m.

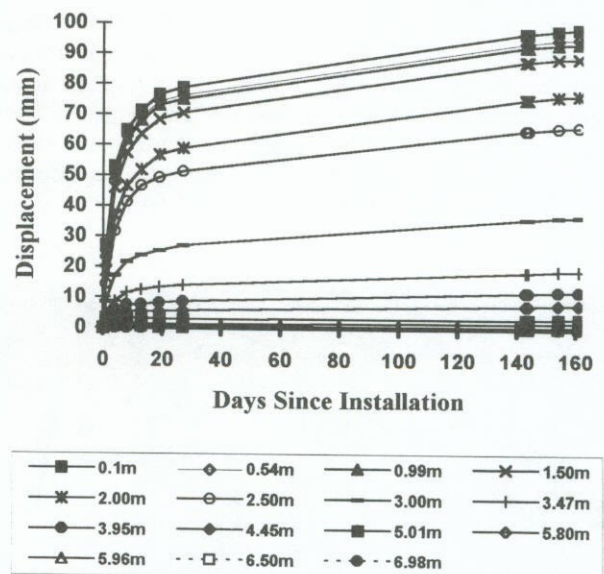


Figure 6 - Displacement Profile
Multipoint (sonic) Extensometer

Failure Sequence (NSA Eng., 1998)

In-mine observations conducted outby the fall area in the 8 East maingate, along with sites in the 8 East and 9 East tailgates led to the formulation of the following explanation of the failure sequence.



1. Minor cutter type failure began to occur along the low side (down dip) rib following development. Over time, the failure migrated upward along the rib line.
2. Vertical loading increased the effective roadway width by up to 2m through immediate rib spall. The increased entry width facilitated increasing beam deflection and bed separation.
3. The cross measure cutting of immediate roof by the ABM20, coupled with the long term standing time, promoted slaking.
4. The effects of front abutment (both vertical and horizontal) in the area accelerated roof deterioration.
5. Large vertical deformations along the down dip side (a result of the low side cutter extending above the bolted beam), cantilevered off the high side rib line, placing increased demand on the supplemental support system (trusses).
6. Roof guttering then formed along the up dip side, eventually causing partial detachment from the rib. The bolted beam became completely detached from the rib. All load was then transferred onto the trussing system, exceeding its capacity, resulting in the fall of ground.

Rehabilitation

In order to prevent further propagation of the fall, work crews immediately began to install steel sets at 0.5m (1.64') intervals and hardwood cribs (0.76m x 0.76m {30"x30"}, at 1m spacing) immediately outby the fall area. As clean up of the fall progressed, steel sets (at 0.5m intervals) were erected. Four point, 1.5m x 1.5m (5'x5') cribs were constructed on top of the steel supports to secure the top of the fall. Upon completion of the fall clean up, the site was further stabilized by pumping a foaming cement into the void. The foam enabled effective movement of the gate end supports under the fall.

Geological information, visual observations and previous roof monitoring results identified the area outby the fall to be of even more concern. Prior to the re-commencement of longwall operations, a strategy was devised to re-support the remaining section of the transitional zone from the FPF through the BSF, a distance of 550m. The plan was to reconsolidate the broken ground mass of the immediate roof through the use of polyurethane resin (PUR) injection. The reconsolidated roof was then to be suspended into an overlying sandstone bed. The following are the specific details of the plan:

- Installation of steel sets for a distance of 20m from the outby fall limit (at 0.5m intervals).
- Installation of three (3) 9.6m (32') long, 17.8mm (0.7") diameter cable bolts at intervals of 1 meter (**Figure 7**). Installation of cable bolts was to be maintained 200m ahead of the retreating face.
- Installation of "IRMA" Injection bolts (alternating (2) 2.5m and (1) 3.5m bolts on 1.3m cycles (**Figure 8**). Injection of PUR was to be maintained 150 ahead of the retreating face (outside the front abutment zone).

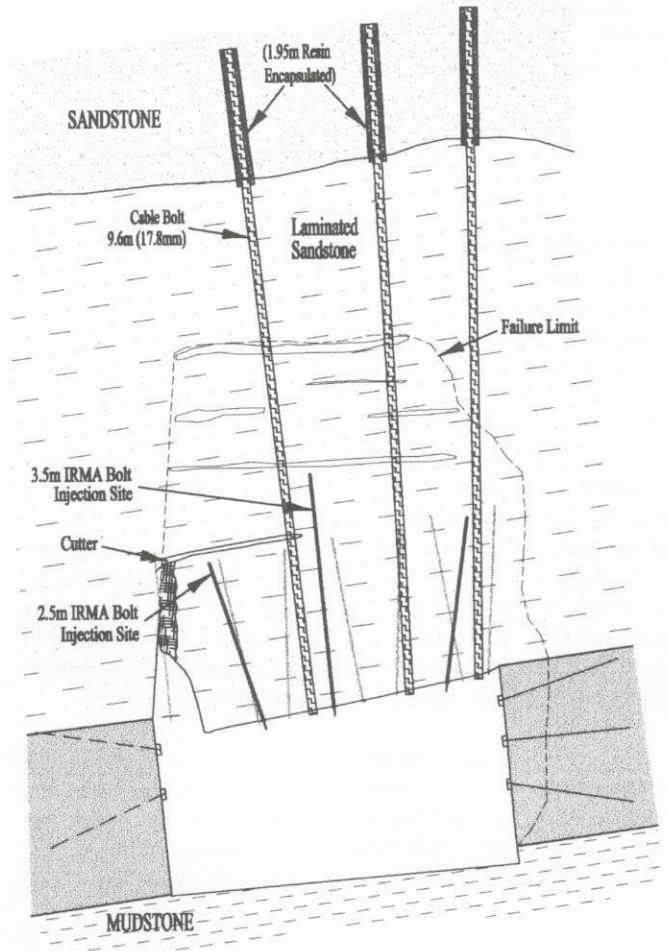


Figure 7. Roof Support Plan for the Rehabilitation of 8 East Maingate.

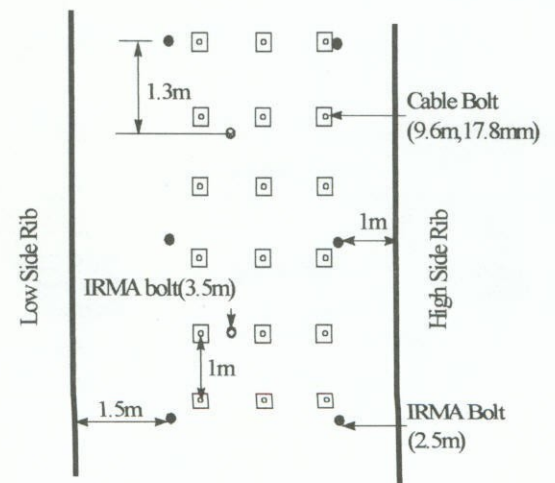


Figure 8. Plan View: IRMA Bolt and Cable Bolt Cycle.

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In addition, a double row of props were to be maintained for a minimum distance of 15 meters ahead of the gate end while the face advanced through this area.

APPLICATION OF POLYURETHANE RESIN

The decision to employ PUR as the primary ground consolidation technique while retreating the face through the Backswamp facies and the transitional zones on either side of it, was based on the successful application of such products in controlling difficult ground conditions in the coal mining industry.

The services of Hietech Ltd. were contracted to perform the injection. Hietech Ltd. employed a two component resin grout (Bevedan - Bevedol) system developed by CarboTech. The low viscosity of the product allowed for penetration of fissures of less than 0.1mm in width, in addition the pressure induced by foaming aided in fissure penetration. The resin was injected by pumping it through hollow 'IRMA' injection bolts preinstalled in the broken roof. The 'IRMA' bolt remained in the hole after resin placement, and was capable of being fitted with a bearing plate, further increasing roof confinement.

Before application of PUR was to commence, a number of obstacles had to be overcome. The most prevalent was the reluctance of the general workforce to handle what was perceived as a high risk health hazard material. Through a series of information sessions conducted with the section crews

and the Occupational Safety and Health Committee, many of the concerns were alleviated and the project was able to proceed.

The general application setup can be found in **Figure 9** with the procedure summarized in the following steps:

1. The two components are shipped to the underground site via mine rail cars in 200kg (196L) barrels.
2. The Bevedan / Bevedol products are transferred from the shipping barrels to translucent transfer barrels (with graduated markings), by means of small air operated pumps. The transfer barrels are located in close proximity to the pump operator so that the "draw down" rates of the resin components are apparent.
3. The injection pump (reciprocating piston design) draws the Bevedan / Bevedol components from the transfer barrels at a constant 1:1 ratio.
4. The two components are pumped through separate (high pressure) hoses to the injection site. Upon reaching the site, the components join in a mixer unit, which is threaded to an IRMA injection bolt.
5. The mixed resin migrates through the IRMA injection bolt via a one way valve at the base of the bolt. A "burst disc" at the top of the bolt provides containment allowing enough pressure to build to inflate a rubber packer (approx. 40 bar). Pressure will continue to build until the burst disc fails (at approximately 80 bar), thus allowing the PUR to freely flow.

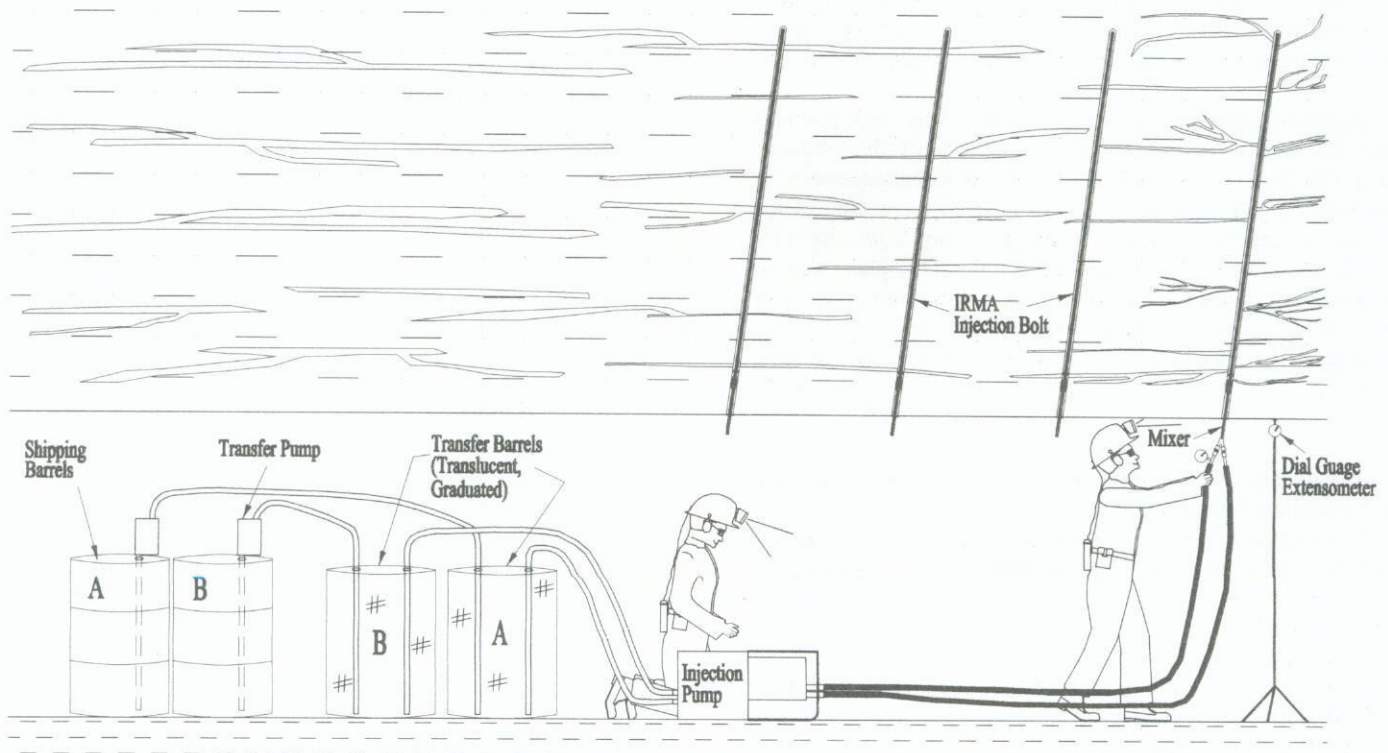


Figure 9. Application of PUR : General Setup.



6. The PUR is pumped into the fractures at a rate of approximately 8 L/min (at 5 bar). Pumping is terminated at a particular site when one of the following conditions occur:

- High back pressure develops (>100 bar)
- Expanding PUR causes excessive roof dilation (>4 mm)

Grouting operations began on Oct. 28, 1998, with the first injection site being located approximately 5m from the outby limit of the fall. Within the initial 10m, high volumes, up to 946 litres (250 gallons) were pumped at individual injection sites. Consumption eventually decreased to an average of 145 litres (38.5 gallons) per hole. A total of 550m of the gateroad was subject to PUR injection. Over this distance a total of 593 sites were injected with 86374 litres (22,820 gallons) of PUR.

Immediately after the fall of ground, 7.3m (24') electronic and mechanical sag stations were installed in the roof outby to monitor stability while the fall was being cleaned up. The decision to cable bolt and inject polyurethane required extensive drilling of the roof outby the fall. Drilling was accomplished with portable pneumatic bolters with water flushing. The roof monitoring stations measured the acceleration in roof movement as the drilling and polyurethane injection was being done (Figure 10). After polyurethane injection was completed sag stations were rendered ineffective and had to be reinstalled. Figure 10 also shows that even after a previous total of more than 200mm (8 inches) of roof movement the cable bolting and polyurethane injection made a huge improvement in rate of roof movement and made possible the subsequent successful wall retreat. The results of this sag station were repeated similarly on numerous monitoring stations.

Wall Production

Once the fall had been cleaned up and cable bolting and PUR had progressed outby the front abutment zone longwall production was reestablished. Production continued successfully under 500m of rehabilitated maingate roof without incident.

During longwall production, sag stations were continually re-established ahead of the front abutment zone of the longwall. Figure 11 shows results of the mechanical sag stations against distance from the face as the front abutment of the longwall approached. The results verify previous research which divided front abutment effects into 3 zones based on depth of cover. Onset of front abutment, 9.3 times the square root of depth (134m)(Peng, 1986), first acceleration, 5 times the square root of depth (72m)(Mark, 1990) and near face acceleration, 0.015 times depth (10m)(DeMarco, 1999). A graph showing rates of roof movement in these zones is shown in Figure 12. These graphs were used as a stability guideline. If any data appeared outside the pattern the area was checked and more standing support placed.

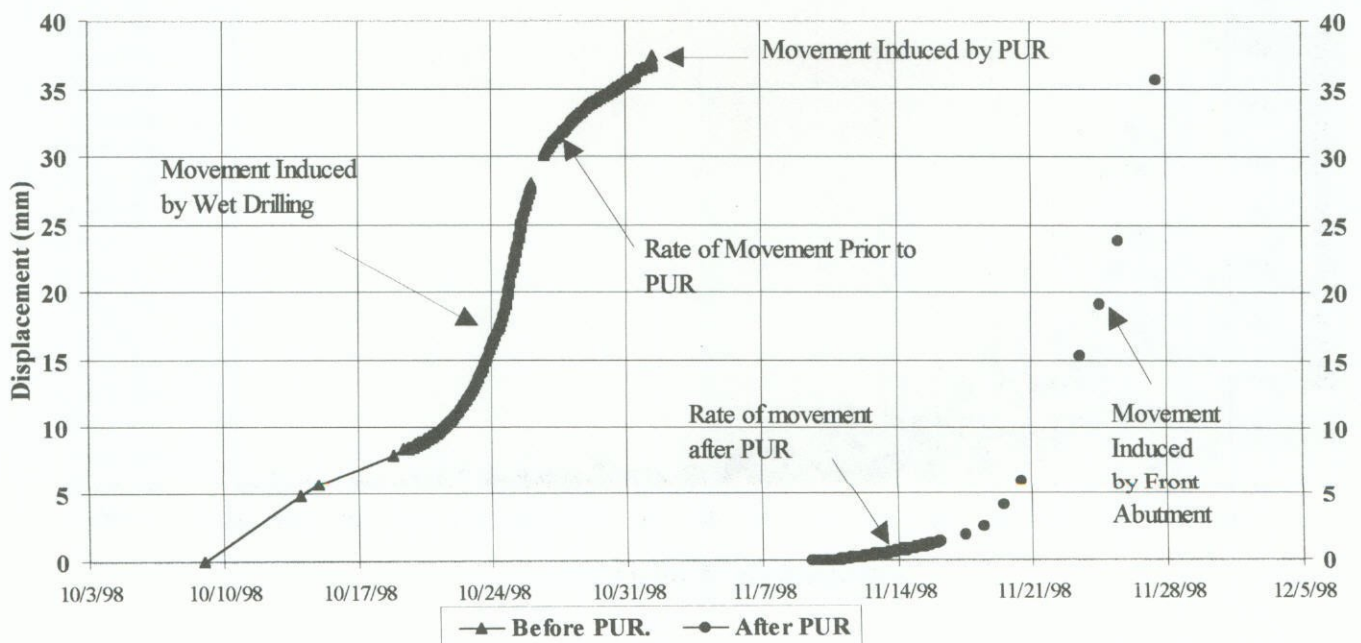


Figure 10. Roof Movement Versus Time For Various Stages of Rehabilitation of 8 East Maingate.

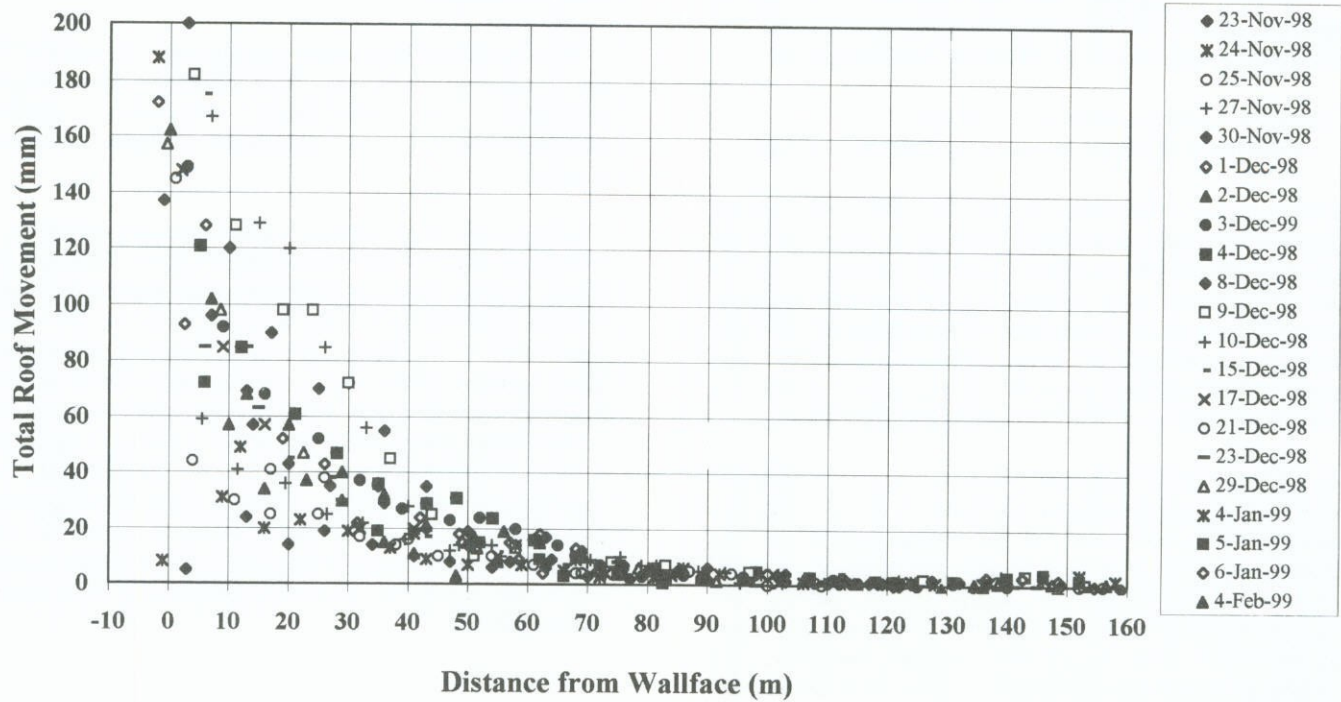


Figure 11. Total Roof Movement Versus Distance from Wallface for Cable Bolted and Polyurethane Grouted Roof in Front Abutment of Longwall.

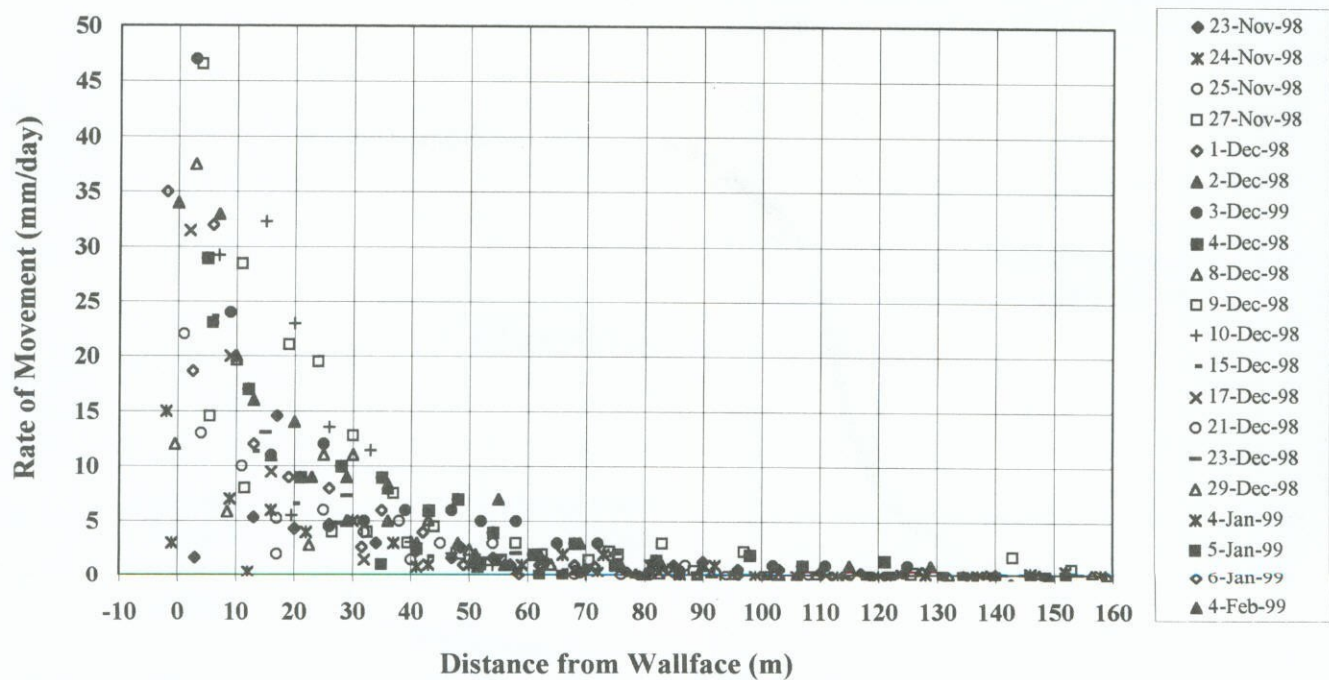


Figure 12. Rate of Roof Movement Versus Distance from Wallface for Cable Bolted and Polyurethane Grouted Roof in Front Abutment of Longwall.



CONCLUSIONS

The combination of depth, interaction effects, weak geology, front abutment, horizontal stress, increased roadway width and time was more than the roof in the 8 East maingate could withstand.

Polyurethane resin injection and cable bolting were effective techniques in reconsolidating an extremely poor roof which had a severe cutter and over 8 inches of roof sag at the centerline.

Monitoring verified previous research into the zones of front abutment, enabled a collection of data to determine the difference between normal and anomalous roof movement near the gate end, demonstrated the effect of wet drilling on weak roof, and demonstrated the ability of polyurethane and cable bolts to stabilize broken ground.

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Design Considerations for Bump-prone Longwall Mines

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ABSTRACT

Tailgate pillar and tailgate-face corner bumps have been a major threat to longwall coal mines where bump-prone conditions exist. The yield pillar design concept, in which the longwall chain pillars are designed in such way that they can yield in a non-violent or controlled manner during longwall retreat, has been used in many longwall mines to prevent tailgate pillar bumps. However, this practice often shifts the side abutment load onto the tailgate-face corner especially when a massive and strong sandstone member is present within the caving zone or in the vicinity of the coal seam. Therefore, the abutment pillar design concept has also been used in some bump-prone coal mines to minimize the stress concentration on the tailgate panel rib for the control of tailgate-face corner bumps. The disadvantage of using the abutment pillar design concept is that it is more costly and puts additional pressure on mine operations for gate entry development and significantly reduces the recovery of coal reserve. In addition to pillar size, panel layouts, such as the panel width and panel orientation, also play a critical role in pillar or face bumps, and should be taken into account.

Based upon the Cyprus mine design and operational experiences at Willow Creek and case studies at other bump-prone longwall mines in the U.S., the authors attempt to identify the critical factors causing coal mine bumps and to introduce some guidelines and criteria for the design of both yield and abutment pillar systems and panel layouts.

INTRODUCTION

Coal bumps are defined as sudden and violent failures of entry or face ribs in underground coal mines where a bump-prone coal seam is exposed to a high stress environment during either development (rarely) or mining process (often). They occur in such a magnitude that large volume of coal fragments can be expelled into the extracted areas in a very short period of time, which may or may not induce roof falls or floor heave, depending on roof or floor conditions. Hazards associated with coal bumps

include fatalities or injuries caused by suddenly failed coal fragments either hitting the individual or by forcing the individual into nearby equipment or entry ribs, and damage to or loss of equipment.

Although such events have a significant impact on miners' safety and mine production, confusions still exist regarding their definitions. For the same event, some people call it a bounce while others call it a bump if such an event is not accompanied by a release of large volume of gases (1-3). At the former Bureau of Mines, a bounce was defined as a sudden forceful impact or vibration felt in the mine, which may be accompanied by face or rib sloughage and/or pillar and roof separation; while a bump was defined as an instantaneous release of roof, rib, or floor material propelled into a mine opening (2). In reality, a bump event may also involve a sudden forceful impact or vibration felt in the mine. According to this definition, a bump event is more severe than a bounce event because an instantaneous release of roof, rib, or floor material propelled into a mine opening may injure miners, damage equipment, or make the working area not accessible, which will interrupt the normal production process. On the other hand, the face or rib sloughage and/or pillar and roof separation involved in a bounce event will most likely not interrupt the normal production process if no large volume of coal fragments are involved or thrown over the face conveyor onto the walkway in a longwall face or across the entry center on the other side of the entry. Therefore, from safety and production point of view, an event that may have the potential of interrupting the normal production process (injuring people, damaging equipment, or making the working area not accessible) should be defined as a bump. Otherwise, it should be defined as a bounce.

Since more than 90% of the longwall mine bumps occurred in either tailgate pillars or tailgate-face corners, the importance of the chain pillar size and the effect of the side abutment load induced by the adjacent gob can never be over-emphasized for longwall mining in the bump-prone coal seams. Unfortunately, no effective guidelines or methods are available to determine which pillar design concept should be used for what type of roof lithology, how to ensure an intended yield pillar system to behave as expected, what the minimum abutment pillar size should be for

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the bump control, and whether the panel orientation should favor the breakage of the roof strata along the gobline or behind the face supports. In this study, the authors try to answer some, if not all, of these questions based upon the Cyprus mine design and operational experiences at Willow Creek and case studies at other bump-prone longwall mines in the U.S.

FACTORS CAUSING COAL MINE BUMPS

Generally, coal mine bumps can be characterized as pressure or static and shock or dynamic bumps (1,2). A pressure bump is induced by static loading on a strong coal seam where the pillar or face stress exceeds its failure strength. A shock bump is triggered by dynamic loading on a strong coal seam where the already-highly stressed pillar or face is under the influence of tremors or vibration induced by the failure or slippage of massive strata in the vicinity of the coal seam. Whether it is a pressure bump or a shock bump, the occurrence of a pillar or face bump is mainly determined by the ability of storing and releasing strain energy of the coal seam when it is heavily loaded. It is believed that a structurally strong coal seam can store large amounts of strain energy before its failure and that a stiffer and stronger immediate roof or floor member can provide a greater lateral confinement at the interface between rock and coal, which determines the location of the abutment loads around a opening and affects the mode of pillar and face failures. For example, a well-structured coal seam can yield easily near pillar and face ribs even under a low stress environment and thus cannot store significant strain energy before it fails; a weak immediate roof or floor member can fail easily and has a less lateral constraint to the coal seam under a high stress environment and thus throw the abutment stress zone further into the solid coal; and a low value of stress rarely cause sudden coal failure and thus coal bumps.

Coal Seam Strength

A strong coal seam is defined as a less-structured coal seam with a large value of uniaxial compressive strength. Since the small coal specimens for lab testing usually contain less or small scale of structures and represent only the strongest part of a coal seam, coal strength alone cannot be used to reflect its structural strength and thus its actual behavior. Therefore, the concept of the structural strength of a coal seam, defined by both its strength in lab value and well or not-well developed cleats or fractures, and macro scale and gradation of strength and cleats along the direction of its thickness, should be introduced to describe its ability of storing and release strain energy and yielding behavior, and thus its bump potential when exposed to a high stress environment. The failure mode of coal specimens in lab testing and the location of the yield zone near pillar or face ribs can be used to explain why a strong coal seam is more bump-prone.

Failure Mode of Coal Specimens in Lab Testing

Table 1 lists the testing results for Willow Creek's D seam. It can be seen that the violent shattering failure occurred when the specimens reached a strength of greater than 4,000 psi, while a non-violent failure mode was observed for those specimens with

a strength of less than 4,000 psi. Although this one-event testing with a limited number of specimens is not conclusive, the results do indicate that the D seam may fail in a violent manner in the areas where the coal seam is less-structured and exposed to a high stress environment.

Location of the Yield Zone near Pillar or Face Ribs

A structurally -weak seam can yield easily near the pillar or face ribs and throw the peak stress further into the solid coal, which in turn reduces the potential of coal bumps. For example, auger drilling, volley firing, hydraulic fracturing, and water infusion have been used as preventive measures to control coal mine bumps (1-8). The purposes of the first three methods are (a) to create fractures in the highly-stressed pillars or faces and thus to release the high strain energy and (b) to force the high stress zone back deeper into pillars or faces and thus to reduce the chance of coal bumps. The function of coal seam water infusion (low pressure), which has been successfully used in Ruhr District,

Table 1 Failure Mode and Strength of the D Seam

Specimen No.	Uniaxial compressive strength	Failure Mode
	(psi)	
1-1-1	7,652	Violent
1-1-2	5,948	Violent
1-2-1	4,604	Violent
1-2-2	4,035	Violent
1-3-1	3,341	Non-violent
1-3-2	3,705	Non-violent
1-4-1	3,678	Non-violent
1-4-2	3,367	Non-violent
2-1-1	5,913	Violent
2-1-2	6,848	Violent
2-1-3	6,826	Violent
2-2-1	1,498	Non-violent
2-2-2	3,274	Non-violent
2-3-1	2,731	Non-violent
2-3-2	2,154	Non-violent
2-4-1	3,839	Non-violent
2-4-2	3,320	Non-violent

Germany, is to reduce the coal seam strength and change its elastic behavior, and thus to prevent coal bumps. To explain the effect of water infusion on coal seam strength, 28 coal samples with 20 air-dried and 8 saturated were taken from a bump-prone seam, called No. 18 seam, in Ruhr District (Table 2). The No. 18 seam is liable to heavy bumps and four major bump events



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occurred in the region from which the samples were taken. The strongest of these bumps completely destroyed a 262 ft long roadway with a width of 20 ft.

Table 2 Results of Mechanical Property Tests

Coal seam	Moisture condition	UCS	E	Energy index	No. of samples
		(psi)	(10 ⁶ psi)		
No. 18	Air-dried	1,523	0.15	5.32	20
	Saturated	856	0.07	0.88	8

Note: UCS - uniaxial compressive strength and E - Young's modulus.

Borehole-bursts testing results in Germany also show that borehole-bursts occurred in almost all types of hard coal specimens with a volatile matter of greater than 10% and no borehole-bursts were observed when such hard coal specimens had been saturated with water.

In summary, all the preventive methods discussed above are used to weaken the overall structural strength of the coal seam and to force the abutment zone deeper into the solid coal, and thus to minimize the potential of coal mine bumps. Therefore, it can be concluded that a less-structured coal seam with a great value of uniaxial compressive strength will be more bump-prone than a well-structured one.

Coming to coal behavior, although the energy index and post-failure stiffness of coal have been put forward as measures of bump proneness by many researchers, the application of each individual method to determine if a coal seam is bump-prone is questionable. For example, it is believed that a coal seam with an energy index of greater than 2 will be considered characteristic of bump-prone coal and greater than 5 of highly bump-prone coal (1). However, an energy index of 4.72 was obtained for the No. 54 coal seam in Ruhr District, which has been mined for more than 20 years and had no bump occurrence even at depths of 1,968 to 4,756 ft. This raises a good question as to how representative the laboratory samples and strength numbers are compared with the coal seam behavior as it relates to fractures, cleats, layering of hard coal over or under the structured layer. The energy index of coal is defined by:

$$W_{ET} = \frac{E_c}{E_d} \quad (1)$$

where W_{ET} is the energy index of coal, E_c is the elastic strain energy, and E_d is the dissipated strain energy during loading release from 80% of the ultimate strength of coal.

Stiffness of the Immediate Roof and Floor Members

The feature most common to the coal seams where coal bumps occurred is the close proximity to strong, thick and rather rigid strata. In most coalfields these strata consist of sandstone. In rarer cases they consist of other types of stiffer and less

yieldable rock strata. A mine roof or floor consisting of such stiffer and less yieldable members will provide a greater lateral confinement force to the coal seam and thus result in the peak stress near the entry or face ribs. In other words, under the same stress and coal seam condition a weaker roof or floor member will result in a larger yield zone around an excavation transferring the peak stress deeper into the coal pillar or face, which can dramatically reduce the coal bump potential. The concept of throwing the abutment zone further into the solid coal has successfully been used in many bump-prone coal mines to minimize or eliminate coal bumps by controlled blasting, auger drilling, hydraulic fracturing, or water infusion.

Stress Environment

In longwall mining, the chain pillars between longwall panels usually experience the development or tributary load due to entry development, the first front abutment load due to the previous panel's mining, the side abutment load induced by the previous panel's gob, and the second front abutment load due to the active panel's mining. Therefore, the total accumulated vertical stress in these chain pillars can reach as high as 2 to 4 times the original vertical stress, depending upon the pillar size, entry width, gob size, and roof conditions. The similar stress concentration can also be found in the tailgate-face corner if the chain pillar system between panels is not wide enough to minimize the impact of the side abutment load induced by the adjacent gob. Therefore, a high frequency of coal bumps will be expected in the tailgate pillar or tailgate-face corner during longwall retreat (Table 3). In Table 3, each case either represents at least more than one bump events that occurred in the same longwall panel or many bump events in a specific coal mine. It should be noted that some face bumps were reported in a general description and no specific location along the face was given except in the case of the Basin seam in Colorado operated by Mid Continent (2). For example, among the nine bump events (either coal or/and floor) occurred between January 1983 and July 1984, five were between shields #7 and #46 and four affected tailgate-end face and tailgate entry. The worst one occurred in the tailgate-end face and tailgate entry on April 20, 1983, which resulted in a rapid floor heave that extended about 1,200 ft along the tailgate entry and 120 ft into the face. This bump event damaged 40 shields and disrupted panel ventilation.

It is very obvious from Table 3 that if we can prevent tailgate pillar and tailgate-end face or tailgate-face corner bumps, the severity and probability of longwall coal mine bumps can be dramatically reduced. Therefore, the pillar stress condition at each mining stage should be analyzed and approximately estimated. Furthermore, the critical parameters that influence the magnitude of development and mining-induced abutment loads in longwall mining should be identified and quantified for the purpose of optimizing mine design to alleviate stress concentration and thus bump potential.

Chain Pillar Stress

As discussed above, the longwall chain pillars usually experience development, first front abutment, side abutment, and

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Table 3 Historical Data of Longwall Coal Mine Bumps

Coal Seam	Seam Thickness	Cover Depth	Panel Width	Longwall Chain Pillars			Bump Location
				W	L	No.	
				(ft.)	(ft.)		
Sub-3 (UT)	6-7	8-20	450-610	30-85	85-120	1-2	Seam dip: 4 to 6 deg.
Case 1		10-16	500	50, 50	85	2	TG pillar & TG-end face (violent)
Case 2		8-16	500	40, 40	85	2	TG pillar & TG-end face (violent)
Case 3		8-14	450	30, 30	85	2	TG pillar & TG-end face (violent)
Case 4		8-12	500	30, 30	85	2	TG pillar & TG-end face (violent)
Case 5		8-16	520	35, 85	120	2	TG pillar & TG-end face (minor)
Case 6		12-20	610	85	120	1	Face (violent)
D (UT)	10	12-22	640	85	85	1	Seam dip: 4 to 6 deg. HG pillar (minor) & bleeder pillar (violent)
Lower Sunnyside (UT)	4.5-12	5-25	300-635	25-45	80-85	1-2	Seam dip: 3 to 12 deg.)
Case 1	4.5-6	5-13.5	520	45, 45	85	2	TG pillar & TG-end face (violent)
Case 2	4.5-6	5-13.5	500	30 or 45	85	1	TG pillar & TG-end face (violent)
Case 3	6	20	500	> 30	80	1	TG-end face (violent)
Case 4	5	10-25	635	> 30	80	2	TG-end face & TG pillar (minor)
Case 5	5	10-25	430	20, 45	80	2	TG-end face & TG pillar (violent)
Case 6	5-6	15-22.5	300	25, 35	85	2	TG bumps (violent)
Case 7	5-6	15-23.5	300	30	85	1	Face (violent)
Case 8	5-6	15-25	300	30	85	1	Face (violent)
Case 9	6-12	10	550	40	85	1	Face (violent)
Case 10	6-12	10-18	550	40	85	1	TG-end face and TG pillar (violent)
Case 11	6-12	15-22	550	40	85	1	Face & TG pillar (minor)
Case 12	6-12	15-23	550	30	85	1	Face (minor)
Case 13	6	15-25	550	30	85	1	Face (minor)
Case 14	4-6	10	350	40, 40	80	2	TG-end face & TG pillars (violent)
Case 15	4-6	5-10	350	40, 40	80	2	TG-end face & TG pillar (violent)
Case 16	4-6	5-10	400	40, 40	80	2	TG-end face & TG pillar (violent)
Basin (CO)	10	30	800	N/A	N/A	2	Face & TG panel rib (violent)
Pocahontas No. 3 (VA)							
Case 1	5	18-24	450	30, 30, 80	80	3	TG 80' pillar and TG-face corner (violent)
Case 2	6	12-22	450	30, 80, 30	80	3	Center 80' pillar & TG-face corner (violent)
A (UT)	8	16	700	30, 30	80	2	TG-end face & TG panel rib (violent)

second front abutment loads during their service lives. The development load on a pillar is determined based upon the tributary loading concept as (10):

$$L_d = (W_p + W_e) (l_p + W_e) h \gamma \quad (2)$$

where L_d = development or tributary load on a pillar, in lb
 W_p = pillar width, in ft
 l_p = pillar length, in ft
 W_e = entry or crosscut width, in ft
 h = depth of cover, in ft
 γ = average density of overburden strata, in lb/ft³

The pillar stress due to entry development is

$$\sigma_d = \frac{L_d}{A_p} = \frac{(W_p + W_e)(l_p + W_e) h \gamma}{144 W_p l_p} \quad (3)$$

where σ_d is the development stress, in psi and A_p is the pillar bearing area in in².

The side abutment load for a critical or supercritical panel can be determined by:

$$L_s = \frac{h^2 \tan \beta (l_p + W_e) \gamma}{2} \quad (4)$$

where L_s is the side abutment load due to gob effect and β is the shear or abutment angle of roof strata along the gobline or called angle of draw, in degrees.

The distance of the side abutment load distribution from the gob edge over the chain pillar system into the next panel can be estimated by (11):

$$D = 9.3 \sqrt{h} \quad (5)$$

where h is the depth of cover, in ft. The distribution or reduction factor of the side abutment load can be determined by:

$$R_s = 1 - \left[\frac{D - (W_p + W_e)}{D} \right]^3 \quad (6)$$

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The side abutment stress on a pillar is determined by:

$$\sigma_s = \frac{R_s L_s}{144 W_p L_p} \quad (7)$$

Currently, no analytical methods available to determine the first front abutment load or headgate front abutment load. However, according to Mark (10), it can be roughly estimated by:

$$\sigma_{hf} = 0.5 \sigma_s \quad (8)$$

and the combination of both the first and second front abutment loads on a pillar adjacent to the tailgate side or tailgate front abutment stress is determined by:

$$\sigma_{ff} = 0.7 \sigma_s \quad (9)$$

The accumulated vertical stress on a headgate pillar outby the first longwall face can be determined by:

$$\sigma_h = \sigma_d + \sigma_{hf} \quad (10)$$

The accumulated vertical stress on a tailgate pillar outby the second longwall face is

$$\sigma_t = \sigma_d + \sigma_{ff} + \sigma_s \quad (11)$$

Parameters Influencing Pillar or Face Stress Concentration

It can be seen from Eqs. 2 through 11 that the magnitude of vertical pillar stress mainly depends on pillar size, entry width, depth of cover, shear angle of roof strata along the gobline, and how far each individual pillar is from the gobline. As the depth of cover increases so does the pillar and face stresses. However, analysis of the historical data of coal mine bumps in both Germany and the U.S. shows no significant correlation between the overburden depth and frequency of coal bump events. For example, severe coal bumps occurred in Pocahontas No. 4 and Chilton seams in West Virginia and Lower Sunnyside and Sub-3 seams in Utah where the depth of cover is less than 800 ft (4-7), while the No. 54 seam in Ruhr District, Germany, where the depth of cover reaches as deep as 4,756 ft without occurrence of coal mine bumps (1). Coal mine bumps were reported by Bräuner (1) at a depth of cover even less than 328 ft. Therefore, it is difficult to determine the critical depth of cover above which a coal mine bump may occur.

The shear angle of roof strata along the gobline is controlled by percentage of hard rock, such as massive and strong sandstone members, in the overburden strata, and ranges from 18° to 25° (10). As a rule of thumb, if the percentage of hard rock is small, the lower value is recommended, and if the percentage is large, the upper value should be used. In longwall mining, the location of massive and strong strata also play a critical role in pillar or face stress concentration. For example, if a massive and strong sandstone roof with a thickness of greater than 40 ft is located within the caving zone (6 to 8 times mining height), its overhang

before breakage along the gobline or behind the face supports will have a significant impact on tailgate pillar stress or longwall face stress. However, if it is located far away above the caving zone, its influence on pillar or longwall face stress may not be instrumental.

In summary, both the depth of cover and massive and strata in the vicinity of the coal seam will most likely result in high stress concentration in tailgate pillars or tailgate-end faces and thus coal bumps during longwall retreat if a strong coal seam and a stiff and strong roof or floor member is present. However, massive and strong roof strata in the vicinity of the coal seam may play a more critical role than the depth of cover in coal bumps. For example, tailgate pillar and tailgate-end face bumps were more severe and frequent in Castlegate No. 3 mine (Sub-3 seam), where the depth of cover was as low as 800 ft and a massive sandstone ranging from 80 to 150 ft thick present 40 to 100 ft above the seam, than in Castlegate No. 5 (D seam), where the depth cover reached as high as 2,200 ft and no massive sandstone strata was present within 550 ft above the seam.

Although the impacts of many other factors on pillar or face bumps cannot be quantitatively determined, their roles in coal mine bumps can be characterized as the stress effects. They are:

- Multiple seam mining,
- Sudden change of overburden depth in a short distance,
- Local irregularities such as changes in seam thickness (washouts), roof lithology (transition zones), and floor rolls,
- Geological structures such as synclines or faults with small displacement, are occasionally regarded as contributory factors, and
- Horizontal stress field.

DESIGN CONSIDERATIONS IN CONTROLLING COAL BUMPS

Past experiences in controlling longwall coal mine bumps indicate that the most effective and economical way is to have a proper panel layout and to design a proper longwall gate entry system. A proper panel layout, such as its orientation and width, can encourage massive and strong roof or overburden strata to cave along the previous panel's gobline or behind the face supports and thus minimize the mining-induced stress concentration on tailgate chain pillars or longwall faces. A proper longwall gate entry system can either prevent tailgate pillar bumps by employing a purely yield pillar system or control tailgate-face corner and tailgate pillar bumps by using a combination of yield and stable or abutment pillar system, or leaving a wide enough barrier pillar between panels.

Panel Orientation

If the mining direction of longwall panels is orientated about parallel or in a small angle to the primary joints in the overburden strata, they can easily cave or break along the adjacent gobline and thus may result in a less stress concentration on the tailgate pillar and tailgate-face corner. However, this orientation will not



favor the immediate roof caving behind the face supports and thus may result in front abutment stress building up across the faceline due to the nature of longwall mining. This is especially true if a massive and strong sandstone member is present within the caving zone. In practice, it is very difficult, if not impossible, to orientate both the panel and face in the direction of favoring roof caving in terms of joint orientations. Therefore, the best longwall panel orientation should be determined by considering not only joint orientations, but also the roof lithology and chain pillar size between panels.

For example, if a massive and strong sandstone member exists within the caving zone, its cantilever beam overhanging along the gobline and behind the face supports will have a significant impact on the mining-induced abutment stresses in the tailgate pillars and longwall face. The longwall face should be orientated in a small angle to the major joints with a yield-stable or yield-stable-yield pillar design or leaving a wide enough barrier pillar between panels. This design will encourage the sandstone roof to cave immediately behind the face supports. Under this scenario, an abutment pillar or a barrier pillar can be used to force the sandstone roof to break along the gobline and to bear the most part of the side abutment load.

However, if a massive and strong sandstone member exists far above the caving zone, its cantilever beam overhanging behind the face supports will have a less impact on the front abutment stress building up across the longwall face because its back-end will most likely be fully supported by the rock fragments of the immediate roof due to the bulking factor effect. Under this condition, the face orientation is not as critical as the panel orientation because the face is advanced daily and feels much less effect of the overburden strata movement than the tailgate-chain pillar along the gobline. Therefore, the panel edge should be orientated in a small angle to the primary joints to encourage the overburden strata to cave along the gobline. For this scenario, a purely yield pillar system can be successfully used without severe tailgate-face corner face bumps during longwall retreat.

Longwall Gate Entry System

To control tailgate pillar bumps, the pillar size (mainly width) should be designed either as narrow as possible such that it can yield in a non-violent manner, or as wide as possible such that its average strength is greater than its stress and will never fail during its service life. The former approach is called "yield pillar design concept" and the latter is called "stable or abutment pillar design concept." However, in deep longwall mines the stable pillar design concept usually requires a very wide gate entry system and thus have a significant impact on the gate entry development and recovery of the coal reserve. Therefore, a purely stable pillar system has rarely been utilized in deep longwall mines. Although the purely yield pillar system, if designed properly, can eliminate tailgate pillar bumps, it cannot prevent tailgate-face corner bumps if a massive and strong sandstone member exists in the vicinity of the coal seam. Therefore, either a purely yield pillar or a combination of yield and stable or abutment pillar system should be used, depending on the roof lithology.

Yield Pillar Design

A purely yield pillar system can be divide into two groups. They are one-yield pillar and two-entry system and two-yield pillar and three-entry system. Although these two systems have been used for many years in the Western U.S. bump-prone coal mines (Table 3), no methods are available to effectively design the yield pillar so that it can yield in a non-violent manner. That is why some of the intended yield pillars with a width ranging from 30 to 50 ft in the Sub-3 seam and Lower Sunnyside seam in UT experienced violent bumps. As a rule of thumb, the width-to-height ratio has been used to estimate the pillar width for yield pillar design (12,13). It was concluded that no successful yield pillar designs employed width-to-height ratio of greater than 5. In other words, a width-to-height ratio of greater than 5 may result in a critical pillar size that is too large to yield non-violently and too small to bear the mining-induced abutment loads without sudden failure. Since a pillar bump event is determined by the rate of pillar yielding (gradual or sudden), in addition to the width-to-height ratio, pillar strength, pillar height, and entry width should also be taken into consideration in designing a yield pillar system for bump control. Therefore, in this study three parameters will be proposed for yield pillar design. They are the stability factor, width-to-yield zone ratio, and width-to-height ratio.

The stability factor is used to determine the critical size of a pillar below which it can yield or fail in a non-violent or controlled manner, and is defined as the ratio of its strength and stress under the tailgate load as follows:

$$SF_t = \frac{S_p}{\sigma_t} \quad (12)$$

Where SF_t is the stability factor of a pillar when it is exposed to the tailgate stress; the tailgate pillar stress, in psi, is defined by Eq. 11; and S_p is the pillar strength can be determined by (14):

$$S_p = S_1 (0.64 + 0.54 \frac{W_p}{H} - 0.18 \frac{W_p^2}{H l_p}) \quad (13)$$

where S_1 is the in-situ coal strength, in psi and H is the pillar height, in ft. The in-situ coal strength can be estimated by:

$$S_1 = R C_o \quad (14)$$

where R is the reduction factor from lab to in-situ and C_o is the uniaxial compressive strength of 1 inch cubical specimens in lab value, in psi. According to different researchers, the R ranges from 1/5 to 1/7.7. Since the strength of coal specimens in lab varies significantly even from the same coal seam, $S_1 = 900$ psi was recommended by Mark (10) for all the coal seams based on his analysis of field data.

Ideally, a yield pillar should yield after it is formed or under the influence of the first front abutment load. The yield zone, x_b , is the distance from the pillar rib to the peak stress or abutment

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stress inside the pillar. Theoretically, if a pillar width is less than $2x_b$, it has no core left and should totally yield. Otherwise, it may or may not fail in a violent manner under mining-induced abutment loads, depending on its core strength and how much abutment loading it can take before its failure. To reflect the potential of pillar bumps under mining-induced abutment loads, the width-to-yield zone ratio of a yield pillar is also introduced and defined as follows:

$$R_y = \frac{W_p}{2x_b} \quad (15)$$

where R_y is the width-to-yield zone ratio of a yield pillar and the yield zone width for strong roof strata can be determined by (15):

$$x_b = \frac{H}{F} \ln\left(\frac{q}{p'}\right) \quad (16)$$

with

$$F = \frac{k-1}{\sqrt{k}} + \frac{(k-1)^2}{k} \tan^{-1} \sqrt{k} \quad (17)$$

where x_b = yield zone width near the pillar ribs, in ft
 H = pillar height, in ft
 q = vertical stress due to the overburden strata, in psi
 p' = cohesive strength of the broken coal at pillar ribs,
 $p' = 14.5$ psi
 k = triaxial factor, $k = (1 + \sin\phi) / (1 - \sin\phi)$
 ϕ = angle of internal friction of coal, in degrees

Note: $\phi = 30^\circ$ to 37° (Wilson), $\phi = 23^\circ$ to 42° (Barron), and $\phi = 25^\circ$ to 50° (Salamon).

Because the deviation of the physical and mechanical properties of a coal seam is so significant and the stress conditions are complex and uncertain, the pillar stress and strength as well as the yield zone depth near pillar or face ribs determined using the empirical equations above may not be conclusive. Therefore, in addition to the stability factor and width-to-yield zone ratio, the width-to-height ratio of a yield pillar is also used as a rule of thumb in yield pillar design. It is defined as:

$$R_H = \frac{W_p}{H} \quad (18)$$

where R_H is the width-to-height ratio of a yield pillar.

To determine the critical or threshold value, below which the intended yield pillar can yield or fail in a non-violent manner, some case histories listed in Table 3 were analyzed against the pillar size, entry size, average depth of cover, angle of internal friction, shear angle, and in-situ coal strength. The results are listed in Table 4. It should be noted that the same values of internal friction (35°), shear angle of the roof strata (21°), and in-

situ coal strength (900 psi) were assumed for all the cases in this study. Analysis of the results in Table 4 indicate that tailgate pillar bumps can be eliminated for a purely yield pillar system if all the following conditions can be met:

1. Stability factor $SF_t \leq 0.18$,
2. Width-to-yield zone ratio $R_y \leq 1.5$, and
3. Width-to-height ratio $R_H \leq 5$.

On the other hand, if all the three parameters are greater than their threshold values, the intended yield pillar will yield in a violent manner like all the Cases in the Sub-3 seam, and Cases 1 and 10 in Lower Sunnyside seam. If only one of the four parameters is greater than its corresponding threshold value, like Case 11 in Lower Sunnyside seam, a tailgate bump event with a light manner may be possible.

In summary, the proposed three parameters can be used as a design guide to determine the proper yield pillar width for bump-prone longwall mines.

Abutment Pillar Design

As discussed earlier, a purely yield pillar system, if designed properly, can eliminate tailgate pillar bumps. However, the potential of tailgate-face corner or tailgate-end face bumps may or may not be preventable, depending on the roof lithology. For example, in the cases of Lower Sunnyside and A seams, even though the 30 ft wide pillar yielded as expected and produced no bumps, minor or violent tailgate-end face or face bumps could not be prevented. It is believed that the 180 ft thick and strong sandstone that is located about 150 ft above the Lower Sunnyside seam and the 80 to 150 ft thick and strong sandstone that is located 40 to 80 ft above the A seam played a critical role in the side abutment stress concentration along the tailgate panel ribline. Therefore, the abutment or stable pillar design concept has also been used to isolate the side abutment load effect and thus to minimize the potential of tailgate-face corner bumps. However, due to the bump potential of the abutment pillar when exposed to the mining-induced abutment loads, a yield-stable-yield pillar system has been widely used in the Pocahontas No. 3 seam where a 50 to 120 ft thick sandstone exists 0 to 150 ft above the seam and the depth of cover ranges from 1,800 to 2,400 ft. Conventionally, if a structure is defined as stable its strength must be greater than its stress. Because of the pillar size used in this seam vs. the significant mining-induced abutment loads due to the deep cover, it is more accurate to define the chain pillar system as a yield-abutment-yield pillar system rather than a yield-stable-yield system. Similar yield abutment yield pillar system has been successfully used by Jim Walter Resources in Alabama Blue Creek seam.

The purposes of using this chain pillar system are (a) to minimize the impact of the side abutment load on tailgate-face corner bumps and (b) if the center abutment pillar bumps, its impact on longwall production will be minimum due to shielding provided to the face gate by the adjacent yield pillar. Therefore, the question for designing this system is how narrow the center abutment pillar should be to prevent the abutment pillar and

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Table 4 Comparison of the Three Parameters for the Purely Yield Pillar System

Coal Seam	h (ft)	W_p (ft)	S_p (psi)	SF_t	R_y	R_H	TG pillar bumps
Sub-3							
Case 1	1,300	50	3,367	0.33	2.7	7.1	Yes (violent)
Case 2	1,200	40	2,918	0.29	2.2	5.7	Yes (violent)
Case 4	1,000	30	2,414	0.28	1.7	4.3	Yes (violent)
L. Sunnyside							
Case 1	925	45	3,578	0.55	2.7	7.5	Yes (violent)
Case 7	1,925	30	2,720	0.12	1.5	5.0	No
Case 10	1,400	40	2,397	0.19	1.7	4.4	Yes (violent)
Case 11	1,850	40	2,397	0.13	1.6	4.4	Yes (minor)
Case 13	2,000	30	3,308	0.16	1.5	5.0	No
A	1,600	30	2,184	0.13	1.4	3.8	No

tailgate-face corner bumps during longwall retreat. Two pillar configurations for the center abutment pillar used at Island Creek's mine site were analyzed and the pillar stability factors under the tailgate load are listed in Table 5. According to Gauna (16), the 80-by-80 ft (rib-to-rib) center pillar began bumping 500 ft ahead of the retreating longwall face. The failure of this abutment pillar allowed the side abutment load induced by the previous gob to concentrate on the corner of the active face. Occasionally, this situation resulted in the tailgate-face corner bumps as the shearer mined out the tail portion of the face. Significant tailgate entry convergence and roof control problems were also present in the T-junction where the tailgate drive motor was located. The maximum roof-to-floor convergence reached 2.5 ft in the tailgate T-junction.

To reduce the side abutment load effect and to have a better roof control at the tailgate T-junction, the center abutment pillar size was increased from 80-by-80 ft to 130-by-180 ft (rib-to-rib). As a result, the tailgate entry convergence was reduced from 2.5 to 0.5 ft and the tailgate face corner bumps were minimized. The center abutment pillar did not bump until approximately 100 ft inby the face. The bump density was greatly reduced in comparison to the bump events experienced with the old chain pillar system. It is obvious from Table 3 that the 80-by-80 ft pillar is too large to yield non-violently when exposed to the tailgate load, which agrees well with the conclusion obtained from the yield pillar cases in Table 4 which states that SF_t is greater than 0.15 the pillar failed in a violent manner. The 130 ft wide abutment pillar with a stability factor of 0.64 did function as expected, which agrees well with the findings from Zelanko and Heasley (17). They concluded that the abutment pillars with a safety factor of greater than 0.6 produced no bumps in terms of their case studies in both the Pocahontas No. 3 seam in VA and the Harlan seam in KY.

In summary, a stability factor of 0.6 should be considered the

minimum value for the abutment pillar system.

Independent Panel Layouts

Similar to the abutment pillar concept a barrier pillar can also be employed between panels to isolate the impact of the side abutment load induced by the adjacent gob on the active panel and thus minimize the potential of tailgate-end face bumps. However, how wide the barrier pillar should be to effectively prevent the next panel's tailgate-end face bumps is still not completely understood. For example, a 150 ft wide barrier pillar was left between Area 6 and Area 7 in the Sunnyside No. 1 mine where the depth of cover was 2,000 ft and a 180 ft thick sandstone existed about 150 ft above the coal seam. Severe bumps along the tailgate-end face and along the barrier pillar rib began soon after the longwall mining in Area 7 was initiated. According to Koehler (6), the excessive side abutment load from Area 6's gob was overriding the 150-ft-wide barrier pillar, causing the bump events. Experiences from Ruhr District, Germany, indicate that when the width of a coal block between an old roadway and the active longwall face is greater than 164 ft (50 m), coal bumps are unlikely (1).

Since it will exhaust significant coal reserve and additional pressure on mine operations for gate entry development, the independent panel layout is recommended only for those mines with a massive sandstone roof located immediately above the coal seam or within the caving zone, where total development and support cost for two gate entries with a barrier between them is less than those for multiple gate entries with very large abutment pillars.

CAES STUDY - CYPRUS WILLOW CREEK'S D SEAM MINE

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Table 5 Pillar Stability Factors vs. Coal Bump Potential in Pocahontas No. 3 Seam

Center abutment pillar size	Depth of cover	S_p	SF_t	TG Pillar bumps	TG-end face bumps	Width of each pillar
(ft)	(ft)	(psi)				(ft)
80 × 80	2,200	5,004	0.28	frequent	occasional	30, 80, 30
130 × 180	2,200	8,683	0.64	no	rare	30, 130, 30

The D seam at Willow Creek has a thickness ranging from 8 to 23 ft and a depth of cover ranging from 1,500 to 3,000 ft in the areas of interest. The D seam coal has an average strength of 3,606 psi and may fail violently when the applied stress exceeds 4,000 psi. The D seam is a composite seam with one to four layers of coal with different strength and structure. Its roof lithology is very similar to that of Castlegate No. 5 mine in the same area. The immediate roof mainly consists of thinly interbedded mudstone and siltstone. However, the roof lithology is highly variable, with sandstone or shale channels. No massive sandstone members present within 650 ft above the seam. The Castlegate sandstone with a thickness of about 500 ft is located at a distance of ranging from 650 ft to 763 ft above the seam in the longwall mining area. The lower portion of the Castlegate sandstone with a thickness of 350 ft is massive and strong. The immediate floor primarily is mudstone and siltstone. The predominant face cleat in the coal seam and the primary joints in the roof run N60°W, which matches the primary trend of fractures in the Castlegate sandstone. The secondary joints in the roof run N75°E.

The high strength of the coal indicate a high bounce or bump potential in some local areas where the coal seam is less-structured and sandstone channels in the roof are present. However, due to the absence of massive and strong roof and floor members in the vicinity of the D seam, the potential and severity of coal bumps can be minimized if the panel orientation encourage the overburden strata, to break along the gobline and the longwall chain pillars can yield before they are exposed to the side abutment load.

Panel Orientation

Since no massive and strong roof members are present in the vicinity of the seam, significant roof overhang behind the face supports is unlikely. However, the ability of the Castlegate sandstone's breakage along the gobline after two or three panels are mined may have a significant impact on the tailgate-end face bumps. Therefore, the longwall panel is orientated in a small angle (15°) to the primary joints to favor the Castlegate sandstone's breakage along the gobline, as illustrated in Fig. 1.

Yield Pillar Design

Due to the absence of massive and strong sandstone members in the vicinity of the coal seam, a purely yield pillar system was designed for and has been used at Willow Creek's D seam. Considering the disadvantage of the two-yeild pillar system for roof control, one-yeild pillar with a size of 28×80 ft between

panels was originally proposed with the following parameters:

1. $S_p = 1,912 \text{ psi} < 4,000 \text{ psi}$,
2. $SF_t = 0.12 \leq 0.18$,
3. $R_y = 1.2 \leq 1.5$, and
4. $R_H = 3.1 \leq 5.0$.

Stress Monitoring Program and Discussions of the Results

To verify the effectiveness of the proposed mine layouts and pillar design, a stress monitoring program has been implemented at Willow Creek, as illustrated in Fig. 2. Line 3 is located 750 ft outby the D-1's setup room on the headgate side, where Borehole Pressure Cells (BPCs) are used to measure the relative stress changes in the headgate pillar, D-1 and D-2 panels. Its purposes are to monitor the yield pillar's behavior, and peak stress and yield zone depth ahead of the faceline and into both panels. The BPCs in Line 4 are used to monitor the side abutment load distribution in the barrier pillar parallel to the D-1 panel's setup room. Its purpose is to verify if the designed barrier pillar width is adequate to isolate the impact of the side abutment load on the back-end bleeders and thus to provide valuable information for future barrier pillar design.

A significant yielding had occurred in the lower side (1,719 psi drop) of the pillar rib due to the 10-15% seam dip effect. The pillar center (514 psi drop) and upper portion (244 psi drop) along the headgate entry were slightly yielding before the face started. As illustrated in Fig. 3, when the face was 350 ft away from Line 3, the front abutment load had no impact on the pillar stress. The first peak stress of about 3,000 psi at the pillar center was achieved when the face was 60 ft inby. After that the center core started yielding and the stress dropped from about 3,000 psi to less than 500 psi. The yield pillar residual strength started increasing due to the compaction effect under the side abutment load. It can be seen from Fig. 3 that the yield pillar did behave as expected when it was exposed to the front abutment load and fully yielded without bumps before it was exposed to the side abutment load. Therefore, it can be concluded that this pillar will be most likely bump-free when the D-2's face approaches.

The BPCs' data inside D-1 panel indicate that the maximum peak stress (65 ft from the headgate panel rib) can reach as high as 3.4 times the original vertical stress induced by the overburden (Fig. 4). It can be seen from Fig. 4 that the coal seam started yielding from 6 (20 ft and 65 ft from the headgate panel rib) to 9 ft (10 ft and 35 ft from the headgate panel rib) ahead of the longwall face. The low peak pressure and early pressure drop of the BPC at 10 ft from the headgate panel rib are due to the yield

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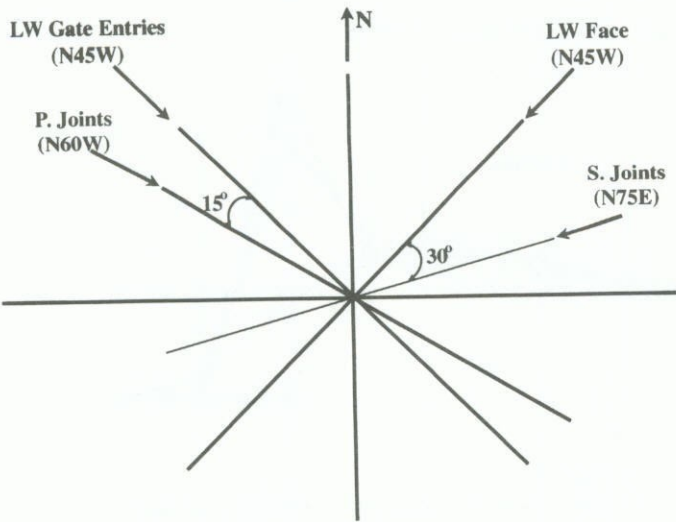


Fig. 1 Minbe Layouts vs. Primary Joints

zone development along the panel ribline. The low peak pressure and early pressure of the BPC at 35 ft may be caused by the local weakness of the coal seam. To establish the relationship between the face bump events and the stress concentration ahead of the longwall face, NSA's RockVision system has also been installed at Willow Creek to generate stress tomograms of the panel during longwall retreat.

As indicated in Fig. 5, the coal within 10 ft from the panel rib started yielding when the face was 50 ft inby and took load again when the face was about 9 ft inby. The BPC at 35 ft from the panel rib started yielding when the face was 300 ft outby. It may be concluded that the yield zone depth will be greater than 35 ft and less than 65 ft when the D-2 face approaches. A deeper yield zone may reduce the potential of the tailgate-end face bumps during the D-2 panel's retreat.

Based on the BPCs' data from Line 3, it can be concluded that

Fig. 2 Layout of BPCs @ Line 3 and Line 4

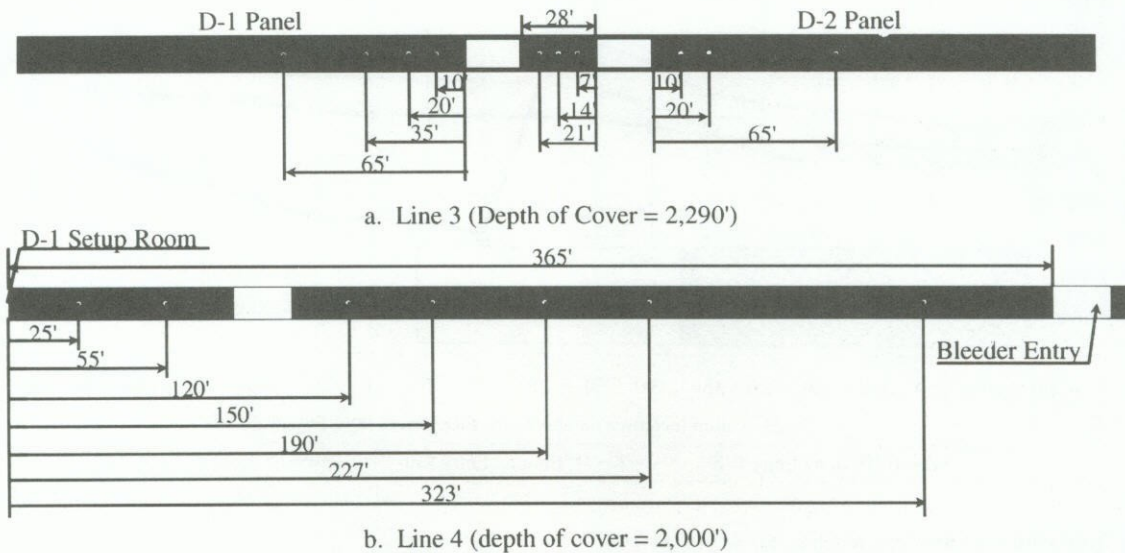
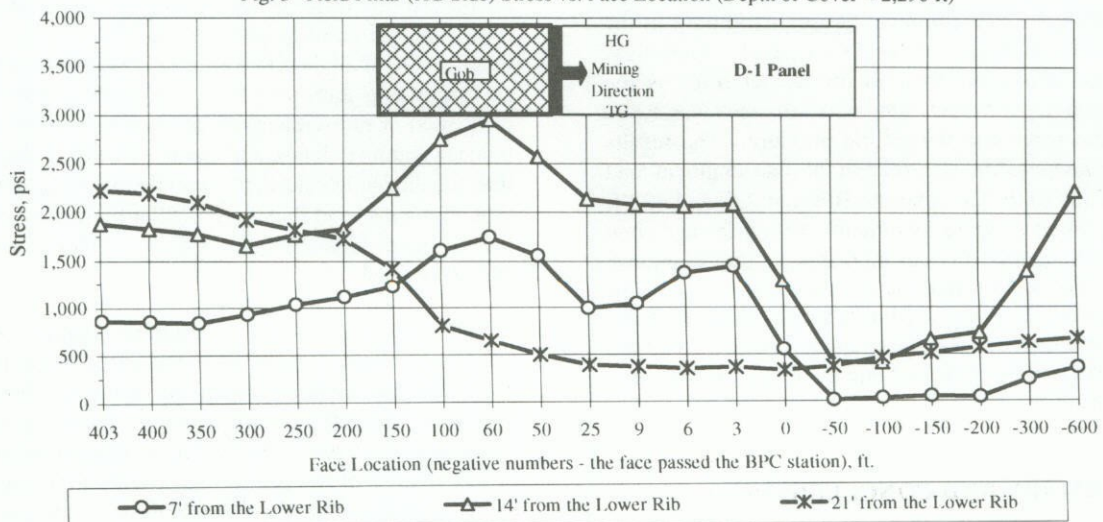


Fig. 3 Yield Pillar (HG Side) Stress vs. Face Location (Depth of Cover = 2,290 ft)



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Fig. 4 D-1 Panel Stress vs. Face Location (Depth of Cover = 2,290 ft)

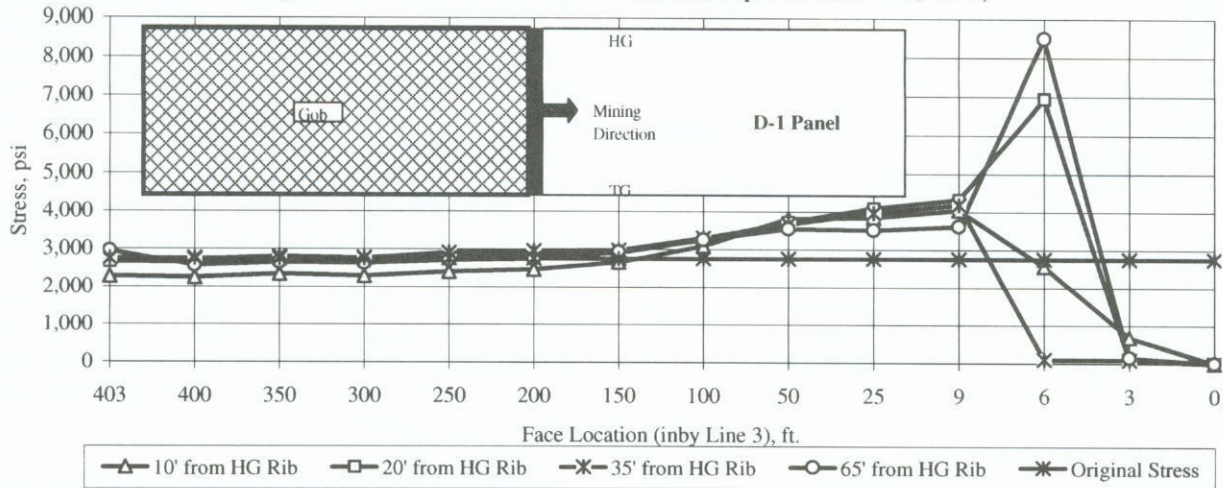
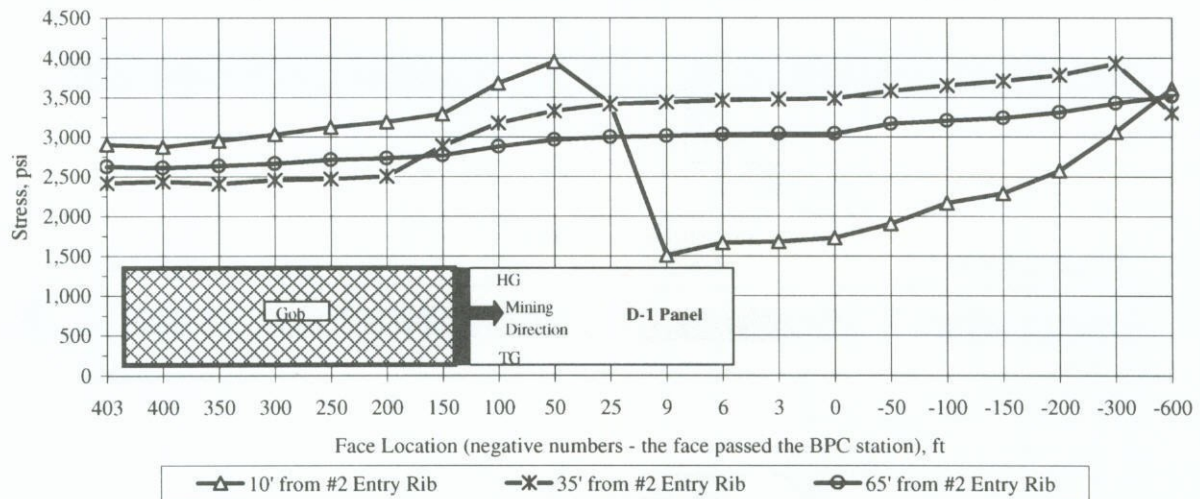


Fig. 5 D-2 Panel Stress vs. D-1 Face Location (Depth of Cover = 2,290 ft)



the zone of the front abutment load can reach as far as 300 to 350 outby the longwall face.

Figure 6 shows the side abutment stress distribution in the barrier pillar at the back-end of the D-1's panel. The stress increment or side abutment stress in the barrier pillar is the difference between the stress recorded when the face was 1,880 ft outby the setup room and the setting pressure. The results indicate that the obvious influence zone of the side abutment load is within 227 ft due to the fact that the BPCs at 227 and 323 ft have the similar value of stress increment. This influence zone may increase as the depth of cover increases or after one more panel is mined. It is obvious that the current panel with a width of 800 ft is subcritical and the overburden strata have not been fully affected. Therefore, the magnitude and influence zone of the side abutment load will be observed and quantified as the mine matures in the future.

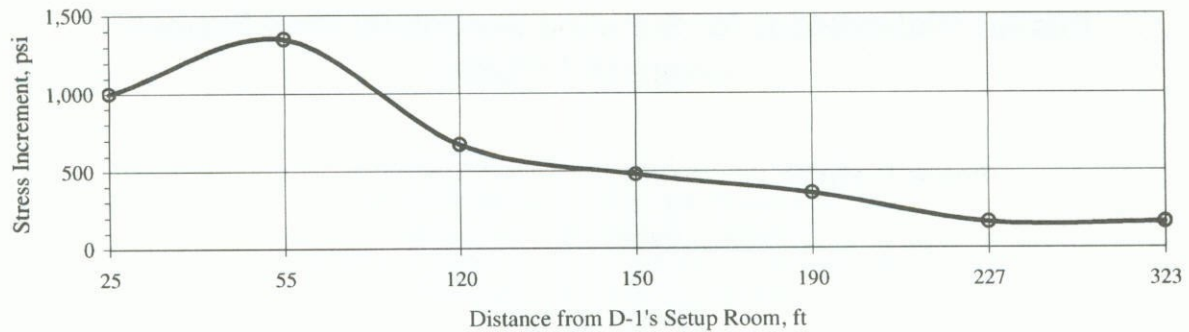
SUMMARY AND CONCLUSIONS

In this study, three criteria have been introduced to design the yield pillar system for bump-prone longwall mine mines. They are the stability factor under the tailgate load, width-to-yield zone ratio, and width-to-height ratio of the pillar. It is strongly believed that if all the three criteria are equal to or less than their corresponding threshold value, the yield pillar will yield in a controlled or non-violent manner when exposed to the mining-induced abutment loads, and that if all the three factors are greater than the threshold values, the intended yield pillar may yield in a violent manner and thus severe pillar bumps are expected. For the abutment pillar design, a stability factor of greater than 0.6 should be used.

The longwall panel orientation should encourage the overburden strata to cave or break either along the gobline or behind the face supports, depending on the roof lithology and the proposed longwall gate entry system. For the bump-prone coal seams where a massive and strong sandstone member is present in the vicinity of the coal seam or within the caving zone, the panel orientation should encourage the roof strata to cave or break

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Fig. 6 Stress Distribution in the Barrier Pillar (Depth of Cover = 2,000 ft)



behind the face supports to prevent high stress concentration along the faceline. Under this scenario, a yield-abutment, a yield-abutment-yield, or a yield-barrier-yield pillar system is recommended to prevent tailgate-face corner bumps. For the bump-prone seams where no massive and strong sandstone members are present in the vicinity of the coal seam or within the caving zone, the panel orientation should favor the breakage of the roof strata along the gobline to minimize the potential of tailgate-face corner bumps.

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Design Methodology for Standing Secondary Roof Support in Longwall Tailgates

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ABSTRACT

Maintaining ground stability in the gate roads, particularly the tailgate, has always been critical to the success of longwall mining, both in terms of safety and productivity. Several new support technologies have been developed in recent years to replace conventional wood and concrete cribbing for secondary roof support. Since their performance characteristics are unique, the best practices that have been developed with conventional wood cribbing may not be applicable for these alternative support technologies. Therefore, with so many options to consider and the importance of achieving adequate ground control at minimal cost, the trial and error approach to longwall gate road support is no longer prudent. This paper discusses a design methodology for standing secondary tailgate supports. This design technique requires in-mine measurements of tailgate support loading and convergence to establish a tailgate ground reaction behavior based on the support and strata interaction. The methodology uses the performance characteristics generated in the National Institute of Occupational Safety and Health (NIOSH) Mine Roof Simulator (MRS) to match the stiffness and load characteristics of various supports to the measured ground reaction behavior. It can be used to determine the appropriate application of alternative roof support systems or to design in-mine trials such that a fair and equitable comparison of different support systems can be made. A case study of the methodology at a western Pennsylvania mine site is presented in the paper, including a comparison of four alternative support technologies to conventional wood and concrete cribbing historically used at this particular mine.

INTRODUCTION

The longwall tailgate support system must flawlessly control the tailgate ground conditions. Safety considerations, especially the limited escape routes from a longwall face, demand that the tailgate entry be a negotiable travelway. The location of the face electrics, support equipment, and belt line in the head gate entries dictate that the tailgate may be the only

option for mine workers to escape from the face in the event of an emergency. A recent example is the longwall gob fire which occurred at a mine in Utah this past year (1998) in which several miners evacuated through the tailgate entry to safety. In addition to the emergency travelway requirements, inadequate tailgate support that results in poor ground control and blocked tailgates due to roof falls can severely retard or halt production. The heavy reliance by mines on the longwall production for survival dictates that loss of production for protracted time periods cannot be tolerated. Ventilation is another issue that depends on proper tailgate support. As the panel lengths continue to increase, excessive closure or restriction of the tailgate entry by the deformation and/or density of the standing support can be problematic and potentially unacceptable. In gassy mines, it also may be required that the tailgate be kept open in by the longwall face in order to establish effective bleeder ventilation of the tailgate area. Another important issue to consider is the material handling aspects of tailgate supports. Therefore, the onus is on the mine engineers to design a support system that maintains adequate control of the tailgate ground conditions at all times and with minimal ventilation resistance and material handling considerations.

Historically, the importance of ground control has led to very conservative applications of tailgate support. Most mines use conventional wood crib structures. When properly designed, conventional wood cribs provide effective ground control in most longwall tailgate entries, and in the past have been cheap enough that mines could afford to use a high density of cribs at relatively little cost. However, increasing timber costs, inconsistent timber quality that led to poor crib performance, and inadequate supplies of timber for western mines have reduced the advantages of conventional wood cribbing, and have encouraged many mines to consider other options for tailgate support. All of these factors have prompted the support manufacturers to develop innovative support technologies as alternatives to conventional wood cribbing.

Today, there are several alternative support technologies that have been developed by various support manufacturers and

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safety tested at the NIOSH Safety structures Testing Laboratory, whereby their performance characteristics have been determined. With increasing pressures to reduce support costs while ensuring the safety of the mine workers, mine operators more than ever before are looking for ways to optimize longwall tailgate support. Since the performance characteristics of these supports are unique, the best practices that have been developed largely through trial and error with conventional wood cribbing may not be applicable for these alternative support technologies. In addition, these new supports have limitations, which if not properly recognized, can lead to poor support application and inadequate tailgate ground control.

Therefore, with so many options to consider and the importance of achieving proper ground control at minimal cost, the trial and error approach to longwall tailgate support can be costly and indecisive. This paper proposes a design philosophy for longwall tailgate secondary support whereby the ground reaction behavior, as a function of the support load density and stiffness of the support system, is determined by measurement of underground support loading and roof-to-floor convergence. The goal of support design is then to optimize the use of the support by designing to the ground reaction curve, and controlling the convergence to acceptable limits that will ensure stability of the mine roof. This approach will allow mine operators to maximize the use of alternative support systems while ensuring the safety of the mine workers by avoiding risky and time consuming trial-and-error assessments of support technologies. In addition, it will provide MSHA with a means to assess various support systems on an equivalent basis when approving roof control plans. A case study is included in the paper which relates the use of the proposed design methodology to a trial of alternative support technologies at a western Pennsylvania longwall mine operating in the Pittsburgh coal seam.

TERMINOLOGY

Secondary Support - Secondary support is that which is intentionally added to assist the primary support (roof bolts) in controlling the mine roof when it is known that additional roof loading will occur. In longwall mining, it is installed in advance of the abutment loading. Secondary support is not to be confused with supplemental support which is additional support, that is installed beyond the primary and secondary support either for insurance purposes or in response to unanticipated poor ground conditions.

Ground Reaction Curve - A concept of how the ground reacts to the presence of a newly created opening. Specifically, as the ground deforms and sheds load to other structures, there will be a proportional decrease in roof loading and required support capacity to maintain equilibrium of the mine roof and floor.

Critical Convergence - In relation to the ground reaction curve, the critical convergence is where failure of the ground is inevitable, and the full weight of the failed rock mass above the mine entry must be supported by the secondary roof system to

prevent a roof fall from occurring. The goal of secondary support design is to prevent this convergence from occurring.

Support Load Density - The load carrying capacity of an installed support system per unit area of exposed ground (tons/sq ft.) at a particular ground deformation.

Minimal Acceptable Support Load Density - Lowest load density of support that should be provided since a lower support load density will allow convergence greater than the critical convergence and thereby allow failure of the roof rock to occur that may lead to a roof fall.

Support Density - Term typically used to refer to the number of supports per unit area. Should not be confused with support load density, which is the capacity of the support system per unit area as a function of convergence.

Support Stiffness - A measure of how quickly a support develops its load carrying capacity in relation to convergence. For an individual support, stiffness can be determined from the slope of the load-deformation performance curve. "Softer" supports have a flatter slope than "stiffer supports" when plotted to the same load-displacement scale. Softer supports require more convergence to develop an equivalent load carrying capacity than stiffer supports.

Support System Stiffness - The resistance to load of a group of supports. The system stiffness is the sum of the individual support stiffness. Hence, a double row of supports would have twice the system stiffness of a single row of the same supports.

KEY FACTORS TO CONSIDER IN GATE ROAD SUPPORT

A design philosophy for standing supports must be based on the interaction of the support with the surrounding rock mass. The question that needs to be addressed to formulate a design methodology is to determine to what extent the support system is controlling the ground. To do this it is necessary to understand both the ground behavior and the characteristics of the individual support and the support system.

Understanding the Function of the Support System

Obviously, the support system is employed to prevent roof falls. How this is accomplished is the important issue. While secondary supports provide the last means of support in the event there is roof failure above the bolted horizon, the primary function of the secondary support system is to assist the primary support system in maintaining the integrity of the immediate roof. As the ground deforms by the creation of an opening due to mining, it gradually sheds load to the surrounding mine structures, which in the case of longwall mining are the gate road pillars and longwall panel. Secondary support must be placed in sufficient time and develop sufficient capacity to bring deformation of the ground into equilibrium before a critical deformation is reached at which point failure of the ground is

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inevitable. Otherwise, the secondary support will be required to carry the entire dead weight of the detached rock mass to prevent a roof fall from occurring. This embodies a fundamental concept in rock mechanics known as the "ground reaction curve" (1).

In longwall mining, the tailgate entry is subjected to three phases of loading and equilibrium. Each will have a distinct ground reaction curve. The first phase occurs on development where the mine opening is created and the primary support (roof bolts) are installed. Relatively little ground movement occurs during this phase since the development loads are small and the primary support is sufficient to provide equilibrium. The next phase is adjacent panel mining. The future tailgate is subjected to side abutment loading, and while secondary support is typically installed to ensure that equilibrium of the rock mass is obtained, the convergence is typically minimized by the load density of the support. The final phase of tailgate behavior is where the active tailgate is subjected to front abutment loading from the panel extraction. It is this phase where the secondary supports play their most important role in preserving the stability of the entry. A hypothetical tailgate ground reaction curve is shown in figure 1. It should be noted that the ground reaction curve will be a function of the several factors in addition to the load density of the support system. These include the geology, roof spans, vertical as well as horizontal stress around the opening, and some time dependent factors such as creep. Hence, the ground reaction curve is generally unique to a specific mine and can change within the mine as these factors change. From the perspective of secondary support design, it is important that the ground reaction curve be examined under the worst case load conditions where ground control is required. Since the ground reaction is dependent upon roof span, a different ground reaction behavior will typically be observed in intersections compared to the non-intersection areas of the entry. Hence, the support design must be altered for the intersections to accommodate this difference in ground reaction.

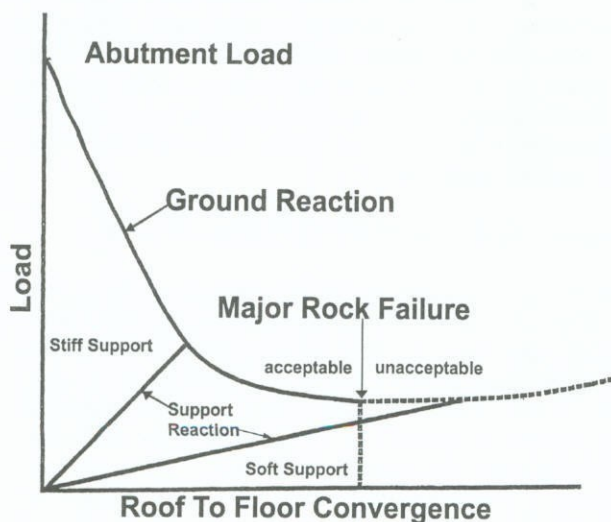


Figure 1. Ground reaction curve concept.

It is seen from the hypothetical tailgate ground reaction curve (figure 1) that if the goal is to prevent convergence completely, then the full abutment load must be resisted by the support system. This resistance for all practical purposes cannot be obtained by the installation of secondary support since the required capacity would need to be equivalent to that of all the coal that was removed by the mining process. The support capacity required to achieve equilibrium is reduced as the deformation increases, since the roof is shedding load to other mine structures as it deforms. In other words, by allowing the roof to deform and shed some load to the coal pillars and longwall panel, less support capacity is required since the roof load is decreased. Hence, the lowest required support capacity would be one that is developed just before the critical roof deformation occurs where failure of the immediate roof is fast approaching. However, designing to this lower limit of support capacity leaves no margin of error in the event that load conditions worsen. Also, it can be seen that if a support system is too soft (develops load carrying capacity too slowly), equilibrium of the mine roof will never be achieved and failure of the roof will be inevitable.

In summary, since most standing supports are passive supports, convergence must take place before sufficient support loads are developed to provide equilibrium. While some supports have active loading capability, the magnitude of active loading is not sufficient to achieve equilibrium or the active loading cannot be maintained indefinitely. The supports must be loaded in compression to provide the required load resisting forces to achieve equilibrium of the rock mass. The amount of convergence required to produce equilibrium is then a function of the stiffness of the support system. Equilibrium will be achieved at less displacement for a stiffer support system since the support resistance to roof loading will develop quicker (at less displacement) compared to a softer support system. Hence, as the support load resistance (load density) of a support system increases, the convergence at which equilibrium is attained will decrease. If too much convergence is permitted to occur by utilizing too soft of a support system, failure of the rock mass (mine roof) will be inevitable. Hence, the goal of support design is to provide sufficient support stiffness to ensure that the required support capacity to achieve equilibrium of the rock mass occurs before the rock mass deforms to the point of failure. However, a prudent mine engineer would ensure that sufficient support capacity is developed long before the critical convergence is reached. Since minimizing convergence is achieved by increasing the support capacity (load density) and generally the cost of support, the goal of optimizing the support selection is not to install more support than is necessary to provide a reasonable margin of safety to prevent roof failure.

Understanding the Performance Characteristics of the Support

The load-displacement characteristics of numerous roof support technologies have been determined at the NIOSH Safety Structures Testing Laboratory through full scale testing in the unique Mine Roof Simulator load frame (2). The load-displacement response of these various support systems are

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documented in figures 2 through 7, grouped by the following description of the support type: (a) conventional wood cribbing (figure 2), (b) engineered wood crib supports (figure 3), (c) conventional and engineered timber post supports (figure 4), (d) non-yielding concrete supports (figure 5), (d) deformable concrete supports (figure 6), and (e) yielding steel supports (figure 7).

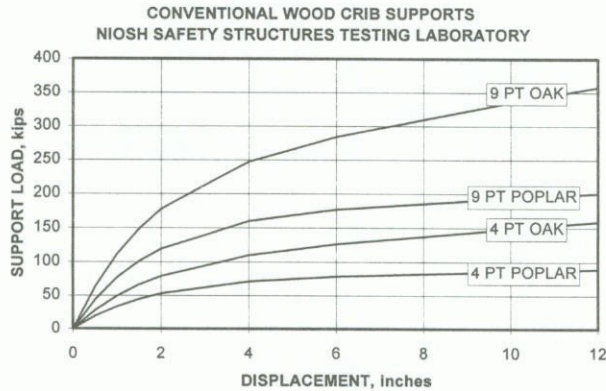


Figure 2. Load-displacement performance data for conventional wood crib supports.

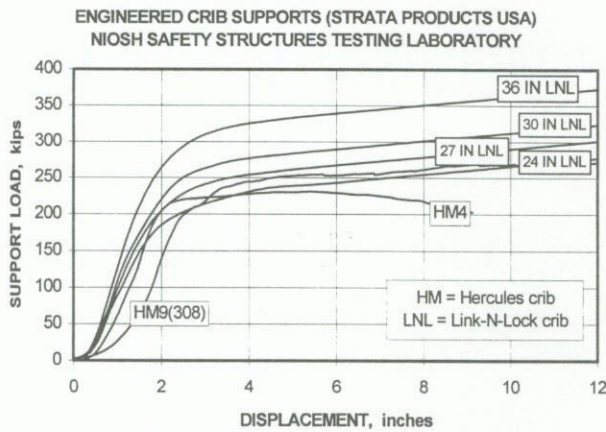


Figure 3. Load displacement performance data for engineered wood crib supports.

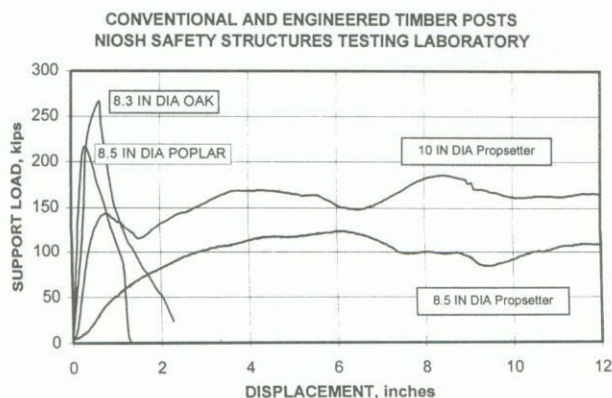


Figure 4. Load-displacement performance data for conventional and engineered timber props.

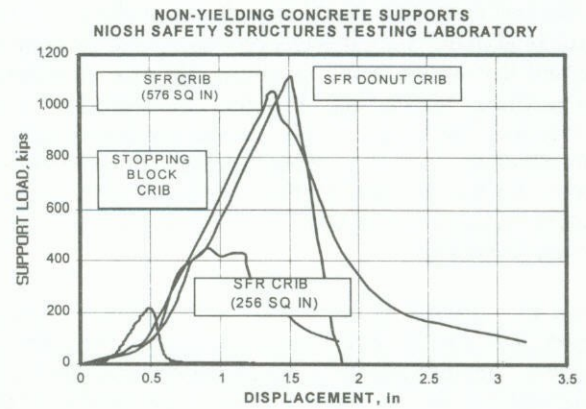


Figure 5. Load-displacement performance data for non-yielding concrete support.

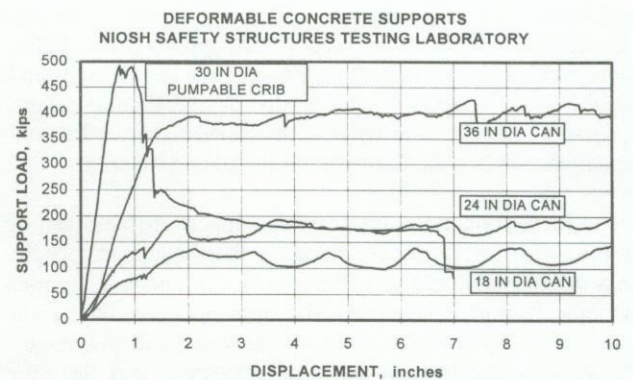


Figure 6. Load-displacement data for deformable concrete supports.

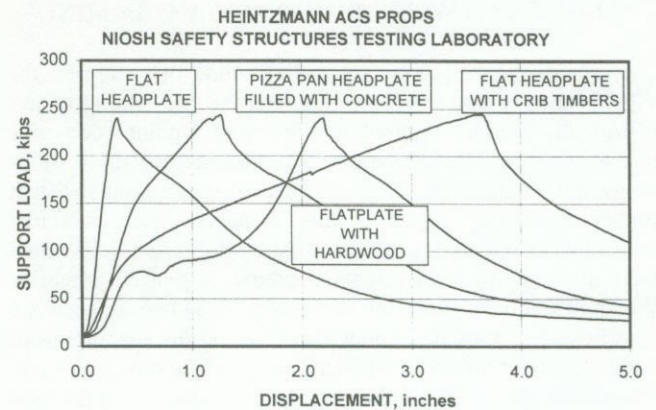


Figure 7. Load-displacement performance data for yielding steel supports.

As previously indicated, all secondary supports must be loaded in compression to produce the required capacity to achieve equilibrium of the mine roof. In other words, the roof has to move down before the standing support develops

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sufficient load carrying capacity to achieve equilibrium of the mine roof and floor. Since it is this very downward movement of the roof that we are trying to control, the most important design parameter for standing supports is the stiffness of the support system. Stiffness is simply a measure of how quickly a support develops its load carrying capability in response to convergence of the mine roof and floor. Stiffer supports develop equivalent load carrying capacity with less displacement than softer support systems.

While the stiffness of the support is the primary design parameter, it is not the only issue to consider in the support application. Another important design parameter is the load carrying stability of the support. More specifically, it is important to know how well the support can sustain its load carrying capability as a function of convergence. Stiff supports, such as the non-yielding concrete supports (figure 5), which develop load carrying capacity quickly but fail at little convergence are not practical in many longwall tailgate applications. In order to keep such supports from failing prematurely, a large number of supports must be installed, such that the roof loading is sufficiently shared among several supports, while achieving a high enough load density to keep the convergence below the failure point for any one support.

The ideal support is one that the support stiffness can be controlled through the support design so that the support capacity at a given displacement can be engineered to match the ground reaction behavior. Furthermore, the ideal support would be able to maintain this capacity through a wide range of displacements without shedding load or failing prematurely, and therefore provide a margin of safety in the event that ground conditions worsen unexpectedly.

MEASUREMENT OF THE GROUND REACTION CURVE AT A WESTERN PENNSYLVANIA MINE

Studies were made at a longwall mine operating in the Pittsburgh coal seam in western Pennsylvania. The mine has historically used a staggered double row of 4-point wood cribs on a 4 ft spacing between supports in adjacent rows (figure 8) for longwall tailgate support. Due to uncertainties and inconsistent timber qualities, the mine also employed concrete cribs constructed from normal mine ventilation stopping blocks which were also employed in staggered double row arrangement as shown in figure 9. Since the stiffness of these two supports are significantly different (figures 2 and 5), and the supports were employed in a similar arrangement, a ground reaction curve was determined for this particular mine by measurement of the load on individual supports and the associated roof-to-floor convergence in the vicinity of the supports. The results are shown in figure 10.

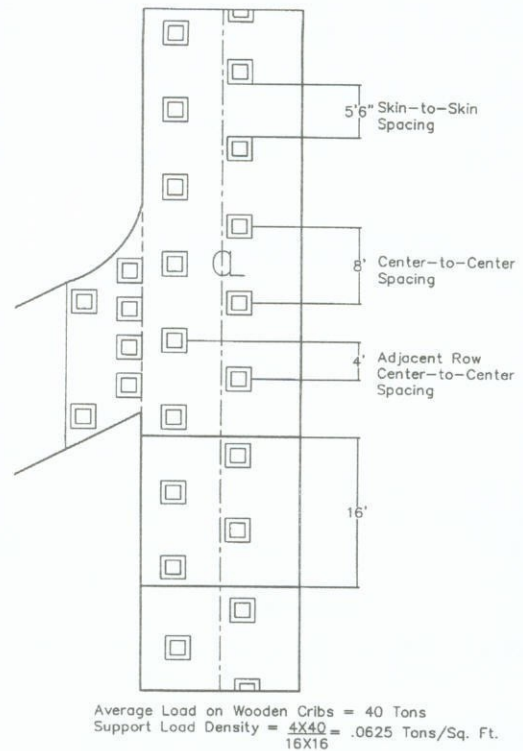


Figure 8. Arrangement of 4-point wood cribs in a western Pennsylvania mine.

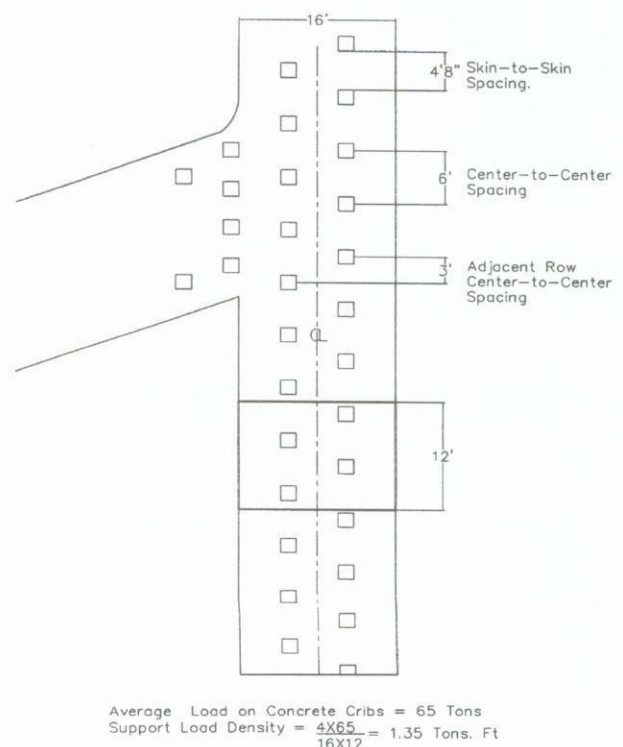


Figure 9. Arrangement of stopping block concrete crib support in a western Pennsylvania mine.

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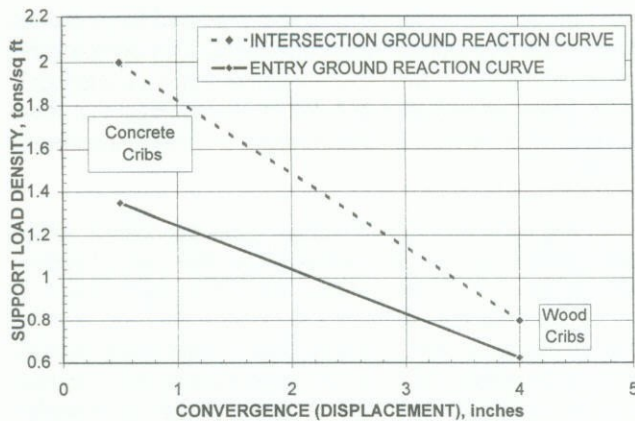


Figure 10. Measured ground reaction curve for western Pennsylvania coal mine.

First, it is important to note where these measurements were obtained. Since, the abutment loading changes dramatically in longwall mining as the face approaches, the ground reaction curve should always be established at the most severe load condition, which generally will be just behind the tailgate shield. In this case, the mine wanted to maintain sufficient control of the tailgate entry to maintain a ventilation air way back to the next open cross cut. Hence, measurements of support loading were obtained to distances of 50 to 100 ft inby the face. For reasons previously explained, a different ground reaction was measured through the intersections compared to the entries (figure 10). These measurements were also made under the deepest cover, again to establish the worst load condition.

The following analysis applies to the entries at positions where there was no influence of the crosscuts. The load on the 4-point wood cribs was estimated at 40 tons with a roof-to-floor convergence of 4 inches. Qualitatively, it was noted that the integrity of the immediate roof was showing signs of deterioration at this convergence, which suggests that 4 inches is approaching the critical convergence where failure of the roof is inevitable. Some of the wood cribs in the area were also showing signs of premature failure, probably due to poor timber quality. Conversely, the convergence in the concrete crib supported area was only 0.5 inch and the measured load on the cribs was 62 pct greater at 65 tons per crib. These support loads are converted into a support system load density of 0.625 tons per square foot for the wood cribs (equation 1) and 1.35 tons per square foot for the concrete stopping block cribs (equation 2).

$$\begin{aligned} \text{LOAD DENSITY (Wood Cribs)} &= \frac{4 \text{ Cribs} \times 40 \text{ tons/Crib}}{16 \text{ ft} \times 16 \text{ ft}} \\ &= 0.625 \text{ tons/sq.ft.} \end{aligned} \quad (1)$$

$$\begin{aligned} \text{LOAD DENSITY (Concrete Cribs)} &= \frac{4 \text{ Cribs} \times 65 \text{ tons/Crib}}{16 \text{ ft} \times 12 \text{ ft}} \\ &= 1.35 \text{ tons/sq.ft.} \end{aligned} \quad (2)$$

These two data points are then used to establish the ground reaction curve for the tailgate entry inby the longwall face as depicted in figure 10. Since measurements were made on only these two support systems, a linear approximation to the ground reaction curve is made using these data as end points with a straight line connecting between them. This curve can then be used to determine the support load density required to control the convergence inby the longwall face from 0.5 to 4.0 inches. For example, if the goal is to limit the convergence to 2 inches, then drawing a line from 2 inches vertically upward until it intersects the ground reaction curve, and then drawing a line horizontally to the y axis, reveals that a support load density of 1.04 tons per sq ft must be provided. An algebraic solution to the problem can also be found by determining the slope and y-intercept for the ground reaction curve. Once the algebraic equation for the line is determined, the support load density at any displacement can be calculated.

$$\text{Load Density (Convergence)} = -0.2071 \times \text{Convergence} + 1.45$$

$$\begin{aligned} \text{Load Density (2 in convergence)} &= -0.2071 \times 2.0 + 1.45 \\ &= 1.04 \text{ tons / sq ft} \end{aligned} \quad (3)$$

The next requirement is to transform the required support load density into a support system design. Support load density is determined primarily by two factors: (1) the stiffness of each support and (2) the spacing of the supports. Continuing with our example, if we want to increase the support load density of the 4-point wood crib support system from its current 0.625 tons per sq. ft. to 1.04 tons per sq. ft. in order to reduce the convergence in the entry from 4 inches to 2 inches, then we would need to decrease the spacing of the wood cribs from the current 4 ft spacing, but the question is by how much? The required center-to-center spacing to provide a support load density of 1.04 tons/sq. ft. can be determined by first identifying the load capacity of a wood crib at 2 inches of convergence, which is found from the performance data developed from the laboratory tests conducted in the Mine Roof Simulator (figure 2). As shown in figure 2, the capacity of a 4-point wood crib is 27 tons at 2 inches of displacement. The spacing of a single row of cribs is then determined by dividing this capacity by the product of the required support load density and the entry width. As the following analysis shows, the required spacing of 4 point wood cribs to achieve a 1.04 tons per sq. ft. support load density is 1.6 ft. Such a tight spacing can only be achieved through a staggered double row arrangement, where the center-to-center spacing in each row is 3.2 ft.

$$\begin{aligned} \text{Spacing (2 inches convergence)} &= \frac{\text{Capacity (2 inches)}}{\text{Load Density} \times \text{Entry width}} \\ &= \frac{27}{1.04 \times 16} = 1.6 \text{ ft} \end{aligned} \quad (4)$$

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DESIGN METHODOLOGY FOR LONGWALL TAILGATES

The concept examined for the western Pennsylvania mine can be generalized and employed at any mine to optimize secondary support design and application. The first step in the design process is to determine the ground reaction behavior. This can be done by installing at least two, and preferably three, support systems of *varying* support system stiffness, and measuring their loading and corresponding convergence of the mine roof and floor. It is important that these supports cover a wide range of stiffness to provide a *full picture* of the ground reaction behavior. As part of this process, an effort should also be made to determine the critical roof-to-floor convergence where roof failure occurs, since this will be a critical design value for the support design. However, in order to do this, a low density support system must be installed that will allow considerable convergence to occur. Since this poses a risk of inadequate ground control, precautions should be taken. One possibility is to set additional wood cribs through this trial zone with the top layer removed such that they could be reinstalled to provide additional support if necessary, or in the worst case provide ground support after the roof deforms (5-6 inches) to the cribs if they cannot be topped off.

Altering the support system stiffness can be done several ways. One way is to utilize the same support and same spacing down the entry, but increase the number of rows of support across the mine entry from one to two to three, which would proportionally increase the system stiffness by the same factors. Another way is to keep the spacing and number of rows of support constant, but use supports of varying stiffness. This will eliminate the impact of both span and roof coverage issues, which can be limiting factors in the support placement (load deformation) strategies. For example, using conventional wood cribs, the support construction could be varied from a 4-point to a 9-point to a 16-point crib. The support load density would increase in direct proportion to the increase in support stiffness. Adjusting the support spacing down the entry can also be considered. The support load density is proportionally increased as the spacing is decreased. Care should be taken to avoid excessive spacing that will cause span related problems. A good rule of thumb is that the support spacing should not exceed half the entry width. The load bearing area (quality and extent of the contact area) of the support also an important factor to consider. The pressure exerted by any support should not exceed the strength of the mine roof or floor.

Ideally, the loads should be measured underground on the various support systems. Support load measurements are typically made through a hydraulic flat jack. This will be more difficult to do on some support designs than others. Supports with a single contact area, such as concrete cribs are easier to work with than something like a conventional wood crib which has multiple load paths. Theoretically, if loads cannot be measured, they can be estimated directly from laboratory load-displacement data provided convergence (support displacement) measurements are made. However, this estimate may not be accurate for materials such as wood where creep can occur and

distort the approximation of load that produced the measured displacement. Measurements of convergence are essential to this design methodology. Roof-to-floor convergence measurements can be made in several ways, but it is important that these measurements correlate to displacements induced in the support structure.

Readings of load vs displacement (convergence) should be made under the most severe load conditions that occur. For most longwall mines, this will be at the back of the tailgate shield. If it is critical that the tailgate be kept open in by the face for ventilation reasons, then ideally the load - displacement readings should be made in by the face. Of course, this may not be as easy to do, but it is important to realize that the design methodology assumes worst case load conditions, and safety factors will need to be employed if measurements are taken under less severe load conditions. Another good rule of thumb to follow is that the face should be retreated a distance approximating the width of the face before the full ground reaction behavior is established. In other words, if the face width is 1,000 feet, then the face should be retreated at least 1,000 feet before the ground reaction behavior is measured. Of course, the first requirement for any tailgate design is proper pillar design. While in theory a ground reaction behavior can be determined for any pillar design and roof geology, the methodology proposed here assumes that the pillar design falls above the ALPS design line for a given CMRR (3).

The load-displacement data is then used to generate a plot of support load density as a function of convergence (ground reaction curve). Each support type with different stiffness represents one data point on the ground reaction curve. The support load density is determined as the measured load in the support at the observed convergence times the number of supports per unit area of mine entry (equation 5). For a single row of supports employed on a constant center-to-center spacing, the support load density can be determined by equation 6.

$$\text{Support Load Density} = \frac{\text{Number Supports} \times \text{Support Load}}{\text{Area Of Support Coverage}} \quad (5)$$

$$\text{Support Load Density} = \frac{\text{Support Load}}{\text{Center - To - Center Spacing} \times \text{Entry Width}} \quad (6)$$

Once the ground reaction curve is developed, the center-to-center spacing (down the mine entry) of alternative support systems employed in a single row arrangement required to achieve ground control (equilibrium) at a desired convergence can be determined from equation 7. The center-to-center spacing of a double row of supports is simply twice that of a single row arrangement.

$$\text{Spacing (displacement)} = \frac{\text{Capacity (displacement)}}{\text{Load Density} \times \text{Entry Width}} \quad (7)$$

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where Spacing(displacement) = Center-to-center spacing in feet of a single row of supports;

Capacity (displacement) = Individual support capacity in tons at a specified displacement (obtained from laboratory performance data) equal to the desired convergence control;

Support load density = Support load density in tons per sq. ft. at the required convergence (obtained from ground reaction curve);

Entry Width = Width of the entry in feet.

This design methodology is a valuable tool in optimizing the utilization of standing secondary roof support technology. However, as previously described it is still up to the mining engineer to decide how close to the critical convergence he/she wants to operate based on their knowledge of the particular ground conditions. A margin of safety is provided by designing for a convergence that is less than the critical convergence (minimal acceptable support load density). In order to make equivalent comparisons of alternative support systems, a safety factor can be quantified by comparing the design support load density to the minimal acceptable support load density which will be representative of the maximum allowable (critical) convergence. This is referred to as the ground reaction curve (GRC) safety factor (equation 8).

$$\text{GRC Safety Factor} = \frac{\text{Design Load Density}}{\text{Minimal Acceptable Load Density}} \quad (8)$$

Another factor to consider is whether the support is being fully loaded and how much reserve capacity is left in the support at the design load. In the event that load conditions worsen beyond expectations, this reserve support capacity may be needed to support the mine roof. If the support characteristic is such that the support sheds load quickly after reaching its peak load, such as the non-yielding concrete supports (figure 5), then consideration must be given to avoid designing near the peak loading capability of the support. A safety factor for the support can be defined based on the support loading at the required support load density in relation to the peak loading capability of the support (equation 9). Hence a safety factor of 1 indicates that there is no reserve capacity available and a safety factor of 2 indicates that the support is loaded to only 50 pct of its maximum support capacity.

$$\text{Safety Factor (Support)} = \frac{\text{Peak Load Capability}}{\text{Load at Installed Load Density}} \quad (9)$$

ALTERNATIVE SUPPORT STUDIES AT THE WESTERN PENNSYLVANIA MINE

Four different standing support systems and one cribless system in addition to conventional 4-point wood crib supports and concrete stopping block supports were installed in the longwall tailgate entry of this mine. The alternative standing

supports were: (1) Heintzmann Corporation's Alternative Crib Supports (ACS's); (2) HeiTech Corporation's Pumpable Crib Supports; (3) Strata Product's Propsetters; and (4) Burrell Mining Products "The Can". Cable trusses were used in the cribless area. The alternative supports were assigned sections of the tailgate in between the standard wood cribbing. This was done to ensure that there was no interaction between support installations, thereby allowing a fair and equal evaluation to be made (i.e., normal cover, no excessive roof or floor damage, normal geology).

Table 1 shows the installed spacing of the alternative support systems and the typical 4-point wood crib system and concrete stopping block crib system. The support load density of the alternative supports is calculated by matching system performance to the ground reaction curve. Essentially, this requires working backwards through the design methodology. The following steps can be used.

1. Pick an arbitrary convergence within the bounds of the ground reaction curve.
2. Determine the required support load density that matches the ground reaction curve for this convergence.
3. Identify the individual support capacity at this displacement from the laboratory performance data at this convergence.
4. Determine the support load density from equation 6.
5. If the support load density is greater (falls above the curve) than the required support load density - pick a lower convergence, or if the support load density is less (falls below the curve) than the required support load density - pick a higher convergence, and repeat steps 1 through 4 until the support load density matches the ground reaction curve.

An analysis of table 1 reveals that all four alternative support systems were installed with sufficient support load density to control the convergence well below the critical level of 4.0 inches that the 4-point wood cribs provided. The high safety factor utilized in these alternative support applications was to provide a margin of safety in anticipation of a tailgate horizontal stress concentration on the next panel.

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Table 1. Assessment of standing alternative support technologies utilized in study at a western Pennsylvania mine.

Support System	Installed Spacing, ft	Installed Load Density, tons/ft ²	Conv. control inches	Safety Factors		Observed Roof Condition	
				GRC ¹	Support ²	Outby face	Inby face
4-Point Wood Crib	8 ft (DR ³)	0.625	4.0	1.00	1.8	Good	Marginal
Concrete Stopping Block Crib	3 ft (DR)	1.35	0.5	2.16 (0 ⁴)	1.0	Good	Marginal
Heintzmann ACS	5.0 (DR)	1.20	1.24	1.92	2.5	Excellent	Good
HeiTech Pumpable Concrete Crib	9.2 (SR)	1.35	0.5	2.16	1.3	Excellent	Excellent
Strata Products Propsetter	4.0 (DR)	1.12	1.6	1.79	1.7	Excellent	Good
Burrell Mining Products Can Support	7.0 (DR)	1.19	1.25	1.90	1.8	Excellent	Excellent

¹Ground Reaction Curve safety factor is determined from equation 8 as the ratio of the installed support load density to the minimum allowable support load density.

²Support safety factor is determined from equation 9 as the ratio of the peak loading capability of the support to the load developed at the installed spacing.

³DR stands for double row of cribs. All double rows of supports were installed in a staggered fashion. The spacing here refers to the spacing of one row of supports. With the staggered arrangement, the spacing between adjacent supports of both rows is half of that of the individual row (see figure 8 or 9). SR refers to a single row arrangement.

⁴The roof condition was good prior to the failure of the support. Hence, the installed support load density actually dropped to zero once the support failed and accounted for the deterioration in the integrity of the roof.

The HeiTech pumpable support had the highest load density at 1.35 tons per square foot which limited convergence to approximately 0.5 inch. Conditions both inby and outby the face as shown in figure 11 were excellent with the Heitech pumpable support system. However, it should be noted that the Heitech support also had the lowest support safety factor (1.3) of the four alternative support systems utilized, meaning that load development approached the peak loading capability of the support. If the maximum loading capability of the support was exceeded due to unexpected additional roof loading or variability

in the peak strength of the support, the convergence would increase to approximately 4 inches at the installed spacing based on the residual support capacity of approximately 90 tons. Since this is the critical convergence for this mine at which point roof conditions deteriorate significantly, it is critical that the peak pumpable support capacity not be exceeded through the zone where it is desired to maintain full roof control, which means that the spacing must be properly maintained during installation so as not to overload the support past failure.



Figure 11a. Outby area supported with pumpable crib.

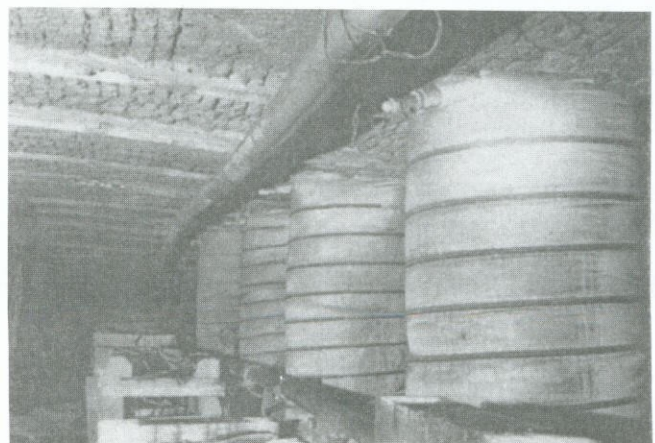


Figure 11b. Inby area supported by pumpable crib.



The Propsetter support had the lowest margin of GRC safety at 1.79, but even this system is conservatively applied in that the convergence was limited to 1.6 inches. Conditions outby the face were excellent as shown in figure 12a and relatively good inby the face (figure 12b). Some of the Propsetter supports well inby the face mostly in the intersection areas (figure 12c), appeared to be in a state of post-yield deformation where "brushing" (yielding) caused the props to tilt from a vertical orientation, which is normal for this load condition. It does not mean that the prop is shedding load. Another possibility is that the props were being dislodged or moved laterally by flushing of the gob material, floor heave, and/or lateral displacements of the roof relative to the floor by the cantilevered roof beam. Despite these occasional abnormalities in the support condition, the Propsetter was able to maintain an effective air way beyond the first open cross cut inby the face. It was also reported by the mine that 5 or 6 Propsetters were dislodged from the mine roof and floor outby the face. Since convergence was minimal outby the face, the cause of these props "falling over" was never definitively determined. The same props were reinstalled and performed well throughout the duration of the test area.



Figure 12a. Outby area supported by Propsetter support.



Figure 12b. Inby area supported by Propsetter support.



Figure 12c. First open cross cut intersection inby the face supported with Propsetter supports.

The Heintzmann ACS support had the most limited yield capability of the four alternative support technologies used at this mine. The ACS also sheds load rather quickly after reaching its peak loading capability at about 2.2 inches (figure 7). However, the installed spacing provided the highest support safety factor (2.5), meaning the loads were kept well below the peak capacity of the support. Likewise the installed load density limited the convergence to 1.2 inches, which is considerably less than the yield point of 2.2 inches. Hence, this is a good example of how a stiff support with limited yield capability can provide effective ground control in a longwall tailgate, provided that a sufficient number of supports are installed per unit area to establish a high enough load density to minimize the ground movement. Figure 13 shows the condition of the entry both outby and inby the face in the area supported by ACS's. Similar to the Propsetter support, a few of the ACS props were tilted inby the face, but continued to provide support capability in this condition without becoming unstable.

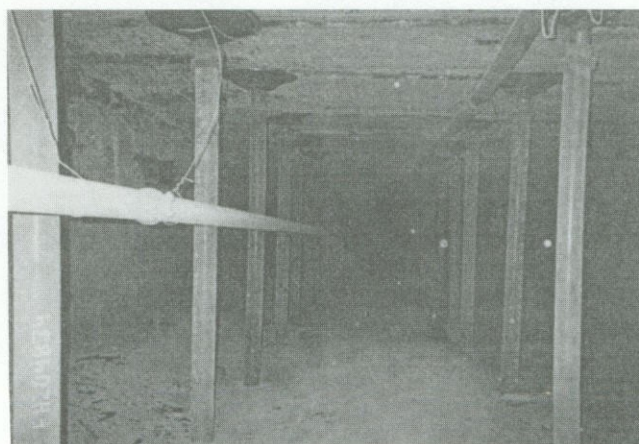


Figure 13a. Outby area supported by ACS supports.



Figure 13b. Inby area supported by ACS supports.

The Burrell Can support installation had a GRC safety factor (1.92) almost identical to that of the Heitzmann ACS supported area. The 1.92 GRC safety factor means that the installed load density is almost twice that needed to prevent roof failures from occurring. The entry conditions, both inby and outby the face were excellent with the Can support as shown in figure 14. Inby, the conditions were slightly better than in the ACS area. This improvement is attributed to the larger surface coverage and improved stability of the Can support compared to the ACS.



Figure 14a. Outby area supported by the Can Support.



Figure 14b. Inby area supported by the Can Support.

USING THE GROUND REACTION CURVE TO OPTIMIZE THE USE OF ALTERNATIVE SUPPORTS

Once the ground reaction curve is determined, applications of any other support technology can be defined by strategic employment strategies relative to the ground reaction curve. Table 2 shows alternative placement strategies for these and other alternative support systems for this particular mine site based on the measured ground reaction behavior with conventional wood and concrete support systems.

Examining table 2 reveals that other support technologies could be favorably used in this application. One example is the Link-N-Lock crib support developed by Strata Products. A 24-inch Link-N-Lock crib could be installed in a staggered double row with a center-to-center spacing of 11 feet per row (5.5 feet between cribs in opposite rows) and limit the convergence to 2 inches. Another alternative would be a single row of 36-inch Link-N-Lock cribs on an 8-foot center-to-center spacing, which also would limit the convergence to 2 inches. In contrast, a single row of 9-point cribs would have to be installed on a 5.2 foot center-to-center spacing to provide equivalent ground control capability.

Three other points can be made by examining the data in table 2. First, the spacing of stiff, high capacity support systems can become excessive at large displacements. It is important to remember that the ground reaction behavior was measured in the immediate vicinity of secondary support. It is assumed that the support loading is sufficiently transferred to control the roof and floor between the supports. Obviously, there are some limitations to this capability, which largely depend on the strength of the immediate roof. Generally, stronger roofs can span greater distances between supports than weaker roof. Currently, this capability is best obtained from empirical data within the mine, but a good rule of thumb to follow in the absence of this information is that the span between supports should not exceed half the entry width, particularly in weak roof conditions such as those observed in the Pittsburgh coal seam. Using this criteria, it is seen from table 2 that 9-point cribs and Link-N-Lock cribs must be employed at a load density greater than 0.63 tons per sq ft to avoid an excessive spacing where failure might occur between the cribs. Surface control of the immediate roof is another issue. Surface control refers to failure of the immediate skin of the roof. This is different from the excessive spacing issue discussed above in that there is no major failure of the roof rock. If surface control is necessary to prevent flaking of the roof skin between cribs, then methods such as wire meshing can be effectively employed as a control measure.

Second, it is seen from table 2 that stiff concrete supports must be installed at a high enough support load density to limit the convergence to less than 2 inches, otherwise the supports will fail prematurely, resulting in no support capability and unstable ground conditions after failure of the support. Thirdly, it is seen that required load density to limit convergence below 1 inch is not practical with the passive wood crib supports including the stiffer Link-N-Lock cribs. Wood is just too soft to generate meaningful loads at such small displacements.

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Table 2. Recommended placement and safety factors for alternative support technologies.

SUPPORT SYSTEM	Center to center support spacing & individual support capacities at convergences of 0.5, 1, 2, and 4 inches.							
	0.5 INCH (LD = 1.35) (SF = 2.16)		1 INCH (LD = 1.24) (SF = 2.0)		2 INCHES (LD = 1.04) (SF = 1.7)		4 INCHES (LD = 0.63) (SF = 1.0)	
	Load (tons)	Space (ft)	Load (tons)	Space (ft)	Load (tons)	Space (ft)	Load (tons)	Space (ft)
4-Point cribs	8	0.7	17	0.9	27	1.6	39	3.9
9-Point cribs	14	0.6	55	2.8	86	5.2	115	11.4
24-inch Link-N-Lock	10	0.5	45	2.3	92	5.5	115	11.4
27-inch Link-N-Lock	11	0.5	49	2.5	102	6.1	127	12.6
36-inch Link-N-Lock	13	0.6	63.5	3.2	132	8.0	162	16.1
Propsetter (8.5 in dia)	12	0.6	25	1.3	42	2.5	57	5.7
Stopping block cribs	65	3.0	0	N/A	0	N/A	0	N/A
SFR Donut Cribs	86	4.0	280	14.1	0	N/A	0	N/A
SFR Block (2 per layer) crib	85	4.0	210	10.6	0	N/A	0	N/A
HeiTech Pumpable Crib	190	8.8	240	12	112	6.7	90	8.9
Burrell Can (24 in dia)	40	1.9	65	3.3	90	5.4	90	8.9
Heintzmann ACS (100 ton)	39	1.8	46	2.3	102	6.1	36	3.6

Note: N/A indicates that the support fails prior to the designated convergence and does not have sufficient post failure (residual) capacity to be considered for application in this condition. LD designates the support load density of a single row of cribs on the designated center-to-center spacing. Likewise, SF refers to the GRC Safety Factor as computed by equation 8 for a single row of cribs. If two rows of supports are utilized, the designated spacing at a specific displacement would be reduced by a factor of 2 and the safety factor would be increased by a factor of 2.

OTHER CONSIDERATIONS IN TAILGATE SUPPORT SELECTION AND APPLICATION OF THE PROPOSED DESIGN METHODOLOGY

While the primary consideration in support design is obviously the prevention of roof falls through proper ground control, there are other factors to consider. These include: (1) the cost of the support, (2) material handling requirements and ease of installation, and (3) the impact of the support structure on ventilation. These issues are beyond the scope of this paper, but will be addressed in a paper at the upcoming Longwall USA conference in Pittsburgh this September (1999).

While this paper is focused on standing roof support applications, several mines have explored the application of intrinsic secondary support such as trusses to replace conventional standing support in longwall tailgates (4). Two points need to be made in reference to the truss support consideration. First, it should be noted that the design methodology proposed in this paper applies only to standing roof support. While some of the basic rock mechanics principles used here may apply to intrinsic support, the support mechanisms are different and these have not been examined in this study.

Another caveat of the design methodology pertains to the application in yield pillar gate roads. While in theory a ground reaction behavior can be established for yield pillar systems, the mechanisms of ground behavior and support interaction are different. In particular, the yield pillar system is a high deformation environment by design. Secondary support should ideally allow the ground to yield in accordance with the pillar deformations and not interfere to the point where the secondary support develops sufficient capacity to damage the roof while it is yielding. Hence, a stiff, high density support may *not* be desirable in this environment, and the important secondary support design consideration may well be the *stability and yield capability* of the support.

CONCLUSIONS

Several alternative technologies to conventional wood and concrete cribbing have been developed in recent years. These new support technologies provide improvements in supporting capability as well as material handling advantages. However, since their supporting characteristics are all different, a design methodology must be developed in order for mines to safely employ these technologies and to maximize their benefits without increasing the overall cost of support. Conservative applications or *trial and error assessments are no longer practical nor prudent for state-of-the-art longwall mines.*

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The design methodology proposed in this paper and examined through a field trial at a western Pennsylvania coal mine embodies a fundamental concept of rock mechanics, that being the "ground reaction curve". Measurement of the ground reaction curve at the western Pennsylvania coal mine indicated that at this mine operating in the Pittsburgh coal seam, the support capacity had a significant impact on the ground behavior in the longwall tailgate. Increasing the support load density by a factor of 2 from 0.625 tons per square foot to 1.25 tons per square foot decreased the convergence in the entry from 4 inches to 1 inch. Conventional 4 point wood cribs installed in a double row with an 8 ft center-to-center spacing in a staggered arrangement (4 ft center-to-center spacing among adjacent rows) resulted in marginal ground control. Concrete cribs constructed from concrete stopping blocks reduced convergence to 0.5 inch, but some failed at this deformation resulting in localized poor ground control due to the support failure.

Four alternative standing support technologies were installed at the western Pennsylvania mine: (1) The Can Support (Burrell Mining Products), (2) the Alternative Crib Support (Heintzmann Corporation), (3) Pumpable concrete support (HeiTech Corporation), and (4) the Propsetter (Strata Products USA). These alternative support technologies were installed at a support load density ranging from 1.12 tons per square foot to 1.35 tons per square foot, providing ground control safety factors of 1.79 to 2.16. Ground conditions for all these support applications were generally very good, which is consistent with the measured ground reaction behavior and installed support density.

The NIOSH Wood Crib Performance Model is currently being upgraded to a windows format. Included in this new version will be the addition of the performance characteristics of alternative support technologies such as those discussed in this paper. In addition, a subroutine will be provided that incorporates the longwall tailgate design methodology proposed in this paper. This program will facilitate the use of this design methodology and will allow mines to optimize the use of any support technology once a ground reaction curve for that particular mine is identified.

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Modern Shield Technology: Better Than Ever But Still Not Perfect

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ABSTRACT

A survey of the longwall industry was conducted to examine the performance of modern shield technology. The results of this survey indicate that state-of-the-art shields perform better and last longer than ever before, but premature failures do still occur. In addition, longwall operators are now using shields longer than ever before and a greater number of fatigue related failures on aging shields are now occurring. A generic assessment of these shield failures is provided, describing the nature of the failures and how widespread they are throughout the industry. An engineering assessment of shield design and factors that cause shield failures is made using both strength of materials and fracture mechanics principles. From this analysis, design practices to improve structural margins of safety and extend the life of the shield are proposed. The existence of unexpected shield failures indicate that current shield performance testing is at times inadequate. Recommendations are made relative to improved shield performance testing protocols including the benefits of testing in an active load frame such as the National Institute for Occupational Safety and Health's (NIOSH) Mine Roof Simulator. Even the most rigorous shield testing procedures, which strive to simulate in-service loading conditions, fail to consider key environmental issues, such as corrosion which is often the cause of structural fatigue failures in modern shields. The survey also indicated that hydraulic failures occur much sooner in the life cycle of a shield than structural failures, and although they can significantly degrade support capability, often go undetected for long periods of time. Methods to detect the hydraulic failures are also provided. The paper concludes with key points to maximizing shield design and life expectancy and ideas for future generation shield designs.

INTRODUCTION

The decline in the number of longwalls from a high of 118 in 1985 to 62 in 1999 has forced a reduction in the number of shield manufacturers from eight to two. The two major suppliers of shields to the US market are Joy and DBT America (formerly

Mine Technik America or MTA). Their share of the market is split fairly evenly - Joy has 32 faces and DBT America has 30 faces (1). The author believes that the merger of the shield manufacturers has been beneficial to shield design in two ways. First, the merger has allowed the best of the British shield designs (Joy) and the best of the German designs (DBT America) to be brought forward. By combining the design strengths from each company, the result has been an improvement in both structural design as well as control system technology. Second, the fierce competition during the decade of the 1980's may have forced manufacturers to "cut corners" relative to design in order to compete for sales, resulting in deficiencies in design that led to premature structural failures and less than adequate control technology. Hence, while competition is generally healthy resulting in lower prices, there needed to be a better balance of quality and price than existed in the 1980's. The hard reality is that shield manufacturers have to make money in order to produce a quality product and to stand behind this product with warranties, and this capability was jeopardized in the past and has been improved with the recent mergers.

A survey of the longwall industry was conducted to evaluate problems with current shield technology and future needs. The survey indicates that shields are lasting longer than ever before, but that **premature failures still occasionally occur** and some mistakes from the past continue to be made in recent shield designs. No shield should fail prematurely these days due to poor design if sound performance testing is conducted. There is no substitute for performance testing, but it must be done properly, otherwise critical design deficiencies may not be discovered.

Due to the maturity of shield technology and the economic pressures of the mining industry, longwall operators are now keeping shields in service longer than ever before. In addition, due to the increases in longwall productivity, the number of operating cycles per year continues to increase. The result is that the operators are again pushing the envelope in terms of extending the life expectancy of the supports. **Today's shields are expected to last 60,000 - 70,000 cycles**, while ten years ago

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the life expectancy was around 35,000 cycles. Hence, while fatigue failures were not a design issue in the early generation of shield supports, they have become critical to the survival of most longwall operators in an ever increasingly competitive market. Much is being learned about the behavior of aging shields which are kept in service long enough to fail from fatigue as opposed to failure from poor design or replacement before the end of their useful life due to technological improvements.

Deciding when to retire an aging longwall face and the specifications for a new shield can be a critical decision for any longwall operator. Most mines do not have structural engineers that can actively participate in these decisions. This paper is written in part to provide mine operators with a better sense of design issues and engineering mechanics that are relevant to shield design. **The intent is not necessarily to have the operators learn enough to know all the answers, but to provide them with enough insight so they know what questions to ask** and what to look for relative to failures that may occur. These insights into shield design should also be beneficial to MSHA in evaluating the safety of longwall shields. The paper concludes with key points to maximizing shield design and life expectancy and ideas for future generation shield designs.

SURVEY OF SHIELD PERFORMANCE

Figure 1 illustrates the maximum capacity utilized throughout the history of the shield support. The "bigger-the-better" philosophy continues to prevail with state-of-the-art shields now providing 1,200 tons of support, nearly 4 times the capacity of the first shield support installed in the mid 1970's (2). The age distribution of the shields operating in 1999 is shown in figure 2. The range of operating life extends from 0 to 14 years of service. An average shield operates about 4,000 cycles per year, translating into a face production of approximately 2.5 million tons per year. Hence, an average 10-year-old shield would have 40,000 loading cycles, while a shield employed on a high production longwall (4 million tons per year) would have about 65,000 loading cycles. Approximately one third of the shields have been in operation from 4 to 6 years (16,000 - 24,000 cycles), and slightly more than one-third (37%) have been operating for more than 6 years (24,000 loading cycles).

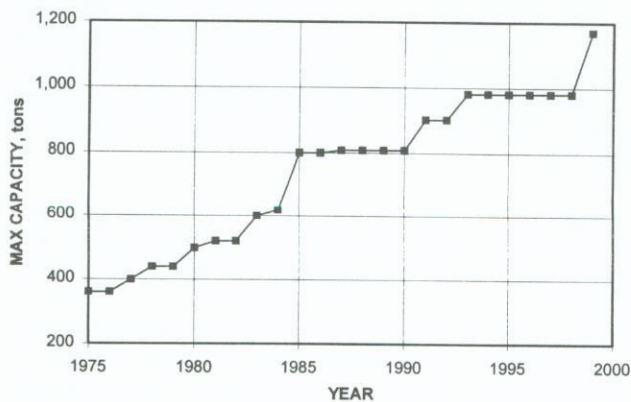


Figure 1. Maximum shield capacity utilization.

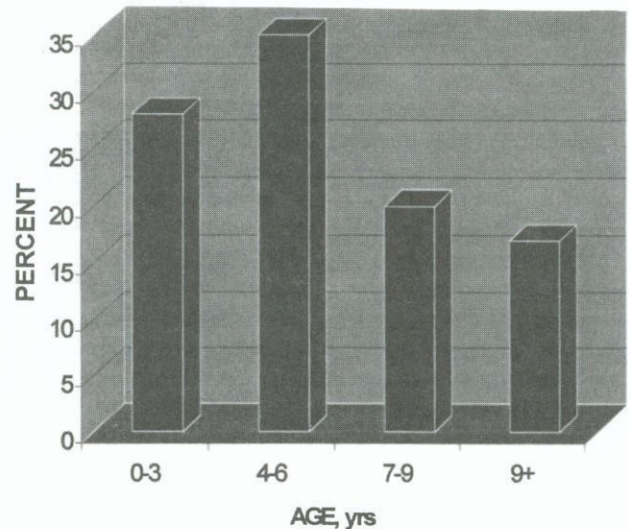


Figure 2. Age distribution of shields operating in 1999.

Figure 3 shows a near-linear relationship of structural failures with age. Failure is defined as structural damage to any support component. It does not necessarily mean catastrophic failure that renders the support inoperable. The linear relationship suggests that **there is more than just fatigue failures occurring**, since the frequency of fatigue failures should increase at a much greater rate as the shield ages. There were a wide range of structural failures reported. Most of the major failures involved either the leg sockets or pin joint problems (i.e., lemniscate link pins and clevises). These two failures are most likely to put a shield out of service and/or require structural modification to keep the support in service. Other structural failures occurred at areas of high stress concentration including the side shield on the canopy structure, the canopy capsule tilt cylinder bracket, the base bridge, the link cut out areas in the caving shield, various mounting brackets including the pilot-operated/yield valve manifold, and dishing of canopy skin covering plate. There were a few isolated cases of leg cylinder casing problems, but no catastrophic failures.

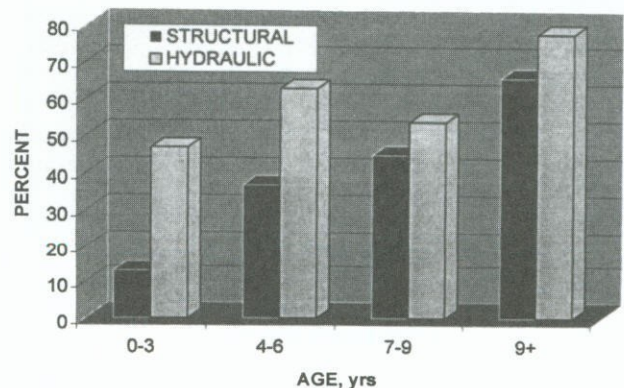


Figure 3. Distribution of structural and hydraulic failures on shields as a function of age.



There were more hydraulic problems reported than structural problems (figure 3), particularly in shields that have been in operation less than 6 years. There were nearly 4 times the number of hydraulic failures in shields operating 3 years or less and nearly twice as many for shields in the 4 - 6 year range than there were structural failures for the same age group. Obviously, the weak link in shield design is the hydraulic system. Numerous hydraulic failures or problems were reported. The high frequency of failures in the first 3 years of operation is distorted by failure of a component called a hydrafuse. The hydrafuse is a safety device incorporated by Joy as part of the leg cylinder design on their recent generation of shield supports in response to a fatality that occurred recently in Australia. An Australian mine worker was killed when the hydraulic fluid in the retract annulus of the leg cylinders was inadvertently pressurized while setting the support. Normally, the retract port is open through the return line back to the hydraulic reservoir whenever the leg cylinder is being pressurized. This is necessary since the pressure would be highly intensified (by a factor 30 or more) if the port was blocked, as in the case of the Australian fatality. The resulting pressure intensification caused the cylinder end cap to rupture, striking and causing a fatal injury to the mine worker.

The hydrafuse is incorporated into the retract circuit and acts as a yield valve to prevent the inadvertent build-up of pressure in the retract annulus of the leg cylinder. The design utilized by Joy features a brass clip that shears off at a predetermined load (hydraulic pressure). Once the clip is broken the "fuse" must be replaced. With a failure pressure of nearly twice the pump operating pressure, it was expected that the fuse would provide the desired safety on the rare occasions when there was abnormal pressure, and that replacement of the hydrafuse after the failure would not be much of an inconvenience. To the surprise of everyone, hydrafuse failures began occurring on several shields at several mines. The failures seemed to occur whenever the support had set idle for a period of time, and whenever the leg pressure was at or near yield pressure and almost always on the top stage retract circuit. A series of tests were conducted on a longwall shield under a variety of loading conditions at NIOSH in the Mine Roof Simulator in order to determine if unexpected pressures were occurring in the retract circuit. None were found. Similar measurements of retract pressure on active shields in underground mines that reported hydrafuse failures were taken by Joy, and again no abnormal pressure spikes were found. However, it is believed that a water-hammer effect is being created by the valve operation. Joy has since installed a restrictor valve in the circuit to dampen out any potential pressure spike. Early indications from the field installations indicate that this may be working to prevent the undesirable failure of the hydrafuses. Unfortunately, the restrictor also increases the time required to lower the top stage. Fortunately, in two-stage leg cylinder designs, the bottom stage is lowered first and lowering of the top stage is only required when the bottom stage is fully collapsed. Hence, the increased time to lower the top stage will not be a problem during most production mining.

Other cylinder problems caused pressure losses that limited support capacity. One example was due to poor fabrication where the internal bore of the top stage was off center resulting in a weakened casing that was unable to sustain the pressure intensification that occurred in the top stage. Other problems were reported with defective yield valves that would not reset resulting in the inability of the shield to adequately hold load after yielding. Similar problems were reported with the staging valve which also caused loss of pressure in the leg cylinders at a few mines. One mine reported clearance problems with the leg cylinder and the base rib plates where the cylinder leaned into the rib plates causing internal damage to the cylinder and seal leakage. Failures also occurred with the advance ram cylinders including failure of shuttle valve springs and structural failure of a hollow tube relay bar design.

Most hydraulic problems are related to internal leakage due to seal wear and/or corrosion of the cylinders. These problems typically begin when shields have been in service 4-6 yrs. As figure 3 shows, 60 pct of the shields had some sort of hydraulic problems during this time frame. Many problems go undetected for extended periods of time, resulting in degraded support capacity that can contribute to ground control problems in heavy loading conditions. Methods to detect the onset of internal hydraulic leakages are discussed later in the paper.

There is a trend towards the use of low treatment synthetic fluids in western mines. These "low treatment" systems utilize synthetic oils in concentrations of only 1 to 2 pct compared to the conventional 5 pct mineral oil/water emulsions. The major disadvantage of the synthetic "low treatment" system is that there is little room for error. A small drop in the oil concentration can lead to lubrication problems and accelerated corrosion of leg cylinder bores, pistons, and rods. Therefore, maintenance of the oil/emulsion is much more critical than in the "high treatment" systems, where the oil content can be reduced from 5 to 4 pct with little if any detrimental effects. Bacteria growth can also be accelerated in very low concentrations of oil emulsions, which can cause more severe corrosion problems than if there were no oil at all. One western mine reported severe leg cylinder corrosion due to problems with a low treatment synthetic emulsion on shields that have been in operation less than 3 years.

Problems with the electro-hydraulic control systems were also reported. These included sticking solenoid valves (problem with soaping in emulsion formulation) and chattering valves due to fluid dynamics at high flows. Most mines reported that there have been significant improvements made in the latest generation of electro-hydraulic controls. In particular, the DBT (MTA) PM-4 system seems to be a significant improvement over the previous generation PM-3 design.

ENGINEERING ASSESSMENT OF SHIELD DESIGN AND FACTORS THAT CAUSE FAILURES

Structural failure can be divided into the following four basic categories: (1) general yielding or excessive plastic deformation, (2) buckling or general instability, either elastic or plastic, (3)

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subcritical crack growth (fatigue, stress-corrosion, or corrosion fatigue) leading to weakening of the component or unstable crack growth, and (4) unstable crack extension, either ductile or brittle, leading to either partial or complete failure of a member. Structural shield failures are generally of types 1 and 3 and occasionally type 4. A better understanding of the cause of these failures can be obtained by reviewing the principles of engineering mechanics for structural design.

Classical structural engineering design is based primarily on a strength of materials approach. The goal of this approach is to define the load requirements for the structure, and then to proportion the sizes and shapes of the various components to prevent tension failure or to prohibit instabilities such as buckling in a compression failure. Failure is handled by employing safety factors to ensure that the allowable stress in components remain within the elastic range of the strength of the steel. These basic engineering principles are used in current shield design, except **much higher allowable stresses are permitted in shields** compared to the safety factors employed in the conventional design of structures such as buildings and bridges. Failures by general yielding, such as that shown in figure 4, are indications that shields have been designed without sufficient margins of safety to prevent yielding. This is in part due to the low life expectancy of early generation shield supports. As the demands for greater life expectancy grew, the manufacturers were forced to employ more conservative design approaches. However, even with state-of-the-art shields, it is safe to say that the margins of safety may still remain below that of conventional structural design where life expectancies are much longer and failure of any sort is unacceptable.

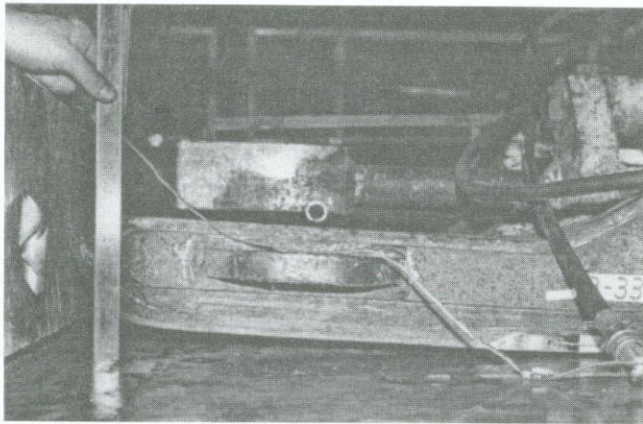


Figure 4. General yielding of toe of base unit in shield design from early 1980's.

Finite element modeling continues to grow in application as a powerful tool to optimize the design of complex structures. Recently, shield manufacturers have incorporated finite element modeling into their shield design practice. This practice has helped to improve the overall design of the current generation of high capacity longwall shields. An important criteria in the use of finite element modeling for shield design is to employ realistic boundary conditions. Unlike buildings in which the load applications are well defined and limited in scope, the various contact configurations of the shield canopy and base that

are established by the mine roof and floor and strata caving mechanics, complicates the use of finite element modeling in shield design. Errors in the assumptions made in the boundary conditions, such as not properly constraining the model, can lead to poor shield design. Another very significant factor is to recognize the limitations of finite element modeling. A finite element model should be able to accurately define nominal stress developments in shield components, but may or may not (depending on the complexity of the model) accurately define areas of stress concentration which are critical to the long term structural integrity of the shield. The common practice is to model the shield as a homogenous structure. In reality, the weaknesses of a shield construction are often due to weldments where complex anisotropic properties occur that are not easily modeled. Without being able to accurately model these complexities, the best practice is to derate the model by incorporating factors of safety to account for these issues. However, this requires considerable experience by the modeler, which is best obtained through calibrations of models with empirical performance test data. Another limitation on the use of finite element modeling is the time required to construct and debug the model and to analyze various load conditions. An entire shield model may require as much as a full man-year of effort. The bottom line is that although progress has been made in applying this technology to improve shield design capabilities, **a shield should not be designed solely on the results of a numerical model.**

Since the longwall face and shield supports are advanced with each shear cut, the shield is subjected to repetitive loading and the potential for fatigue related failures. Fatigue is the process of cumulative damage that is caused by repetitive fluctuating loads. Ductile as well as brittle materials are susceptible to fatigue failures. Since there is no large amount of plastic deformation prior to the fatigue failure even in ductile materials, **fatigue failures appear with little or no warning.** The failure mechanism is considered to be quite complex, but in general a fatigue crack is initiated at some microscopic or macroscopic stress riser. The crack itself then acts as a stress riser to promote localized yielding in the vicinity of the crack tip. In general, the more severe the stress concentration and the greater the load fluctuation, the shorter the time to initiate a fatigue crack. After a certain number of load fluctuations, the accumulated damage causes propagation of a crack or cracks leading to failure of the structure.

Conventional structural design for things such as machines and bridges employ relatively conservative design practices relative to fatigue loading to ensure the desired life and safety of the structures. The fatigue (endurance) limit of conventional steels typically is about 50 pct of the tensile (yield) strength. Generally speaking, this means that the nominal load (stress) developed in the shield components must be kept below 50 pct of the yield strength to prevent fatigue failures from occurring, thereby providing an indefinite life expectancy relative to fatigue failure. Such a design approach for shield supports would require much larger component sizes which would significantly increase the weight of the shield, and as such, are considered impractical for shield design. As the stress is increased beyond

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this endurance limit, the number of cycles to failure is reduced, starting at about 1 million load cycles. For a life expectancy of 100,000 cycles which is more representative for longwall shields, the allowable stress to prevent fatigue failure is much closer to the yield strength of the steel, suggesting that a factor of safety of 2 is not needed in shield design at least in relation to fatigue issues. Of course, these generalities are subject to the specific properties of the steel and the support construction. The allowable stress may be considerably less in some circumstances. For example, the allowable stress to prevent fatigue failure for a plate welded on the flange of an I-beam (similar to a plate welded on the bottom of the base pontoon) is only 15 ksi (5).

In addition to fatigue, stress corrosion causes failures to occur under *statically* applied loads with stress developments well below the yield strength of the material and independent of the number of load cycles. This can account for structural failures that occur at low production mines with aging shields where the number of cycles is low but the age is high. In other words, a shield doesn't necessarily have to have a lot of cycles to fail. The failure occurs due to crack initiation caused by the corrosion and subsequent propagation of this crack or material defects similar to that observed for fatigue loading. The author believes that **stress corrosion is a significant and mostly overlooked factor in current shield design**. The mine environment is often wet and acidic which accelerates corrosion of shield components as shown in figure 5. The base units in particular are subjected to extensive corrosion since the wet debris continually forms around and often covers much of the base structure. An area of particular concern is the leg socket which collects rock and coal debris throughout much of the shield's operating life. When combined with fatigue loading (corrosion-fatigue), stress corrosion causes a further reduction in the useful life of the longwall shield. Corrosion-fatigue damage occurs **more rapidly** than would be expected from the individual effects or from the algebraic sum of the individual effects of fatigue, corrosion, or stress-corrosion cracking.

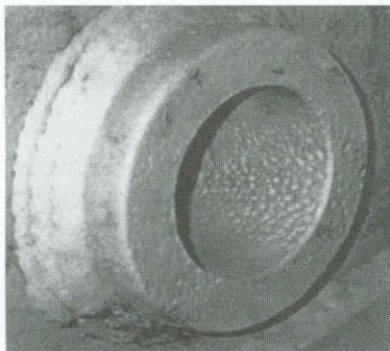


Figure 5. - Corrosion of a lemnicate link clevis.

The science of fracture mechanics was developed in the 1946 to 1966 period to analyze unexplained failures in several large scale complex structures where brittle fractures at stress levels no larger than were expected when the structure was designed were observed (3). However, it was not until the 1970's when

fracture mechanics developed to the point where practical applications were considered and incorporated into the design of structures where brittle fracture and fatigue failure is of concern. The underlying premise in fracture mechanics is that real structures contain numerous discontinuities of some kind. These discontinuities can be flaws in weldments, poor fabrication practices where torch cutting leaves surface scars and abrasions, or cracks in base or weld materials due to things like corrosion. These discontinuities act as stress risers and unstable, rapid fracturing occurs when the stress intensity factor at the crack tip reaches a critical value. Such discontinuities certainly exist in longwall shields and are undoubtedly a **primary source of structural failures**.

The goal in fracture mechanics design is to keep the stress intensity level below the critical value that promotes crack propagation, much like the goal of strength of materials design is to keep the design stress within the material's elastic range. To ensure that a structure does not fail by fracture, the number of cycles to grow a small (often microscopic) crack to a critical crack length must be greater than the life of the structure. Hence, the key idea to consider in a fracture mechanics design is the ability of the steel to absorb strain energy. If the material has a high absorption capability, such as mild steel, crack growth will be limited to plastic flow of the material and general yielding will typically prevent brittle fractures from occurring. Conversely, the probability of fatigue related failures increases with low energy absorption materials. Unfortunately, from a fracture mechanics perspective, state-of-the-art shields are now constructed from high-strength steels (100,000 to 120,000 psi yield strength) which are much **more susceptible to brittle fracture** than the early generation of shield supports which were constructed from mild steel (35,000 to 45,000 psi yield strength). The motivation for the high strength steel of course is to minimize the size of the components and keep the weight of the shield down, which is an ever increasing challenge for the design engineers as the capacity of the shields continue to rise. In essence, the term high-strength steel is a misnomer for two reasons. First, the fracture toughness is reduced which means that the allowable stress, as a percent of the yield strength, must also be reduced to prevent fracture from occurring. In other words, the full advantage of increased strength of the steel may not be realized. Secondly, unlike lower strength steels where the weld material is of equivalent or higher strength of the base metal, it is not uncommon for weaker weld materials to be used in the high strength steel constructions.

There are three primary parameters that are important in fracture mechanics design: the material toughness (K_{Ic}), crack size (a), and the stress level (σ). Their relationship can be expressed mathematically by equation 1 and by examination of figure 6. To the right and above each curve is the region of failure, whereas below and to the left of each curve is a crack size at a specified stress level that will not cause failure. As seen in figure 6, a larger K_{Ic} (tougher steel) will shift the curve up and to right, indicating that for a given stress level (σ), a larger crack size (a) can be tolerated before global failure occurs. The general approach to shield design is then to formulate a fracture control plan by defining the largest crack size likely to be found

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in the structure and then estimating the stress intensity factor using equation 1. Once this is done, the allowable stress to prevent fracture for a specific steel can then be determined.

$$K_c = \sigma \sqrt{\pi a} \quad (1)$$

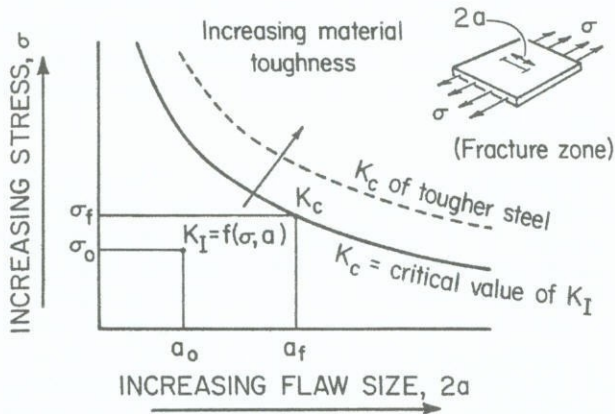


Figure 6. Fracture mechanics principles regarding material toughness, stress, and flaw size.

Some recommendations for fracture mechanics design that pertain to shield supports are summarized as follows:

1. For fatigue loading, the total life of a structure depends on the cycles required to initiate a crack and the cycles required to propagate the crack to a critical size that causes failure. Most of the life of a structure is spent in crack initiation. Therefore, any decrease in the initial flaw size has a very significant effect on the fatigue life of a structure. For example, doubling the initial crack size could reduce the life expectancy by a factor of 2 or more. This means that **weld quality** and fabrication practices or corrosion which creates a flaw in the material **can be very critical to the life expectancy of a shield**.
2. Fatigue is also significantly affected by the magnitude and stress reversal of the load fluctuation. Performance testing should ideally be conducted from no load to full yield pressure in the leg cylinders under various roof and floor contact configurations and displacement profiles that maximize the load fluctuation in each of the support components.
3. For welded constructions, the possibility of residual stresses exist. Depending on the toughness of the material, most cracks that initiate in the presence of residual stresses could arrest quickly as soon as the crack propagates out of the region of high residual stress, but the initial crack size for any subsequent crack growth will be fairly large and shorten the life expectancy of the structure. Hence, proper welding techniques to control residual stress developments can be critical to the life expectancy of the shield support.

DESIGN PRACTICES TO IMPROVE STRUCTURAL MARGINS OF SAFETY AND EXTEND SHIELD LIFE

Since most structural shield failures can be attributed to some form of fracture, the basic elements of fracture control can significantly improve shield life. These are: (1) use a lower design stress, (2) minimize stress concentrations, (3) reduce flaw size or control crack growth, (4) minimize corrosion, and (5) use materials of improved toughness. These factors are illustrated in figure 7.

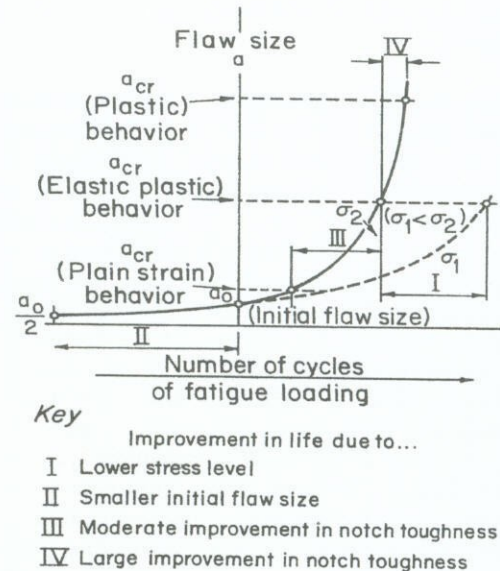


Figure 7. Impact of reducing flaw size or stress magnitude or increasing material toughness on extending life expectancy of a structure.

Lower Design Stress

Some margin of safety should be employed in the design stress relative to the yield strength of the steel. Civil engineers typically use a factor of 1.66, meaning that the allowable stress is about 60 pct of the yield stress for non-fatigue loading and further reduced by 50 pct or more when fatigue loading applies. While these levels of safety are not practical in shield design due primarily to cost and weight limitations, it is important to recognize that a small reduction in (tensile) stress developments will significantly reduce crack growth since the two are related by an exponential function. Past practices of designing to or near yield strength should be avoided in modern shield design where the life expectancy exceeds 50,000 loading cycles.

Link pins and clevises are a prime example of historically poor design practices in shield supports which continue even today. Deformation and/or excessive wear in the pin clevises is undoubtedly the primary cause of premature shield retirement and/or structural rebuild. There are clear indications that these areas are subjected to stress beyond the yield strength of the steel. This poor design is caused partly by manufacturers not giving sufficient credence to the load conditions which cause

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high loading in the caving shield - lemniscate assembly, namely loss of frictional contact at the roof and floor interface and standing the support on the toe of the base. With the possible exception of shields designed for low seam heights, there is adequate space available to increase the bearing area of these clevises and pin diameters to reduce the stress and substantially improve the life expectancy of these components. The joint design problem also needs to recognize the importance of pin tolerance. First, the pin contacts only a portion of the clevis. Typically, arcs of 45 to 60 degrees are used in the design analysis. Obviously, the 45 degree arc assumption will lead to more conservative designs. Conservative assumptions should be made to allow for reduced areas as wear occurs (figure 8). Excessive pin tolerances can lead to point loading by allowing the pin to rack within the joint clevis as illustrated in figure 9. Corrosion effects are also often ignored or not sufficiently accounted for in the design of pins, despite the fact that corrosion is a leading cause of premature pin failures or abnormal wear. Corrosion causes pits to occur in both the pins and the clevises (figure 5) which can reduce the bearing area by 25 to 50 pct and cause a proportional increase in stress.

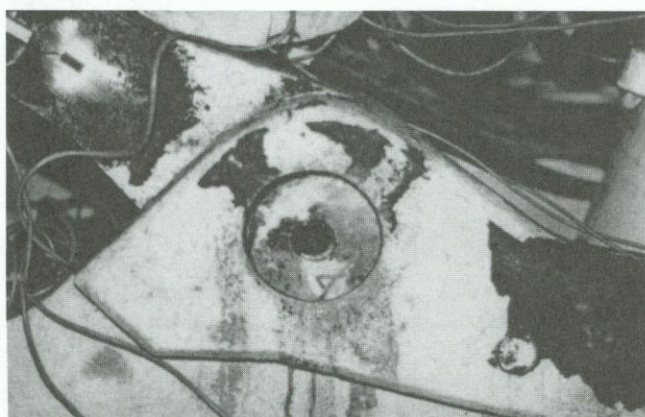


Figure 8. Reduction in lemniscate pin contact area for worn joint.

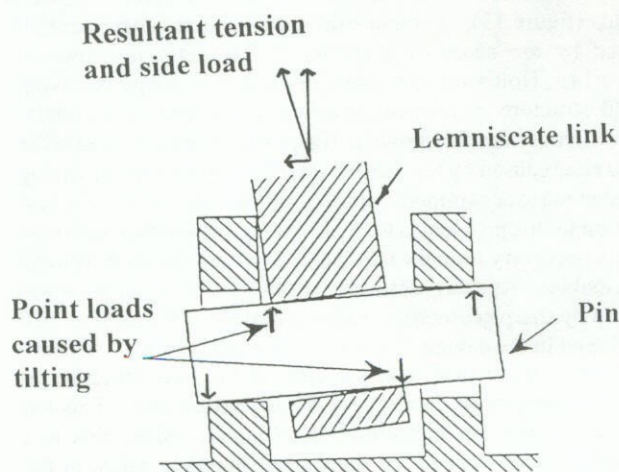


Figure 9. Ranking of lemniscate pin due to excessive pin wear or poor pin design tolerances that lead to stress concentrations and potential failures in the lemniscate joints.

Several failures have occurred where the leg socket casting weldments break due to fatigue or stress-corrosion, and thereby cause subsequent failure of the bottom base plate as the loading is transferred fully to the bottom plate instead of being distributed to the side rib plates. Most of the time the leg cylinder and casting punch through the bottom plate and into the mine floor rendering the support inoperable. A common modification to alleviate base leg socket failures of this nature is to add another plate to the under side of the base pontoon as shown in figure 10. Typically, this plate is about 1 inch thick and usually covers most of the length of the base pontoon. Reinforcement is also typically added to the top area of the side base plate. While this plate stiffens the side plate, its primary purpose is to restore the location of the centroidal axis, which was changed by the addition of the bottom plate, to its location in the original base design. Due to past failures of this nature support manufacturers are now beginning to incorporate a thicker bottom plate in the initial shield design.

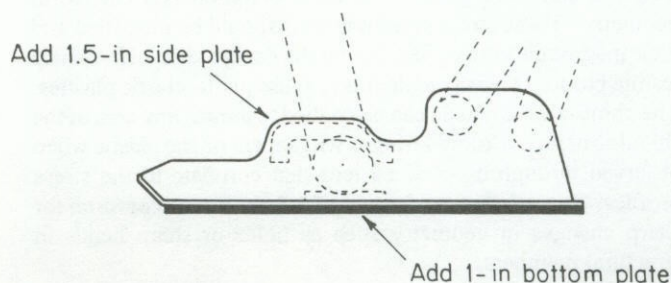


Figure 10. Modification made to strengthen base in order to prevent leg socket casting failures.

Some mines have successfully reduced operating stress levels by **derating the shield support** prior to underground installation. The derating is accomplished by simply installing yield valves with a lower operating pressure than specified in the design. Since the leg capacity controls the maximum load developments within the support, lower leg loads translates into reduced component loading. For example, if a 1,000 ton shield support is derated by 10 pct to a 900 ton capacity, the margin of safety relative to the tolerable crack size that will prevent fatigue fracture may increase by 20 to 30 pct.

If failures are observed underground, **procedures to reduce shield loading should be implemented**. A good practice to reduce face weighting is to maximize the rate of advance. Idle faces generally have a tendency to create higher shield loads. Also, setting forces should be optimized. Conventional wisdom has been to increase the set pressures or to ensure that full setting (pump) pressure is maintained. However, in displacement controlled loading where the load development is proportional to the shield stiffness and the support resistance is not controlling the ground movements, reducing the setting loads may also be an option to reduce the total loading on the support. Another possibility is to reduce the depth of the shearer cut. Much of the shield loading is generated from the shearer pass.

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A reduced cut may reduce shield loading, although there has not been any data collected to substantiate this hypothesis.

Another issue related to lowering the design stress is to ensure proper operation of the support once it is put into service in the mine. Stress developments are greatly enhanced whenever the support is stood on the toe of the base. When a shield is set with the canopy tip up, the leg forces will typically rotate the canopy into full roof contact and in the process lift the rear of the base off the ground. This base-on-toe configuration increases the magnitude of stress in the lemniscate links by as much as 300 pct. Modern two-leg shields employ a control system for the canopy capsule cylinder which prevents setting the shield with the canopy tip up. Hence, if this system is deactivated by the mine personnel, the life expectancy of the pin joints and the entire shield can be significantly shortened.

Minimizing Stress Concentrations

There are numerous sources of stress concentrations in longwall shield supports. The most common is a change in geometry. These stress concentrations should be identified and their magnitudes quantified during the design and performance testing phase. One way to do this is to use photo-elastic plastics. The photo-elastic plastic can be applied to almost any area of the shield structure. Colored fringes will appear on the plastic when observed through a polarized lens that correlate to the stress profiles. **Stress intensity factors of 2-3 are not uncommon** for sharp changes in geometry such as holes or sharp bends in structural members.

One example of this is a lemniscate link design with a bend in the link. The bent link is typically employed on a shield with a low profile designed to operate in low coal seams. The bend is necessary to provide clearance with the caving shield in the collapsed or low operating height. Lemniscate links are primarily axially loaded members, but the "bent" geometry induces additional stresses due to bending, and thereby significantly reduces the margin of safety for this component. Figure 11 depicts failure of a bottom lemniscate link on a 620-ton shield that occurred during performance testing at the NIOSH Safety Structures Testing Laboratory. Although the shield had 45,000 load cycles from underground service prior to the testing, a new link was installed on the shield that was performance tested in the laboratory. This was a new link design that was fabricated for the mine by an outside vendor (not the support manufacturer). Failure occurred after only 14,000 loading cycles. This failure illustrates two problems. First, the design (allowable stress) was too high. Test results revealed that the nominal stress in the link exceeded 90 pct of the material strength. In addition, it appeared that the link side plates had been torch cut, adding an additional stress riser to the already sharp change in geometry at the bend in the link in the area where the failure occurred. These two factors resulted in an unacceptable time to failure. Figure 12 illustrates another problem where the fabrication process left large flaws that led to premature fatigue failure of the base rib. This problem may have been alleviated if the surface was smoothed to remove most of the surface flaws.

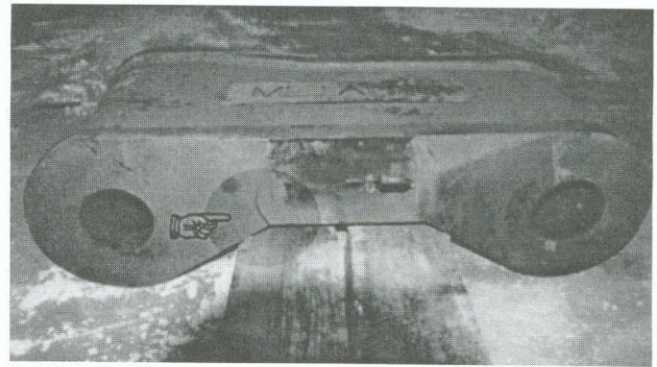


Figure 11 - Failure of bottom lemniscate link.

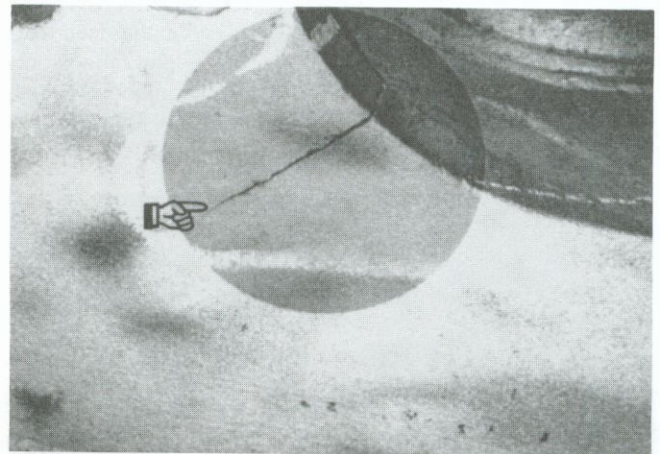


Figure 12. Failure of base rib side plate where fabrication process left large flaws in surface structure.

Any hole in a structural plate is another area where stress is concentrated. The structural components of a longwall shield (canopy, caving shield, lemniscate links, and base) are connected by a pin and clevis arrangement. These areas are also sources of stress concentration and fatigue failures on aging longwall shields (figure 13). Another example of a stress concentration caused by too sharp of a change in geometry is shown in figure 14. Holes are sometimes cut into the canopy or caving shield structure to accommodate the placement of hydraulic hoses. In the example shown in figure 14, failure occurred at the stress riser caused by the sharp corner in a cut-out on the caving shield made to accommodate hosing for the side shield. The best solution to this problem is to avoid the hole altogether and if the hole is necessary to make sure that the corner radius is as large as possible. Another example of failure due to stress risers created by sharp geometries is shown in figure 15, where a crack developed in the caving shield near the canopy hinge. A clean-out hole is often placed in the side of the base structure to facilitate removal of debris from the leg socket area. This too can be a source for concentration of stress. Since this is a critically loaded area of the base, care should be taken in the design to minimize the stress concentration by incorporating a favorable geometry and orientation with respect to the stress field in this member.



Figure 13. Failure in base lemniscate link clevis.

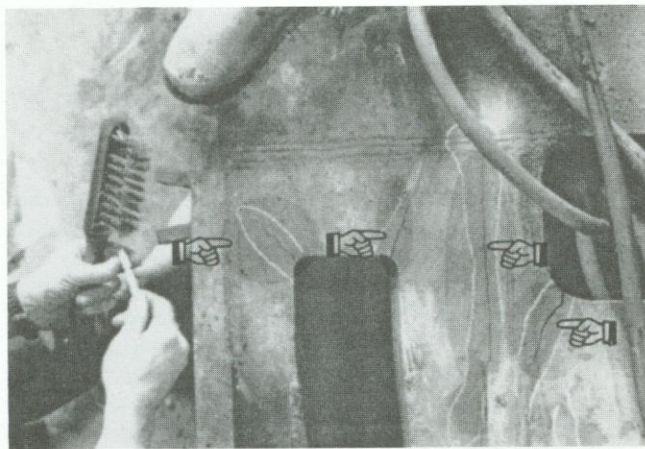


Figure 14. Failure due to stress concentrations in cut out sections of caving shield.

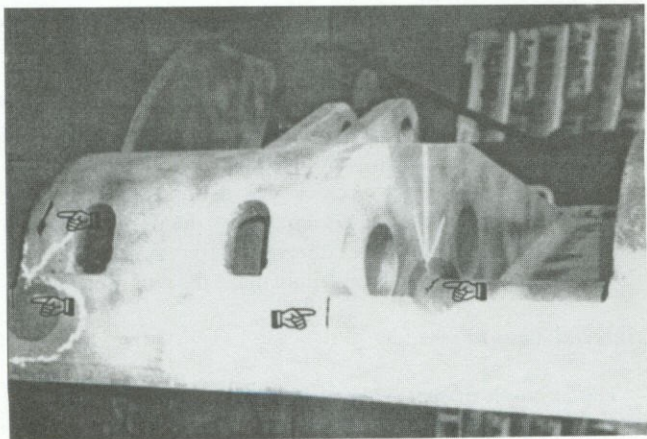


Figure 15. Failures of caving shield due to stress concentration in sharp corners where lemniscate links connect.

Figure 16 shows failure of a canopy leg socket casting due to a stress riser created by a sharp change in geometry. The failure

in this case occurred in the casting itself. This failure occurred on several shield supports in the late 1980's, all of which utilized this same basic socket design. Again, this failure probably could have been prevented by a smoother geometry. Leg sockets are often a source of fatigue failures in aging longwall shields. The leg socket is a critical area since the full load developed within the hydraulic leg cylinder must be transferred into the canopy and base structures to be distributed to the mine roof and floor. An examination of the structural mechanics associated with these base socket failures has led to some other design changes that are intended to reduce the stress concentration in the leg socket casting/base structure connection. The rectangular geometry of the socket casting and its placement between the side rib plates of the base structure position it at right angles to the principal tensile stress caused by the bending of the base structure. This orientation creates a stress intensity factor which helps to promote fatigue-induced fracturing of the weldment. Some shields are now being designed with more of an elliptical or zipper shaped casting (figure 17) so that the front and/or rear edge is not perpendicular to the principal tensile stress, thereby resulting in a reduction of the stress intensity factor.

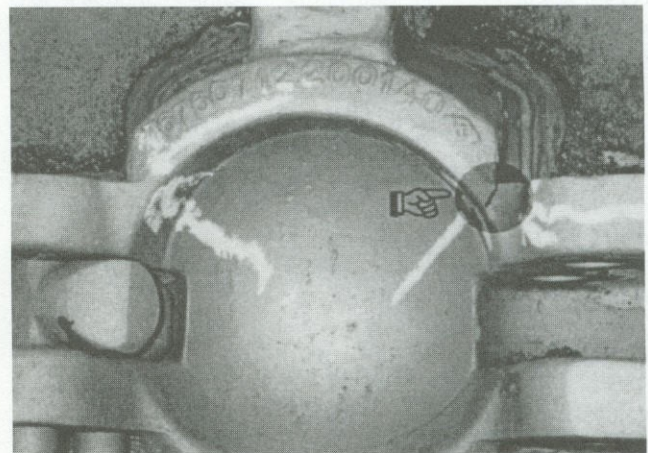


Figure 16. Failure due to stress riser created by sharp corner in canopy leg socket casting.



Figure 17. "Zipper-shaped" leg cylinder casting to avoid stress concentration caused by orientation of casting to principal stress field.

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Reducing Flaw Size or Controlling Crack Growth

Weldments are an essential part of the shield fabrication process. However, since weldments are a primary source of structural flaws, the quality of the welds is critical. As the principles of fracture mechanics illustrate, the initial flaw size created by the welding process is critical to the crack propagation and the margin of safety achieved in this structure. Once a crack develops, it is desirable to keep the crack contained within the weld and not have it progress to the base material adjacent to the weld. This action may depend on the nature of the heat-affected zone in the immediate vicinity of the weld. In general, the heat affected zone results in anisotropic material properties and residual stresses that tend to reduce the toughness of the steel in this area and increase the likelihood of fracture into the base metal. For the high strengths steels used in modern shield supports, proper heating and cooling of the steel during the weldment process is crucial to preventing crack initiation and growth in the heat affected zone. This is why it is very difficult to conduct repairs to damaged shields underground, since it is virtually impossible to be able to preheat and properly cool the steel when welding at the longwall face. Another approach that can be used to keep crack growth contained to the weldments is to create breaks in the weldments. This practice is sometimes used in leg socket castings. The break in the weld at the corners of the casting acts like a crack arrester to stop the growth of the crack. This same technique is used in the airplane industry by drilling holes in the metal at the end of an observed crack before it reaches a critical crack length.

Perhaps the best approach to avoid problems associated with weldments is to eliminate them when possible. A good example of this pertains to the leg socket design. A typical construction for the leg cylinder base socket is shown in figure 18. A casting 3-4 inches thick and 18-24 inches long with a spherical seat to accommodate the bottom of the hydraulic cylinder is placed on top of the bottom cover plate on the base pontoon, and is welded in place along the four sides of the top of the casting to the side rib plates and cross plates that connect the two side base plates together. This design is highly dependent on the welds to transfer load into the side rib plates and maintain the structural integrity of the socket connection. An alternative design that is now being employed in some canopy leg sockets to alleviate the weld fatigue problem is to cut rectangular holes into the side base rib plates and extend the width of the casting so that it bridges across the base but is supported by the side rib plates (figure 18). In this configuration, leg cylinder loading is transferred directly to the side rib plates of the base structure entirely through base metal contact, and is not dependent on the weldments to achieve this load transfer.

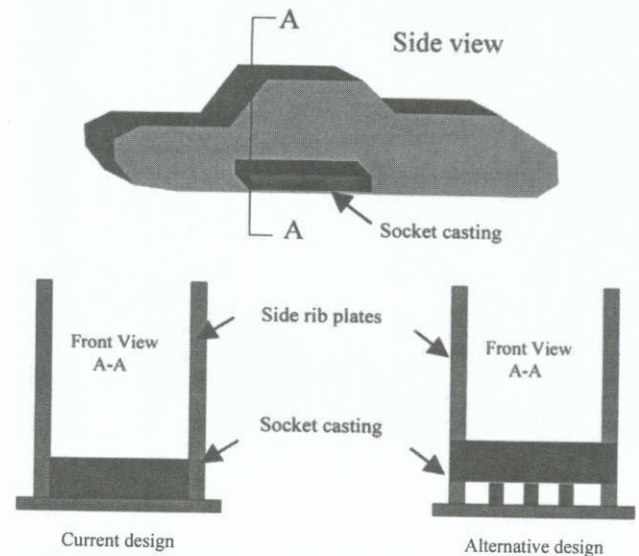


Figure 18. Conventional and alternative leg cylinder casting design.

Corrosion Control

Some steps can be taken to circumvent the problems caused by corrosion. The most convenient approach is to try to protect the shield from the environment. For shield supports, this mostly applies to painting with an industrial paint that is resistant to the wet mine environment. However, for some components such as the pins and clevises where there is considerable wear due to the kinematics of the shield during load application, painting or even plating with more resistant material is not an option. Impregnated pins with zinc phosphate through sheridizing has proven beneficial, but the effectiveness is limited due to the wear in the joint. Another option worth considering is some form of lubrication for these joints. A simple grease fitting would be an improvement, although somewhat impractical to maintain on an active longwall face. Hence, a sealed joint of some form would be preferable.

Tests have shown that compressive stresses on the surface of materials that are exposed to the environment will not necessarily minimize corrosion-fatigue crack initiation, but they can reduce the possibility of crack growth and thereby prevent failure from occurring. This could be done by induction hardening the joint pins. The concern with this approach would be that the induction hardening might make the pins more brittle.

Material Toughness

The strength of materials approach essentially ignores the toughness of the material, which as described is the most significant material property for fracture control. There is a tradeoff in the use of high strength steels if they provide superior strength but reduced toughness with greater chance for failure. Since the author is not familiar with the German and British steels used in the shield construction, no specific

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recommendations are made, but the issue should be investigated with the shield manufacturer when new shields are purchased.

ISSUES REGARDING PERFORMANCE TESTING

There is no substitute for performance testing to reveal design deficiencies. However, in order for performance testing to be effective, it must properly simulate the in-mine service conditions, and should always be conducted on a prototype shield *prior* to the start of production shield fabrication to ensure that any design deficiencies discovered in the performance testing can be properly corrected.

Simulation of the In-mine Service Conditions

There are two basic aspects to shield loading. The initial load condition is determined by actively setting the shield against the mine roof and floor. Subsequent loading is produced by the movement of the surrounding strata during the caving process and the associated internal forces developed within the support structure.

As the shield is set against the mine roof and floor, there is a tendency for the canopy to be displaced horizontally relative to the base. This is due to the resultant horizontal component of the leg forces, which causes either slippage of the canopy along the roof interface or displacement (compaction) of the fractured strata or debris immediately above or below the shield.

Once the shield is set against the mine roof and floor, load development within the shield is controlled by: (1) the contact configuration established with the mine roof and floor; (2) vertical displacement of the canopy relative to the base induced by deflection of the main roof beam and weight of fractured immediate roof strata being directly supported by the shield; (3) face-to-waste movement of the immediate roof as the strata breaks into disjointed blocks from the face abutment loading and loss of confinement; and (4) waste-to-face loading induced by gob material acting on the caving shield and/or the internal forces developed within the shield due to the leg forces and component reactions.

Performance Testing Load Frames

The first issue of significance in performance testing pertains to the capability of the load frame where the tests are conducted. There are three types of load frames available for full scale shield testing: (1) static load frame where shield loading is produced by pressurization of the leg cylinders by an external power supply, (2) active uniaxial load frames where the load frame can simulate roof-to-floor convergence, and (3) active biaxial load frames where both roof-to-floor convergence and controlled horizontal displacement of the canopy relative to the base can be provided.

Only a biaxial load frame, such as the unique Mine Roof Simulator at NIOSH's Pittsburgh Research Laboratory (figure 19) can truly simulate the ground movements associated with longwall mining. The limitation imposed by the static and

active uniaxial load frame is the inability to simulate the forward translation of the canopy due to slippage on the mine roof and the face-to-waste racking of the shield. Table 1 depicts the significance of this behavior through measured strain responses in the lemniscate links and base substructure during performance testing in the NIOSH Mine Roof Simulator load frame.

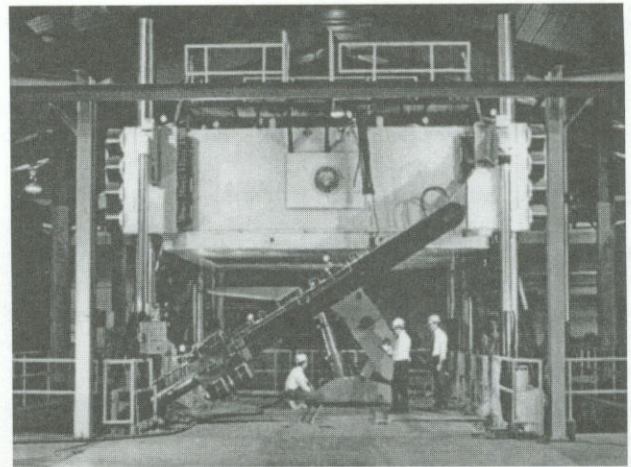


Figure 19. NIOSH unique Mine Roof Simulator.

Table 1. Significance of load condition (zero-friction test) on component stress development.

SHIELD COMPONENT	MINE A	MINE B	MINE C
TOP LINK	Measured strain, microstrain		
Restrained configuration	-1,608	-124	-338
Zero-friction configuration	-2141 (+33%)	-516 (+316%)	-1,033 (+205%)
BOTTOM LINK			
Restrained configuration	1,818	86	305
Zero-friction configuration	2,241 (+23%)	264 (+207%)	737 (+142%)
BASE			
Restrained configuration	-1,535	-1,060	-1,203
Zero-friction configuration	-1,745 (+14%)	-1,596 (+51%)	-3,676 (+206%)

Slippage of the canopy on the mine roof due to loss of frictional resistance at the roof interface can be simulated in static and uniaxial load frames by placing rollers on the canopy to create a "frictionless canopy test". Unfortunately, the zero-friction test with rollers on the canopy is not very practical for extensive cyclic loading since the rollers may require frequent adjustments due to frictional effects. Hence, this test is typically

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not included in the manufacturers performance testing protocol. If the frictionless canopy test cannot be conducted, then the support should be configured so that it is standing on its toe, which will require horizontal constraints at the toe of the base and rear of the canopy (at the caving shield connection). The horizontal force couple generated by this configuration allows load transfer to the caving shield - lemniscate assembly, similar (although not as severe) to that obtained with the zero-friction test.

Likewise, face-to-waste racking of the shield cannot be directly simulated by a static or uniaxial load frame. The contact configuration which most closely duplicates the caving shield-lemniscate assembly response for this condition in static or uniaxial load frames is to configure the support so that it is simply supported on the rear of the base. Face-to-waste racking of the shield canopy causes a reversal in the state of stress in the lemniscate links from compression to tension. Since reversal of the loading accelerates fatigue failures, both the base-on-toe and the base-on-rear configuration should be incorporated into safety testing protocols conducted in static or uniaxial load frames.

State-of-the-Art Safety Performance Testing Protocols

Consolidation Coal Company has been instrumental in developing performance test standards for longwall shields. The "Consol test" consists of several combinations of canopy and base contact configurations which are categorized as follows: (1) offset yield loading, (2) base toe loading, (3) three-point canopy torsional loading, (4) two-point canopy torsional loading, (5) side shield loading and base torsional loading, (6) diagonal base contact, (7) three-point base contact torsional loading, (8) symmetric base edge loading, (9) asymmetric base edge loading, (10) leg socket loading, (11) canopy dishing. These tests are designed to be used in static or single axis loading frames.

NIOSH has also developed a protocol for safety performance testing that takes advantage of the biaxial loading capabilities of the unique Mine Roof Simulator load frame. The biaxial loading capabilities allow several test configurations to be incorporated into one loading cycle. There are three test series in the NIOSH protocol: (1) Test Series I -- Transfer of Horizontal Load To The Caving Shield-Lemniscate Assembly (zero-friction test), (2) Test Series II -- Eccentric Twisting and Racking of the Canopy, and (3) Test Series III -- Leg Socket Shear Tests under Constrained Shield Conditions.

Further Enhancing Shield Safety Performance Testing

Proper simulation of the horizontal loading of longwall shields as described above will significantly improve shield safety performance testing. The one load condition which has not been given much consideration is lateral loading where the canopy is displaced laterally (sideways in a direction parallel to the coal face) relative to the base. This loading condition is most likely to occur for shields operating in high seam heights and in pitching seam operations. The sideways loading can also be caused by interaction among adjacent shields when setting, particularly when the side (caving) shield is protruding out too

far. Some mines have eliminated side shields from their design for this reason. Lateral loading can easily be simulated in the NIOSH Mine Roof Simulator, however, there is no easy way to simulate this loading condition in static or uniaxial load frames. Canopy contact configurations that are designed to produce offset leg yielding (i.e. load concentrated over one leg), three-point contacts designed to induce torsion, or base edge contact configurations that tilt the canopy and/or base produce some degree of lateral loading, but not nearly as much as when there is lateral displacement of the canopy relative to the base during the load application.

Even in the absence of high mining heights, some side loading can be developed by twisting of the canopy due to horizontal racking loads. The NIOSH Safety Performance Testing protocol addresses this issue by aligning the shield so that the direction of horizontal loading (applied displacement) follows a diagonal path from the left canopy tip to the right rear canopy corner as shown in figure 20. The diagonal path is reversed to the opposite corners to vary the direction of loading from left to right. The twisting action of the canopy creates point loading conditions in the shield joints as well as torsional loading of the caving shield - lemniscate assembly. Since the joints do not have a degree of freedom in the lateral axis, this loading action causes accelerated deformation of the joint areas and premature failure of the connecting pins and clevises.

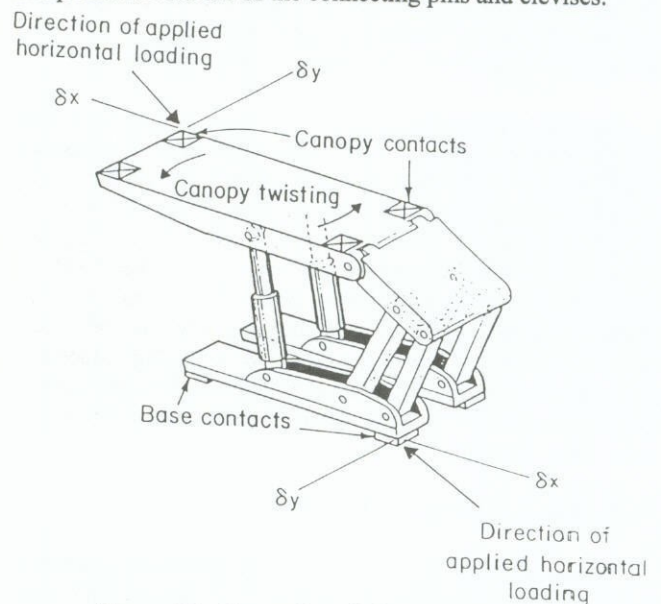


Figure 20. Eccentric twisting of the canopy.

Perhaps the most crucial element missing from state-of-the-art performance testing is the corrosion factor that is present in the mine environment. As previously described, corrosion can be the primary cause of fatigue related failures of shields operating in wet mine environments, and may be responsible for the inability to reveal these failures during the performance testing. Laboratory tests can be conducted where steel specimens are submerged in an environmental tank containing an acidic water similar to what is found in the mine. This would provide meaningful baseline data that may benefit future shield design. Unfortunately, such an approach is not practical with a



full shield while conducting performance testing, although it probably would be beneficial if a prototype shield could be exposed to an aggressive environment prior to testing. Of course, the time factor also makes this somewhat impractical for production shield testing, but such testing would be a worthwhile research project that may lead to scaling factors regarding the correlation between prototype performance testing and life expectancy in the mine.

DESIGN AND REPAIR ISSUES REGARDING SHIELD HYDRAULICS

First, it is important to realize that all shields will experience leakages due to seal wear and/or component failures several times during their life expectancy in the mine. Second, it is important to realize that many of these failures will go undetected for extended periods of time, resulting in significantly degraded shield capacity and ground control capability. Furthermore, if shield life is to be maximized, then recognition and correction of hydraulic failures is important. If both leg cylinders are not functioning properly, eccentric loading in the canopy and caving shield-lemniscate assembly for example will be created, causing increased probability of structural failure in these components. Additional loading may be transferred onto adjacent shields thereby reducing their life expectancy. And, degraded support capacity may induce more severe weighting of the shields, creating a snowball effect that further degrades the support capability.

Component failures and methods to detect them were presented at the 17th International Ground Control conference (4). Observations of the relative positions of the cylinder staging can be used to identify cylinder problems and the cause of hydraulic leakages. One indication of internal leakages is when the bottom stage is consistently fully extended. The bottom stage should be fully extended only on operating cycles which establish a new maximum operating height. Hence, on the majority of operating cycles, the bottom stage should not be at full extension. Another indication of hydraulic leakage is when there is a large difference in stage extensions of leg cylinders on the same support. Observation of the bottom stage position can help to identify the component failure (table 2).

Table 2. Stage movements associated with internal component failures on hydraulic leg cylinders.

COMPONENT FAILURE	Bottom Stage Movement
Staging check valve	Up
First stage seals	Down
Second stage seals	Up
Pilot operated check valve	Down
Yield Valve	Down

Problems with the staging check valve can be isolated by fully collapsing the shield and monitoring the leg pressures on the operating cycle after the support is reset against the mine roof and floor. The requirement is to have the bottom stage fully extended when the support is reset. In this configuration with a staging check valve that is functioning properly, the pressure in the bottom stage will not change significantly until the force in the top stage due to additional roof loading overcomes the setting force developed in the bottom stage (figure 21). An immediate increase in pressure in the bottom stage indicates that the check valve is leaking sufficiently to not allow the pressure in the top stage to be intensified.

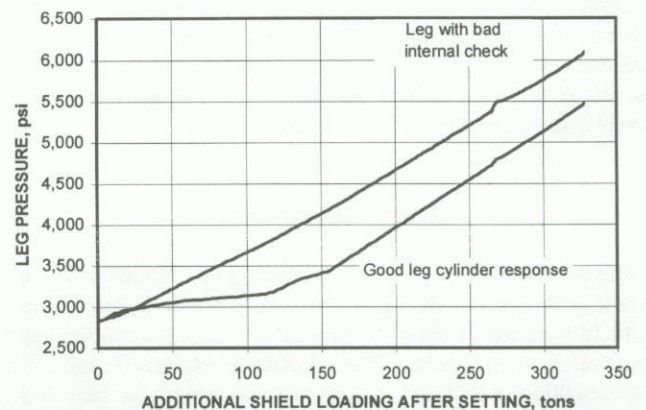


Figure 21. Identification of defective leg cylinder staging valve.

Two factors which reduce hydraulic life expectancy are **contamination** and **corrosion**. It is very important to change filters as needed to maintain a clean emulsion. Debris in the leg cylinder is a leading cause of stage valves failures which reduce shield capacity by 40-60 pct. Debris also reduces seal life and can cause check valves and yield valves to malfunction. Corrosion is also a primary factor in leg cylinder life. A chemical analysis of the emulsion fluid should be done periodically. Bacteria growth or poor water quality can significantly reduce the life of the leg cylinders.

From the design and repair perspective, there have been improvements in plating technology which has reduced the corrosion problem in recent years. Early shield designs used chrome plating for the leg cylinder bores. Chrome is a very porous material and even if the thickness is increased, the structure which is analogous to placing layers of chicken wire over top one another, remains quite porous. Bronze is a much better material and most shield cylinders are now plated with bronze. The harder the bronze is, the better the life that can be expected. An alternative to bronze is to use chrome over top of nickel plating. The electrolysis of the nickel creates a very hard material that is highly corrosion resistant, but the process is also considerably more expensive. Another problem with using nickel as the base metal in the plating process is that the two metals tend to react with one another reducing the bond between them causing the chrome to peel off in time.

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An impending problem with current plating technology is the environmental hazards associated with the plating process. Both chrome and bronze plating have environmental hazards associated with them that are likely to drive up the cost of these conventional plating methods in the near future. Since rebuilding of shield leg cylinders is a routine part of extended shield operation on every longwall, this can have a significant impact on the mining industry and new plating technologies need to be explored. Swanson Plating in Morgantown, West Virginia is now offering laser-controlled plating as an alternative. The laser technology in addition to being more environmentally friendly, reduces the heat affected zone by nearly two orders of magnitude and allows for much more controlled plating using materials that cannot be applied by conventional plating practices. More innovative solutions to consider would be the use of ceramic or composite materials that are much more corrosion resistant than the heavy metals currently used in cylinder construction.

CONCLUSIONS

The evolution of improved shield design continues with the current generation of shield supports. The previous generation of shield supports (1985-1995 era) experienced several serious structural failures. These included canopy sections falling off, bases cracking in half, leg sockets pushing through the base and canopy into the mine roof and floor, lemniscate links breaking, and pins bending and breaking. Generally, these problems have been addressed by the support manufacturers, but as the capacities continue to grow each new generation of shield supports must also go through the growing pains of finding design deficiencies. This is what is occurring now as isolated failures of modern shield design are again cropping up.

While proper shield design is not magic, there is an experience factor that cannot be overlooked. Unlike designing a building or a bridge where the load conditions are well known and fail-safe design philosophies can be readily employed, the mining environment is not well defined or easily understood. Mining engineers typically design to allow failure, while conventional structural engineers design to prevent failure. Furthermore, as the demand for extended shield life continues to grow, continued improvements in quality control and innovations in shield design must be developed if this requirement is to be realized.

While the design of longwall shields appears to be and is a complex issue, it would be beneficial to prospective shield buyers to learn the basic engineering principles associated with shield design issues as addressed in this paper. While it is not necessary to know all the answers, knowing what questions to ask is important. Hiring a structural engineering consultant is another option, but again be aware of the uniqueness of the mining environment and shield design issues.

There is no substitute for performance testing to ensure the integrity and safety of a new shield design. However, the performance testing must properly simulate the in-mine loading

conditions. The so called "Consol Test Program" has proven to be effective in eliminating shield deficiencies during prototype testing. Dynamic loading as provided by the NIOSH's unique Mine Roof Simulator load frame is the most accurate approach to simulating the in-mine loading conditions. NIOSH has also developed a state-of-the-art safety performance testing protocol for shield supports that is now available to the longwall mining companies.

Corrosion is by far the most overlooked factor in both shield design and performance testing. Corrosion control practices are limited, but they need to be followed. Simple things such as using corrosion resistant paint and repainting of shields can be effective. Design changes need to be made to the pin joints and clevises which are subjected to high wear rates to avoid corrosion in these areas. Advanced plating technologies and more corrosive resistant materials need to be employed to improve hydraulic cylinder life, which is by far the weak link in a shield's life line.

If the trend of increasing the width of the shields is going to continue, design changes or the use of innovative materials such as composites, will be required in the near future. The weight of 2.0 meter wide shields is already a problem in high seam operations.

Finally, the following recommendations are made as key points on how to ensure your shield design meets your needs and how to maximize its life once it is put into service.

1. Identify your goals in buying a bigger shield. Are you looking for more support capacity to improve ground control or is your primary goal to extend shield life and your previous support of lower capacity was providing adequate ground control, then consideration should be given to derating the shield. There is a clear correlation between increased loading and fatigue related failures. Derating a shield by reducing the operating yield pressure can provide an additional margin of safety and extend the life of shields where high component stresses are observed. If you need the extra capacity for ground control *and* you want to improve the life expectancy, then be prepared to pay a premium for the shield and insist on a conservative design.
2. Make sure performance testing is completed before production shield fabrication begins. Allow enough time in the procurement cycle for performance testing of a prototype shield to be completed *before* fabrication of the production shields begins.
3. Make sure performance testing is properly done. Performance testing must properly simulate the in-service load conditions. Avoid test procedures which do not properly transfer load to the caving shield - lemniscate assembly.
4. Consider testing a shield to failure. This will tell you what failures to look for and give you insight as to when to look for them, and should be considered even if this means paying extra for the performance test.

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5. Consider buying several extra shields. While this adds cost to the initial purchase, a good preventive maintenance program will pay big dividends in maximizing the life of a shield. For example, if 12 - 16 extra shields were purchased and recycled during each longwall move, problems such as leg cylinder rebuild and restoring pin tolerances to design specifications could be corrected as they arise, and preventive measures such as painting and crack inspection and repair could be done on a regular basis.
6. Avoid setting the shield with the canopy tip up. The worst case loading for longwall shields is to stand the support on the toe of the base. Most shields are equipped with a control system that controls the canopy capsule cylinder operation to allow the canopy to float as the support is being set to avoid setting the shield with the canopy tip up. *Do not deactivate* this system unless it is absolutely necessary.

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Shield Monitoring to Forecast Severe Face Weightings at the South Bulga Colliery, NSW, Australia

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ABSTRACT

The South Bulga Colliery longwall operation, owned and operated by Cyprus Coal Australia, Oakbridge Pty. Ltd., and located near Singleton, NSW, is currently operating under an average 80-ft-thick (24-m) sandstone unit residing immediately above the coal seam, resulting in sudden and severe face weightings. To date, nine severe weightings have occurred at the property (within Panels 3, 4, and 5), three of which resulted in face closures accompanied by significant equipment damage and several weeks of downtime. The remaining six events nearly resulted in face closures, incurring single-pass vertical convergence of up to 8 in (200 mm) along the panline.

In an effort to mitigate and/or eliminate the occurrence of these severe weighting events, mine management has established an in-house program to immediately address the geotechnical and operational mechanisms underlying rapid weightings, as well as specific measures to mitigate these events on future panels. In support of this in-house effort, NSA Engineering Pty. Ltd., is assisting the engineering staff at South Bulga in developing a more aggressive face monitoring program. The aim of this program is to provide weighting occurrence predictions sufficiently ahead of time to ensure that operational controls are optimized prior to attempting to mine through potential weighting areas. This paper reviews the occurrence of weightings to date at the South Bulga Colliery, as well as the latest developments in shield monitoring technologies to forecast these events in near-real time.

INTRODUCTION

Starting in late 1994, the South Bulga Colliery is currently one of Australia's most productive longwall operations. Mining conditions have generally been very favorable to high-production mining, with cover depths considerably less than 600 ft (180 m) over a majority of the panels mined thus far,

and competent sandstone comprising the immediate roof within the gates and along the faceline. Although high-quality roof conditions have expedited mains and gate developments, the massive nature of the immediate roof sandstone has also created severe periodic weighting events along the longwall face in recent panels.

Over the course of longwalling in Panels 3, 4, and 5, nine major weightings have occurred along the faceline. Figure 1 shows the face locations of all reported weighting events including the nine major weightings. Three of these events resulted in face stoppages due to the inability of the shearer to traverse the face beneath collapsed shield canopies. Face recovery was delayed by one to two weeks in each case, resulting in lost production, equipment damage, and the exposure of mine personnel to numerous face recovery hazards. Each closure event was estimated to cost the operation well over a million dollars (AUS) to re-establish face supports and resume mining — an expense that could well have been realized for *each* of the major weightings experienced had any *minor* operational delays been incurred during face convergence. In addition to the costs directly attributable to the downtime incurred during each event, additional costs to the mine have been realized in the form of reduced cutting heights, shield leg rebuilds for failing valves, new large-capacity shield purchases, and associated man-hours spent investigating the maintenance and adequacy of the face equipment, mechanisms related to the failures, applicability of mining geometries, potential spontaneous combustion issues, etc.

In an effort to mitigate and/or eliminate the occurrence of severe face weightings at the South Bulga Colliery, mine management has established an in-house program to evaluate (1) geotechnical and operational mechanisms underlying rapid face weightings; and (2) those specific measures that must be taken to mitigate the occurrence of rapid convergence events in future panels.

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In support of this in-house effort, NSA Engineering Pty. Ltd., is assisting the engineering staff at South Bulga in developing a more aggressive face monitoring program. The aim of this program is to provide weighting occurrence predictions sufficiently ahead of time to ensure that operational controls are optimized before mining through potential weighting areas. Toward this end, NSA has developed customized face monitoring software, integrated within the current face data collection and archiving system, to evaluate shield pressure, cycle status, and convergence information in a near-real-time format such that mine management may make timely and knowledgeable decisions regarding face cutting activities. This information is used to develop time-weighted-average plots of shield loading and deformation behavior from which to **forecast** severe periodic weightings.

Further supporting the development of the monitoring program, NSA also investigated the weighting events experienced to date, including a review of the site geology, development of a proposed failure mechanism for periodic caving, analyses of maximum cantilever and bridging spans for the main roof sandstone, and a review of recommended monitoring options for the property in addition to the planned face monitoring.

REGIONAL AND LOCAL GEOLOGY

The mine is situated in the Northern Coalfield of the Sydney Basin, approximately 14 miles southeast of Singleton, NSW. The current seam being extracted is the Lower Whybrow Seam within the Whittingham coal measures. The local and regional geologic/geotechnical setting of the South Bulga operation provides the basis for characterizing the immediate and main roof failure modes observed to date within the first five longwalls mined. Correlating key parameters describing the setting with periodic and severe weighting occurrences provides not only a general understanding of weighting mechanics, but also the means to forecast potential problem areas in future developments.

Major Geologic Structures

The South Bulga property is bounded by a number of major structural features, shown in Figure 2, which appear to control the development of jointing along the gates and within panels (joint orientation, frequency, dominant set distributions, persistence, etc.) and, ultimately, the geometry of most weightings encountered to date. These bounding features include the following structures:

- **2 West Mains Reverse Fault Zone:** Bounding the 2 West Mains developments is a well-developed northeasterly trending reverse fault zone, which has impacted the final stop-lines of Panels 4 and 5 where the fault offsets have ranged from 9 to 16 ft (2.8 to 4.9 m). Jointing in the vicinity of these reverse fault features sub-parallel faulting and/or runs nearly perpendicular to the fault dip, appearing to be related to the regional stress regime responsible for the faulting.

- **N-S Reverse Fault Along Western Anticline:** Bounding the western edge of the current panel section is an anticline feature, including a north-south trending reverse fault zone approximately 65 ft (20 m) wide and exhibiting up to 8 ft (2.4 m) of throw. Mining has encountered the easterly limb of the anticline along the inby section of the Panel 6 gate development, with ground conditions being adversely impacted by changing grades, heightened water production, and significantly higher frequencies of jointing in the roof (some of which may extend into the coal seam). This feature has also been responsible for improving caving conditions along the longwall in Panel 6 due to the increased jointing within the overlying main roof sandstone.
- **Longwall 5 and 6 End-Panel Normal Faulting:** Bounding the southern extent of longwalls 5 and 6 is a west-by-northwest trending normal fault system, presumed to be associated with the western anticline uplift region.
- **Loder Anticline Extension Bounding East Mains:** Abruptly bounding the eastern portion of the property is a sudden change in grade brought on by the extension of the Loder Anticline. This major regional feature has forced mining developments to the southeast, with considerable northwesterly jointing mapped throughout the early developments of the east mains. As with the western anticline, the eastern bounding anticline structure will significantly impact the immediate ground conditions and eventual caving characteristics of adjacent panel developments.

To date, no major panel structures have been encountered to impede longwalling within the section encompassing Panels 1 through 6, including faults, shear zones, and/or coal wants produced by channel scouring along the bottom of the overlying sandstone unit. Gate and typical face conditions have been excellent, with no reported ground falls (aside from face weightings) occurring during development or longwall mining.

Gate Road Joint Mapping

The following observations can be made concerning jointing and joint trends across the property...

- The distribution of jointing across the property is comprised of essentially two predominant trends; a general northwesterly trend (regional folding) that is strongly related to the panel-section-bounding anticline strikes, and a less frequent trend (depositionally controlled) running oblique to the major trend, along an easterly to northeasterly strike. The location of joint zones, the predominant joint orientations, and the joint frequencies are illustrated in Figure 2.
- In the vicinity of the *major* weightings, which began within the inby region of Panel 3, jointing is periodic and, at times, somewhat infrequent. The majority of jointing along the eastern edge of the weighting area is oriented northeasterly, whereas along the western panels (Panels 5 and 6), the northwesterly trend predominates.

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- Most joints are closed, with little or no fill material observed along the exposed trace. Many have served as water conduits at one time, but are quickly drained of any trapped water shortly following development mining.
- Lateral persistence of jointing is evident for several meters to tens of meters within the mapped gate roads. Major joints commonly extend across the entire gate, and can be traced through the upper portions of the coal seam as well. The vertical persistence of these joints above the coal seam is unknown, but is presumed to extend through the lower portion of the thicker sandstone sequences.

Although jointing in the immediate roof and coal seam may not be a singular indicator of the quality of caving to be expected over an area, joint trend mapping does characterize the mechanical nature of the overburden. Quality caving (no major weightings) has been experienced in areas of thinning sandstone coupled with dominant joint orientations running subparallel to the faceline. The worst weightings to date have been in areas with thickening sandstone and dominant jointing at much higher angles to the faceline—subparallel to the direction of retreat—including the severe weighting areas and the outby areas of Panels 1 and 2 where wind blast problems were reported.

In Situ Stress

Caving conditions may be greatly influenced by the presence of a significant horizontal stress field (generally greater than two times the assumed vertical stress), generally creating premature failure in highly stressed strata ahead of the face at or near gate junctions. Such ground failures about the longwall periphery have not been reported at the South Bulga property, nor have any other stress-indicating roof and/or floor failures been observed. The indication is that the mine is subject to a low in situ stress environment. A review of regional stress field measurements suggests the maximum horizontal principal stress magnitude for the South Bulga property should fall within the range of 725 to 1,740 psi (5 to 12 MPa), or two to five times the assumed vertical stress (1). However, a 1998 measurement taken at the South Bulga property within the roof of the 14 C/T in the Panel 5 headgate development indicates the horizontal principal stress component magnitudes are very similar, and are just over half the measured (and assumed) vertical stress, with an average maximum component orientation of N71°E, which is approximately aligned with the dominant joint orientation at this test location (2). Although these stress magnitudes run contrary to the reported regional stress regime measurements and the early anticipations of the in situ stress condition, they are borne out by the observations to date underground.

Considering the influence that major structures have on stress orientations in low-stress environments, it is likely that the major horizontal stress component varies across the property coinciding with variations of dominant joint orientation and frequency.

Stratigraphic Column

Clearly, the biggest geotechnical contributor to face weightings at the South Bulga property is the existence of the massive sandstone unit overlying the Lower Whybrow Seam. The competency and consistency of this unit varies; however, in general, bedding, and natural parting planes become more evident in the core samples observed toward the eastern panel developments, with less bedding and stronger cementing within the unit defining the western coreholes. Vertical jointing has rarely been encountered within the most recent corings, with very high RQD's evident throughout the surface-to-seam section.

A brief description of a portion of the SB52 drill hole core recovery, located within and midway down Panel 6, is presented as a basis for understanding the stratigraphic setting impacting weightings in the area of Panels 4 and 5. The stratigraphic section for hole SB52 is illustrated in Figure 3.

- The Lower Whybrow Seam is approximately 8.2 ft (2.5 m) thick and may be characterized as moderately strong coal, despite a very well developed face and butt cleat system (0.5 in (1 cm) spacings). UCS values range from 145 to 3,300 psi (1 to 22.8 MPa). Coal stiffness (elastic modulus) is reported at 478,000 psi (3.3 GPa)—an average stiffness for most regional coal seams. Although these average values indicate a weaker coal seam, observations along the face and tailgate areas indicate relatively minor coal failures within the first 3 ft (1 m) of ribline. Face deterioration during weightings has also been reported to be minor, and it is assumed that front abutments reside nearly on the face—no measurable or observable yield zone development is evident, despite the presence of a weak parting midseam.
- The immediate roof overlying the Lower Whybrow consists of 0.7 to 6.5 ft (0.2 to 2 m) of interbedded mudstone and sandstone, overlain by approximately 30 ft (9.5 m) of competent fluvial sandstone. Thin muddy bands exist within the lower and middle sections of this sandstone column, though bedding plane failures are minimal (RQD's ≥ 100). Immediate roof caving would be expected to initiate 3 to 5 m into this sandstone unit above the top of the seam. Although more than 15 bedding planes per foot have been mapped in the lower interbedded mudstone/sandstone unit (lower 6 to 10 ft (2 to 3 m)), only 1 bedding plane per foot can be discerned on the average in the main body of the sandstone.
- At approximately 30 ft (9.5 m) above the seam, there exists a 1.6-ft- (0.5-m-) thick shale unit, exhibiting little bedding plane failure. This unit has been described as highly variable across the property and is best represented as a series of occasional units within the bottom portion of the sandstone column. These units are the major failure planes controlling immediate roof caving and help define the caveability of the entire sandstone unit to the bottom of the Upper Whybrow Seam. The non-uniform distribution and occasional occurrence of these potential parting planes within this massive sandstone body may explain why some

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periods of major face weighting appear to have been skipped over.

- Immediately above this shale unit sits 54 ft (16.5 m) of massive sandstone (this portion of the unit covers the entire mine property developed to date). Of this thickness, at least 33 to 40 ft (10 to 12 m) of the core is virtually intact and massive, showing no bedding plane failures or weaknesses. This section of the Upper-to-Lower Whybrow innerburden is a major contributor to the weightings witnessed to date. UCS values range between 7,250 to 10,875 psi (50 to 75 MPa) for the main roof sandstones, with elastic moduli ranging from 2,000,000 to 3,000,000 psi (14 to 22 GPa).
- Approximately 3 ft (1 m) of Upper Whybrow coal provides for the major bedding failure plane above the lower sandstone innerburden units. Immediately above the Upper Whybrow is 9 to 12 ft (3 to 3.5 m) of soft strata grading upward from carbonaceous mudstone and shale to mudstone and then siltstone/sandstone. A 5-ft (1.5-m) sandstone is next in the column, overlain by 90 ft (27.5 m) of predominant shale strata with interbedded thin sandstones and siltstones. Despite the significant bedding present in this upper shale member, RQD's remain high, with no signs of weak bedding present.

The strata overlying the Lower Whybrow Seam are generally characterized by a strong, massive sandstone residing immediately above the seam, overlain by moderately strong sandstone and shale sequences (though intact strengths of individual units may be quite high compared to other units in the column). Shale units within the thick sandstone overlying the Lower Whybrow Seam, the Upper Whybrow Seam, and several coal seams and carbonaceous shale units provide for caving limits during longwalling. In the absence of these weak bedding planes, particularly within the massive sandstone immediately overlying the seam, caving is difficult, and periodic weighting is more severe.

The primary contributors to the weighting geotechnical setting are clearly the overlying sandstone unit(s), the minimal cover depth over a majority of the panels, and the presence of (or lack of) significant jointing in the roof. The depositional characteristics of the sandstone unit likely determine its ability to span large distances and store considerable amounts of strain energy.

HISTORICAL WEIGHTINGS REVIEW

A review of the weightings to date was conducted to better understand the sequence of events leading up to a major face stoppage. This type of information is useful in relating the characteristics of future shield loading and face convergence monitoring to the onset of severe weighting events. The following is a compilation of personal interviews with the mine's operations and engineering staff, in-house documentation describing the progression of weightings, and notes taken during general discussions on mining conditions since the onset of severe weightings at the beginning of Panel 3.

Longwalls 1 and 2

The first two panels were mined without any significant weighting event occurring. Shield yield valve degradation that occurred while mining Panel 2 is thought to have been the result of malfunctioning blipper valves, causing excessive use of the yield valves. In hindsight, the valve problem may also have been an indicator of minor periodic weighting impacting the face supports. Staff members that were on site when longwalling began recall hanging roof conditions at startup, to the point of potentially causing severe wind blasts (though none were recorded).

Longwall 3

A major weighting event occurred approximately 650 ft (200 m) from the panel start-line, resulting in **12 in (300 mm) of face closure in less than one hour**. The shearer was not stalled by the closure event, and the crew managed to mine through the weighting zone without incident. In addition, a subsequent event was also noted to have occurred 980 to 1,300 ft (300 to 400 m) from the start-line, which was also mined through without incident. As illustrated in Figure 1, face weightings continued throughout the remainder of Panel 3, though none were severe (not to say they would not have been had significant face delays occurred during a weighting). Note that the greatest frequency of weightings reported along the panel occurred within the zone of panel length most impacted by jointing—between C/T9 and C/T14.

A strict face management plan was developed and put in place to address future weightings. Following the first weighting, each shift "string-lined" the face to ensure straightness was maintained the remainder of the panel. Despite these efforts, periodic weighting was identified on approximate 65-ft (20-m) intervals, impacting an average of 30 supports at a time at varying positions along the face (a trend that continued within the major weighting areas of Panels 4 and 5).

Longwall 4

After only 115 to 130 ft (35 to 40 m) of Panel 4 had been mined, a portion of the major sandstone member above the seam failed resulting in a minor windblast. After nearly 590 ft (180 m) of panel retreat, periodic weighting became noticeable—occurring in 65- to 100-ft (20- to 30-m) periods. Major periodic weighting became difficult to define once mining had progressed outby the approximate 330-ft (100-m) cover depth line (around C/T12-13); however, minor weighting events continued to occur.

On May 17, 1997, the first face-stopping weighting occurred at South Bulga, involving approximately 30 shields toward the tailgate end of the face. Coincident with this event was the occurrence of a blown hose, forcing the face to stand idle for more than 90 minutes. In that time, the face converged to the point that the shearer could no longer travel the face, and mining was suspended. Collapsed shields could not be cleared for nearly a week, with an additional week spent repairing

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shield damages. During this time, no additional convergence was experienced along the face—all of the convergence was relegated to an approximate 30-shield section immediate to the initial weighting.

On June 6, a second face-stopping closure event occurred approximately 660 ft (200 m) outby the first event. Once again, operational delays in excess of one hour contributed to the eventual stoppage of the face, allowing sufficient face convergence to occur to preclude shearer passage. The face was down for over a week, yet no additional convergence was monitored adjacent to the section of shields impacted by the weighting. Approximately 30 shields were again involved in the weighting; however, this time the weighting occurred more toward the center of the faceline.

The location of these two events along the panel coincided with the face pulling even with tailgate entry crosscuts. It was initially thought that main roof failures were associated with the formation of fractures extending through the crosscuts; however, this observation was later dispelled as being coincidental as no other major weighting has occurred at a crosscut intersection. It was also felt that the weightings were associated with rapid face advance over the course of the week. The first two face-stopping weightings occurred at the end of high-production weeks (Saturday and Friday, respectively). It was felt that mining rates were sufficiently high to preclude regular and periodic failure of the main roof sandstone, resulting in an eventual override of load onto the face supports. This concept of the roof loading/failure mode has also been questioned due to the occurrence of weightings later in Panel 4 and then on Panel 5 on days throughout the week—including a Monday and Tuesday following non-production weekends.

Characteristic to both events was the formation of dense, subvertical fracturing immediately over the shields in the weighting zone. During the first major weighting, fracturing occurred mid-shield to the faceline, ultimately resulting in a major fracture development along the canopy tips. In the second major weighting, fracturing occurred over the shield legs, across the canopy, and to just within the face side of the canopy tips. The development of a major fracture line along the face lagged the closure event by several hours.

In response to these events, a complete review of the mechanical operations of the face equipment was immediately undertaken, component-by-component. Every yield valve was changed out, and extensive pressure testing of seals and valves was undertaken to ensure the face was up to specification. Blipper valve failures were identified, and a systematic leg change-out program was initiated. The face height was also dropped to 8 ft (2.45 m) to ensure that only the lower stage was being employed to set the support against the roof, thereby bypassing identified valve problems. Finally, the operational procedures for mining through weightings were updated and enforced throughout the remainder of the panel.

Two “near misses” also occurred later in Panel 4; one on July 21, approximately 1,310 ft (400 m) outby the second face-

stopping event, and one on August 13, approximately 820 ft (250 m) outby the first near-miss event. The only thing that distinguishes these events from face stoppages is that there were no operational delays that would allow the face time to converge sufficiently to halt shearing. In all other respects, these near-miss events were identical to those that resulted in face stoppages.

As illustrated in Figure 1, showing a compilation of weightings along the panel, based on Deputy's reports, a majority of the heavy weightings occurred where sandstone thicknesses were greater than 80 ft (25 m) and cover depths were in excess of 360 ft (110 m). Weightings did continue, however, along the length of the panel to the face stop line. This panel appears to separate the property into distinctly different jointing trends, as previously discussed in the above section on jointing.

It should also be noted that few weightings were noted in the daily reports until the longwall drew even with the goaf area created by the first three panels. The inby section of Panel 4, a 660-ft- (200-m-) wide face, was of a subcritical dimension for complete subsidence, suggesting that **the surcharge and/or periodicity of the strata overlying the Upper Whybrow Seam plays an important role in face weighting development**. As is shown in Figure 1, the inby region of Panel 5 experienced numerous weighting events, suggesting that the supercritical panel dimension provided for immediate involvement of the upper main roof strata.

Longwall 5

Longwall 5 was widened to 820 ft (250 m), including the addition of thirty 1,150-ton-capacity shields at mid-face (alternating with the original face supports). Numerous minor weightings were recorded immediately following panel startup (Figure 1), with an apparent “first-fall” occurring around 260 to 295 ft (80 to 90 m) of the start line. The first two major weightings (February 11 and February 26, 1998), both near misses, were located almost directly across from the first two face-stopping events on Panel 4, and were also separated by approximately 790 ft (240 m) of panel mining. A third near miss occurred March 12 approximately 555 ft (170 m) outby the previous weighting, and a fourth event occurred during the Easter break on April 12, nearly 1,180 ft (360 m) outby the March event. Mining proceeded through all four of these major weightings without incident; however, any one of these weightings could have resulted in a face stoppage had any type of operational delay occurred.

On April 21, just 360 ft (110 m) outby the previous major weighting, a face-stopping weighting occurred involving approximately 30 shields toward the headgate side of the panel. Once again, an operational delay was blamed for stalling the face at a critical point during face convergence (the delay lasted for just over an hour). Fracturing was again witnessed to occur over the shield legs to the canopy tip, and no additional convergence was experienced along the face. No significant weightings were experienced over the remainder of the panel.



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One additional factor contributes to difficulties in mining through weighting events—face width. The wider face requires additional shearer tramming per face pass, thereby extending the time required to advance the wall. A typical shearer pass currently requires 35 to 45 minutes, with four to six passes generally needed to clear a weighting area. Over the course of a major weighting, 2.4 to 3.5 in (60 to 90 mm) of closure can take place per shearer pass, with a rapid-onset event experiencing more than 8 in (200 mm) per shearer pass. Many of the near-miss events have just managed to keep the face ahead of critical total closures, suggesting that *the time-to-face-stoppage is nearly equivalent to the current cut-by-cut advance rate.*

In general...

- **All** major weightings to date would have resulted in face stoppages if significant operational delays had occurred.
- All three major face weightings stopped the face for 1 to 2 weeks, **yet no additional weighting impacted the face.**
- The events can occur very rapidly. The first two weightings were assumed the fault of improperly operating blipper and yield valves, though the third weighting was simply a rapid onset of load.
- No change in coal cutting is noted until they are in the weighting, when coal cutting becomes progressively easier.
- Fracturing of the roof starts over the shields and works toward the face, ultimately forming a major fracture just off the tip of the shields witnessed along face stoppages.
- Some face spalling is associated with the weightings, though prior failure of the coal seam, as the face advances into the weighting zone, is not evident.
- The front abutment distance is noted to exist 50 to 65 ft (15 to 20 m) ahead of the face. Approximately 33 ft (10 m) of front abutment zone was noted along the tailgate entry ahead of the face shortly following startup of the Panel 6 wall.
- No floor problems have been incurred at any stage of longwalling, including during severe weightings.
- Shields typically run at close to yield for days, with small increases in load accompanying weightings, putting many shields in yield at one time.
- A typical severe weighting will have more than 15 supports resting on the AFC, with 15 to 20 supports on either side at some stage of progressive closure, forming a trough of deformed supports. No weighting accompanies the adjacent areas despite the face standing for up to two weeks.
- The new 1,150-ton supports were involved in the third weighting event and made no apparent difference in the timing, extent, or magnitude of the weighting event.

MAIN ROOF CAVING CONDITION

Failure Characterization

To further characterize the major weighting events at the South Bulga property, NSA Engineering prepared a simplified analytical evaluation of cantilever and bridging beam spans

based on the general setting parameters provided by the mine engineering staff. To determine span lengths and associated strain energy levels, the Coal Bump Potential Evaluation Program (CBPEP), developed at the Virginia Polytechnic Institute and State University, Blacksburg, VA (with assistance from NSA staff), was used to evaluate a number of cases involving (1) thick sandstone roof, and (2) multi-layered sandstone roof residing on weaker, immediate roof materials (CBPEP ver.2.0; 3). The program implements analytical solutions for single- or multi-layered roof beams resting on single- or double-layer elastic foundations, subject to exponentially distributed abutment stresses, and allows for determination of critical roof span, induced foundation stresses, and strain energies stored in the roof and foundation. This evaluation was conducted to grossly characterize the distribution of maximum spans along the gate lengths as a function of cover depth and sandstone thickness, and to serve as an index for comparison to future weighting conditions along Panel 6. This approach does not account for the presence of jointing in the main roof or the periodic nature of the strata overlying the Upper Whybrow Seam, but does approximate the *intact rock* failure condition observed within the mine. The evaluation results, therefore, should not be viewed as *absolute* determinations, but rather as *relative* assessments of expected roof caving performance.

Both maximum cantilever and bridging spans for two simple roof cases—single-beam strong roof and layered-beam strong roof resting on a weaker foundation—were evaluated. These cases were examined to bracket the conditions described across the property to date by a number of investigators—ranging from strong, massive sandstone resting directly on the seam, and bedded sandstone resting on a weaker “scour” material. The following points stem from this evaluation...

- Cover depth has a profound effect on beam spans, reducing cantilever spans by nearly 40% over the typical operating range of depths at the mine (165 to 490 ft (50 to 150 m)), and bridging spans by nearly 50%.
- Sandstone thickness also has a profound effect on span length, but is not strongly controlled by single-unit thickness. Based on the input parameters listed, the weak roof layer has a very minor impact on the maximum span length for either the cantilever or bridging cases.
- Both cases for the cantilever model predict periodic spans similar to those observed underground for cover depths greater than 330 ft (100 m) (outby this cover depth periodic weighting has been reported difficult to measure) in the approximate 65- to 100-ft (20- to 30-m) range.
- Both cases for the bridging model also predict spans similar to those experienced during wall startup. Accounts vary, and include observations of the immediate roof failing as opposed to failure of the main roof sandstone, but most agree that the first major cave has been observed in the 200-to 330-ft (60- to 100-m) face distance range.

The following characteristics of the weighting regions versus calculated spans, cover depths, and sandstone thicknesses are evident...



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- The clear identification of periodic weighting for Panel 4 was noted to diminish considerably once longwalling had retreated beyond the 330-ft (100-m) cover line. This observation coincides with a sharp reduction in span length (40%) as cover depth and sandstone thickness drop more rapidly in this region. The 330-ft (100-m) cover depth line also limits the occurrence of major weightings on Panel 5.
- A sandstone thickness greater than 80 ft (25 m) appears to be involved in all of the major weightings for Panels 4 and 5, with calculated spans in excess of 80 ft (24 m) (corresponding to an approximate detached-block dead-load of 2,800 tonnes).
- No information was readily available for the first fall distance in Panel 4; however, the identified first fall on Panel 5 is well approximated by the bridging span calculation—approximately 260 to 295 ft (80 to 90 m).
- No *major* weightings are seen in the Deputy's records for the inby reaches of Panels 4 and 5, with all weightings occurring once mining had engaged the entire panel section goaf area. This would suggest that periodic caving in the strata overlying the Upper Whybrow is involved in the mechanics of sudden face weighting.
- Minor periodic weighting noted in Deputy's records for the inby section of Panel 5, as compared to little or no periodic events in Panel 4, indicates that Panel 4 was in a subcritical subsidence state, and that the strata overlying the Upper Whybrow were not fully involved with panel weighting until mining in Panel 5 was initiated.
- Bridging spans for Panels 4 and 5 also remain above 230 ft (70 m) for the major weighting zones, suggesting significant bridging may be occurring from the gate pillars to either end of the panel.

The observations in the modeling approximate those observed onsite, indicating this approach to characterizing the main roof failure mechanisms has merit when applied to forecasting future panel conditions.

Failure Mode Overview

Based on in-mine observations, review of the provided geotechnical information, and review of the personal accounts of weightings by both operations and engineering personnel, the following failure mode appears to characterize the manner in which weightings develop:

- Regular caving of the lower portion of the main sandstone occurs to a height ranging from 10 to 16 ft (3 to 5 m). This regular caving follows the shield line closely, rarely extending more than 5 to 10 ft (1.5 to 3 m) off the back of the shields and never breaking over the shields. Bulking of this broken material precludes visual examination of the hanging nature of the overlying roof.
- The upper 65 ft (20 m) of sandstone bends and detaches from along the bottom of the Upper Whybrow Seam, possibly shearing at the seam boundary to some point either ahead or even with the faceline. It is likely that lower portions of the main sandstone cave up to shale bands between the Upper and Lower Whybrow Seams,

sporadically distributed through the unit, possibly explaining why major periods of weighting appear to be interrupted. Major weightings are likely tied to areas where a majority of the sandstone unit moves as a cohesive mass toward the gob.

- As is indicated in FLAC modeling results produced by SCT(1), tensile failures initiate along the sandstone/Upper Whybrow Seam boundary, extending downward toward the Lower Whybrow Seam. The frequency (spacing) and extent of these tensile failures is a direct function of the thickness of the sandstone beam (laminated beam profile) hanging off the back of the face supports. It is also likely that the development of these failures is being enhanced by the softer overlying Upper Whybrow Seam, which will have the tendency to deform toward the gob more rapidly.
- As the cantilever beam forms, and the strata immediately above the supports are subjected to considerable compressive forces, subvertical shearing results at high angles toward the gob. Shearing has been observed to initiate immediately over the shield legs and progress toward the face as the weighting becomes more severe with successive cuts. No accounts of mining into damaged roof could be found, and no observations of broken or damaged coal have ever been correlated to impending face weightings to suggest failure occurring well ahead of the face. However, minor face guttering associated with shear plane development has been noted.
- As mining proceeds toward a critical block dimension, the rate of face loading increases substantially. Both shear and tensile failures accelerate to define the block from the surrounding strata. This process typically begins to accelerate within 33 ft (10 m) of the eventual failure limit, with severe weighting and face closure occurring within 16 ft (5 m) of the eventual roof failure boundary (commonly characterized by a major failure plane ahead of the canopy tips). Face crews have 3 to 4 hours to mine through the worst of the weightings (on an 820-ft-wide (250-m) face), with face stalls of greater than one hour generally resulting in a long-term stoppage.
- In all cases, the major failure plane into the roof was characterized by a steep angle ($> 80^\circ$ from horizontal) toward the gob, a failure plane location either along the shield tips or just ahead between tips and face, with the plane either developing at the time of the worst weighting or just after. All of the major failures have been through the intact portion of the rock mass. More than one operations person noted that the failure plane appeared to be related to the shearing of strata immediately above the shields, and that it commonly occurred toward the latter stages of the weighting. This would suggest that the failure occurs rapidly during successive cuts, rather than well ahead of the face. The lack of seam weighting and damage, as well as no floor failure along the face, would also suggest that the failure of the roof is local to the shield line (not well ahead of mining), and that the weighting event proceeds rapidly.
- Involvement of the strata overlying the Upper Whybrow Seam can only be surmised at this stage, but it is possible that periodicity in the manner in which these units fail is

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GEOGUARD™ SHIELD MONITORING SYSTEM

contributing to the face weighting phenomena as well. Extensometer data does indicate involvement of the strata above the Upper Whybrow, but not until the face is 65 ft (20 m) past the station—still within the typical weighting period distance. [It is very possible that the extensometer data represent typical caving conditions, and that other mechanisms are at work during a major face weighting. For example, sudden periodic failure of the upper strata may coincide with the maximum cantilever development in the lower thick sandstone unit overlying the seam, resulting in rapid weighting of an already loaded massive beam structure—loading that is rapidly translated to the face supports.]

- Another significant characteristic of the major face weightings is the localized manner in which the face is impacted. Generally, a section of 30 to 40 shields, out of 145 total shields, is severely loaded and/or involved in the face stoppage. During the stoppage, which may last one to two weeks, no additional weighting of the face is observed. Of the three face stoppages to date, one has occurred toward the tailgate, one in the center of the panel, and one toward the headgate. Although the weightings are tied to failure through intact rock along the face line, it would appear that the lateral extent of the failures along the face is structure controlled (i.e., jointing).
- Gas drainage holes, drilled on 330-ft (100-m) centers along the panel, may also impact caving by draining the overlying strata of a significant water content. The drop in pore pressure may minimize buoyant or “lubricating” stresses along pre-existing failure planes subject to shearing (shale units within the main sandstone and/or the Upper Whybrow Seam interface), producing greater friction resistance—increased normal loads and coefficients of friction along the failure plane.

Severe weighting events may be more closely tied to the caving characteristics of the strata overlying the Upper Whybrow Seam than solely to areas of varying sandstone thickness above the Lower Whybrow Seam. Subsidence reporting for the property indicates maximum short-term subsidence, following longwall passage, lags a distance of $0.7D$ (where D = mining depth). The drop to maximum subsidence is relatively steep over a short span, indicating the possible failure mechanism involved above the seam(s)—rapid, yet blocky failures. Short-term monitoring (daily) would be required to catch periodicity in the subsidence profiles.

It is the hope of the shield monitoring program to determine if yet-undetected changes in face loading may forewarn of pending events sufficiently to effect operational changes along the face and avoid stoppages altogether. If periodicity of the strata above the Upper Whybrow is found to be influential in the major weightings, it is likely that precursor information will be available to determine loading trends during longwalling.

Shield Monitoring Goals

Shield monitoring provides a large amount of information, and a major consideration is the selection of appropriate subsets of the data that directly apply to specific ground control problems at individual mines. The principal goal of the monitoring implementation was to provide an **early warning system** to alert the operator to an imminent occurrence of severe periodic weighting events. It was believed that the average loads or load increments experienced by the shields during each cycle could provide a reliable indicator of weighting conditions. Analysis of historical data could provide estimates of the weighting interval, and warnings could then be generated when the face approached the predicted weighting zone, or when loads became unusually high. Earlier research in this area had success using shield loads to predict ground conditions; however, a separate monitoring system was required, and the software analysis functions were separate and required significant operator time to process and interpret. At South Bulga, transducers were installed on every shield, and a monitoring and control network was already in place to permit accessing the data in real time from the surface. To obtain maximum advantage from the existing installation, it was decided to modify the various components into an integrated application that would combine the functions of monitoring, analysis, interpretation, prediction, and warning. The following section describes the structure of the GeoGuard™ shield monitoring system.

GeoGuard™ System Layout

Although GeoGuard™ is a software system that accesses real-time monitored data, the interface to the monitoring hardware is an integral part of the system and significantly influences the capabilities and performance of the software. Data from the shields are accessed using hardware and software components supplied by the shield manufacturer (Joy, in this case), who also provides an interface between the shields and an Allen-Bradley monitoring and control network. The shield data are periodically scanned and are transferred to a PLC using control programs written by mine personnel. These data include the pressures of both shield legs and shield convergence on selected shields. Also, the shearer position is captured, using two separate means, and stored in separate PLC locations. Normally, the data are updated every 2-1/2 minutes; however, the shield data are also updated when each shield is reset.

GeoGuard™ connects to the Allen-Bradley network using the RS-Linx software bridge, available from Rockwell Software. RS-Linx permits accessing the network via a data highway, serial, or Ethernet connection. At South Bulga, the connection was made through the mine's Ethernet network, and the PLC data were accessed through an Ethernet-to-data highway gateway. GeoGuard™ communicates with the RS-Linx program using software calls to set up data pathways and initiate the transfer of data to and from specific PLC memory locations. The GeoGuard™ and RS-Linx software are

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installed on a dedicated PC, and the system runs continuously, capturing data in real time.

The GeoGuard™ software consists of three primary sub-systems: data collection, analysis, and utilities. Specific functions within these primary sub-systems are described below and summarized in the following table:

Data Collection	Analysis	Utilities
Configuration	Cycle calculation	Password checks
On-line collection	Gridding & 3-D plotting	Panel coordinates
Write data files	2-D trending	Face positions
Real-time display	Weighting intervals	Units
	Alarms	On-line help

Data Collection

Configuration involves specifying the location of the data on the Allen-Bradley network and mapping each monitored point (shield leg, shearer, etc.) to a specific data address. Multiple data blocks (PLC files) are supported, and scaling conversions can be defined to display the data in engineering units. Monitoring of individual sensors can be disabled to remove known malfunctioning points from the analyses. When the system is on-line, all defined data blocks are read at a user specified scan interval (normally 1 minute), and the data are written to daily files for a permanent record. The data are monitored to identify when each shield is cycled and time-weighted average pressures (TWAP) and other parameters (set pressure, maximum pressure) are calculated for each cycle and saved in a data file. The most recent data are stored for access by the real-time graphics display, which shows the current leg pressures and other parameters for the entire face. The current face position is compared with predicted weighting locations for each shield and a multi-stage alarm is generated when the face approaches a weighting zone and/or if the shield pressure exceeds a pre-set threshold. The weighting interval predictions and high-pressure threshold values are automatically updated to enable the warning system to adapt to changing ground conditions.

Analysis

Cycles are detected when the average shield leg pressure drops by a specified amount from one data reading to the next. While monitoring, the pressure values are accumulated, and at the end of the cycle, the time-weighted average pressure (TWAP) is calculated and values for TWAP, set pressure, maximum pressure, time-weighted average convergence (TWAC), cycle start and end times, and cycle duration are written to a file. The cycle detection algorithm filters excessively short or long cycles and cycles having abnormally low pressures. A two-dimensional trending function is included that permits trending any of the monitored data points or any of the cycle parameters versus either time or face position. The shearer position and face position trends can also be plotted and superimposed on the other data.

Three-dimensional plots of the shield loading history are valuable for evaluating loading patterns and identifying correlations between loading characteristics and local geology or other factors that might contribute to excessive weighting conditions. Three-dimensional plotting is based on rectangular grids that extend across the panel and over a user-specified interval along the panel. Grid files, generated from the cycle data files, are used for plotting any of the cycle parameters, including the loading increment (TWAP minus set pressure). Several options are available for controlling the appearance and accuracy of the generated grids. This plotting module also permits viewing individual grid files superimposed on base maps of mine layout or geology. Both the horizontal and vertical viewing angles can be varied, and options are available for zooming and scaling. Additionally, the coordinates of plotted features and distances between features can easily be displayed using the mouse. Plots can be saved as high-resolution bitmap files or sent directly to the printer. Grid files are saved in text format for use by other processing or analysis programs.

A major portion of the analysis sub-system is the weighting analysis and prediction functions. The loading history is analyzed, either on-line or by reading the data files, to identify the pressures accompanying peak loading zones and the intervals between zones. The locations of future weighting zones are determined using the average of the historical weighting intervals. The alarm logic is based on the proximity of the face to predicted weighting locations and the current loading intensity on the shields. Alarms are displayed on the GeoGuard™ computer, and the alarm value, as a percentage, can be written to a PLC memory register for access by other control or warning systems.

Utilities

The system automatically starts up after a power failure and is password protected to prevent unauthorized users from modifying the configuration or changing any information that would compromise correct operation of the system. Certain non-critical features, such as trending and real-time displays can be accessed without a password. The monitored data are stored with the date and time when the data were collected; however, the system also needs to know the panel advance history (face position versus time) and the location of the panel relative to the mine coordinate system. Facilities for entering these data are supported, including graphical feedback for ensuring correct data entry. The system can be operated using standard metric units (MPa, mm, m) for pressure, convergence, and coordinates, standard English units (psi, in, ft), or custom units (for example, bar, cm, etc.). An on-line help file is provided that covers installation, operation, configuration requirements, and suggested operating procedures.

Example Data and Results

A sample set of data, collected by the GeoGuard™ system, was obtained from the mine and analyzed to fine-tune the cycle detection algorithm and develop the weighting prediction sub-system. The data span a time period of about 1-1/2

months and a face advance interval of 430 m. Figure 4 is an example plot from the data trending section of GeoGuard™ and shows the raw leg pressures for shield 90 (of 145 total) over a 12-hr period. Superimposed on the plot are the calculated setting and maximum pressures and TWAP for each cycle and the shearer location.

The weighting analysis sub-system analyzes the TWAP values to determine the average spacing between pressure peaks for each shield. These average spacings, projected ahead from the latest pressure peak, represent the estimated or predicted location of the next pressure peak. Peaks are identified when the TWAP value of a given cycle is greater than that for the two neighboring cycles. A modified rolling average and standard deviation of the TWAP values are calculated after each cycle is completed, and a pressure threshold is calculated as the average TWAP plus some fraction of the standard deviation. Those peaks having pressures greater than the threshold are used for determining the average weighting interval. Filter values are also specified to disregard excessively short and long intervals. Figure 5 is a plot of the latter portion of the shield 90 cycle data and shows the average TWAP and pressure threshold ($= \text{average} + 0.8 * \text{standard deviation}$). Also shown on the figure are the projected intervals ahead of each pressure peak. A modified rolling average of the inter-peak spacings is maintained and updated when each new peak is detected. As shown at a face position 440 m, the prediction interval is advanced, in integral multiples, when no pressure peaks are detected. The rolling average technique, used for calculating the weighting interval and pressure threshold, was implemented to enable the analysis to adjust to changing ground conditions over time. Figure 6 shows a three-dimensional plot of TWAP for the same interval as plotted in Figure 5.

When on-line, GeoGuard™ reads the data from all defined data blocks at a user-specified scan interval. Data changes are appended to a daily data file, and the shield data are analyzed to calculate cycle parameters and weighting intervals. The cycle data are also written to a data file whenever a completed cycle is detected. An on-line alarm system, currently under development, is implemented to monitor each shield for two conditions: (1) proximity of the shield (or face) to the predicted weighting location; and (2) average shield load above the pressure threshold. Alarms are structured using a three-stage traffic-light model consisting of a normal (green light) condition, a cautionary (yellow light) condition, and a warning (red light) condition. An on-line alarm display is provided that shows the current color-coded alarm condition for each shield and a graphical depiction of the shield status with respect to distance to the next predicted weighting location and current average pressure relative to the pressure threshold. Values are assigned to each alarm level, and the alarm values for each shield are summed to provide an overall alarm value. This overall alarm value can optionally be written to a defined PLC register to permit access by the mine's control system for activating visual and/or audible alarms at the control center and at the face.

The South Bulga Colliery experienced a large number of periodic weighting events during the mining of three longwall panels. Three of these events resulted in significant convergence and required over a week's delay in each case to free the shearer and up to 30 affected shields. The primary factor contributing to such high-intensity weighting is the combination of a thick competent sandstone roof and a relatively shallow depth of cover. Shield loading results from periodic caving of the main sandstone roof that occurs as a cantilever beam-type failure. High loads normally occur only when the main roof breaks, and potentially adverse weighting conditions may exist for 3 to 4 hours until mining advances past the weighting zone. Continuous advance through the weighting zone is critical – the face-stopping events were associated with the face sitting idle for approximately one hour due to mechanical breakdowns.

In an effort to mitigate or eliminate these severe weighting events, mine management established a program to assess the mechanisms underlying rapid weightings and to develop specific measures to mitigate these events in future panels. NSA Engineering Pty. Ltd. assisted in this effort by implementing a more aggressive face monitoring program based on NSA's GeoGuard™ software linked with an existing industrial monitoring system. The aim of this program is to provide weighting occurrence predictions sufficiently ahead of time to ensure operational controls are optimized before attempting to mine through expected weighting areas.

The GeoGuard™ system monitors shield-leg pressure data continuously and automatically identifies periodic loading conditions and weighting intervals. A three-stage alarm system is implemented to warn when shield loads exceed a pressure threshold and when the face is nearing a predicted weighting location. The system adapts to changing conditions to automatically compensate for changing depths and roof thicknesses over the length of the panel. Many options are available for controlling data collection, analysis, and presentation, and results are permanently stored to support detailed evaluations of specific events.

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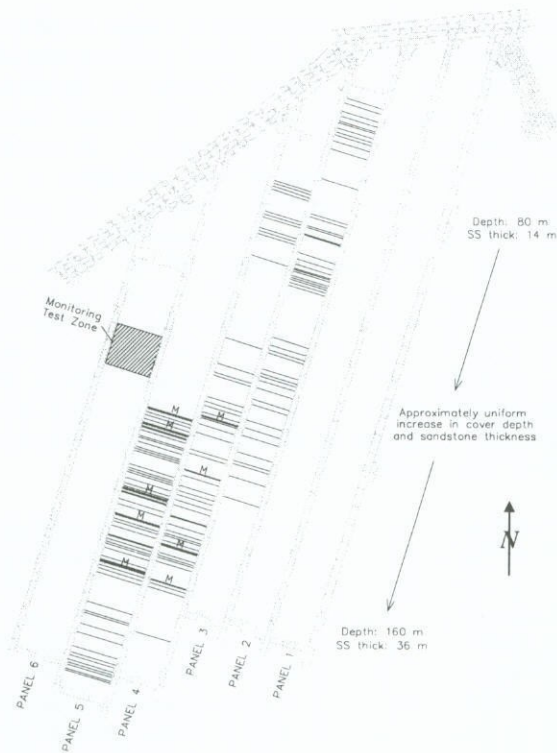


Figure 1. Face locations of reported weighting occurrences. Major weightings labeled "M".

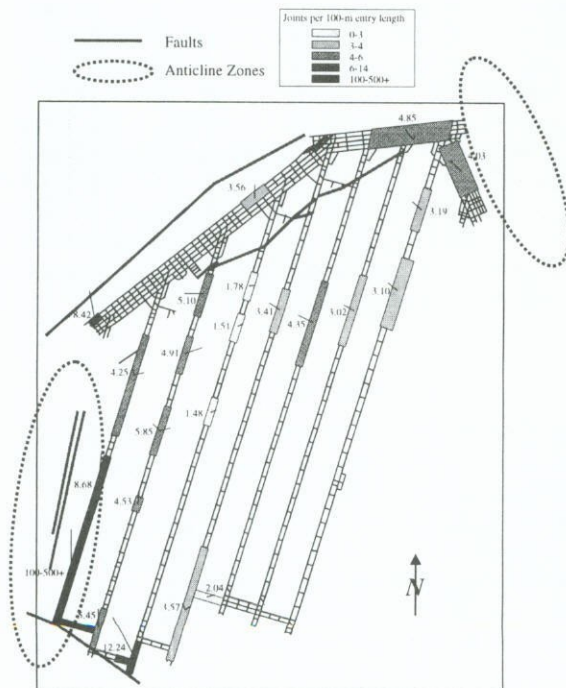


Figure 2. Locations of major faults and anticline structures and locations of jointed zones showing joint frequency and predominant joint directions.

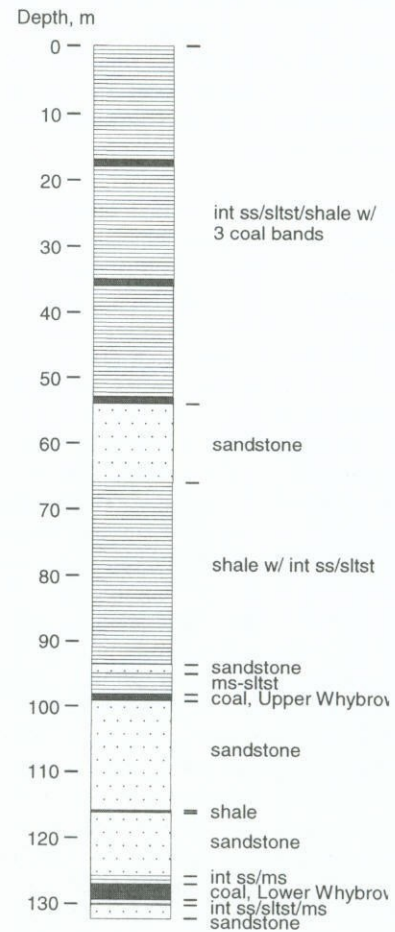


Figure 3. Stratigraphic section of roof above Lower Whybrow Seam.

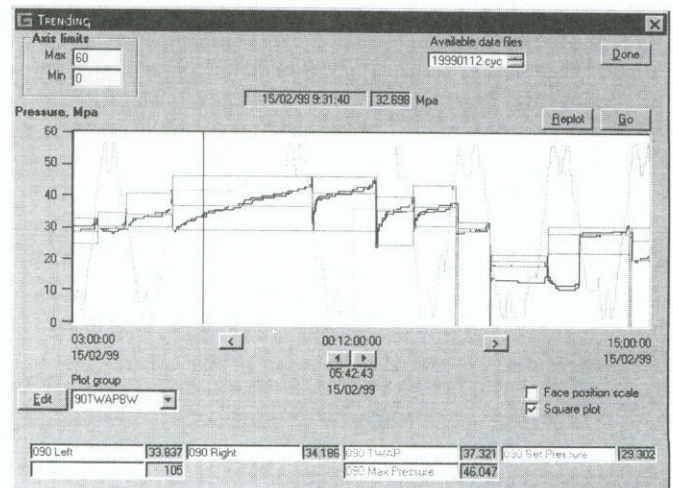


Figure 4. Two-dimensional GeoGuard™ trend plot of leg pressures for shield 90 including calculated cycle parameters and shearer position.

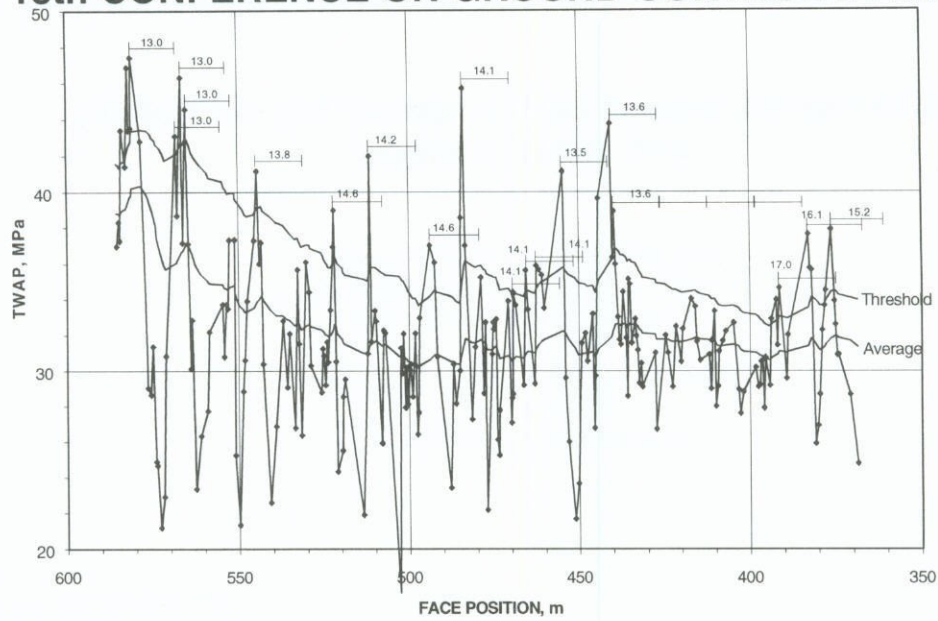


Figure 5. TWAP weighting history for shield 90 showing peak pressures, above the threshold, used for determining the average weighting interval and the calculated average intervals for each peak.

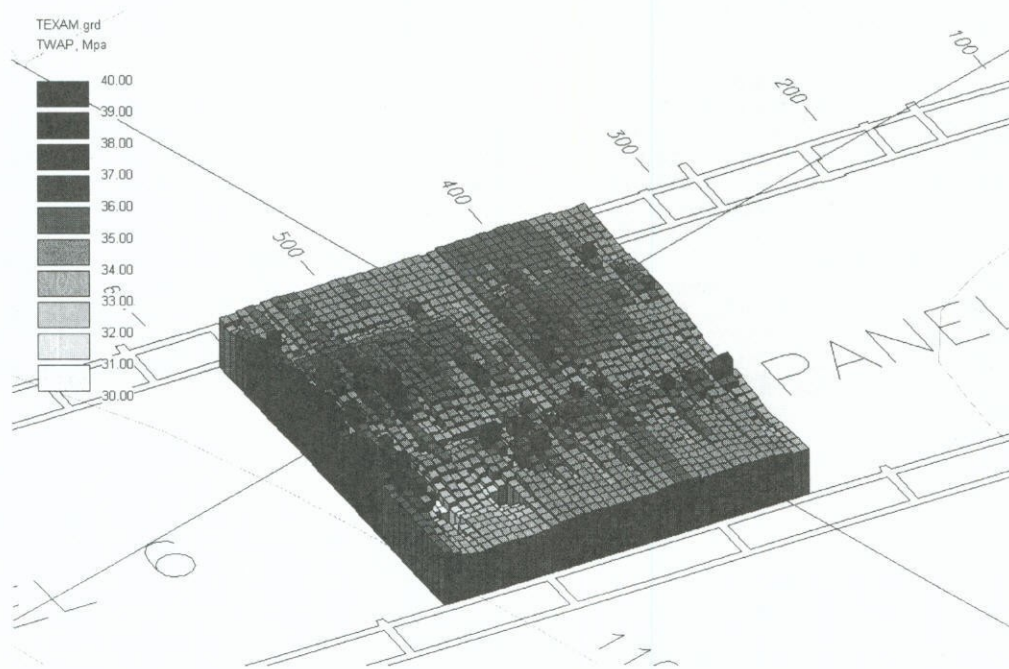


Figure 6. Three-dimensional GeoGuard™ plot of Time-Weighted Average Pressure (TWAP) over a period of 22 days (210 m).

Monitoring of Longwall Seal Behaviour for Permeability and Stress Changes

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ABSTRACT

Dartbrook Mine (Australia) works the Wynn seam, which has a moderate propensity for spontaneous combustion. Prior to any mining a commitment was made to place a segregation barrier pillar between longwall blocks 4 and 5. This was to segregate goaves and improve management controls in the event of a spontaneous combustion event within previous longwall goaves.

An alternative to the segregation barrier known as a Consolidated Barrier was developed; firstly as a concept, then computer modelled and finally field tested. It consists of a wider than normal chain pillar with concrete Mega-Seals placed in cut-throughs. Permeability and stress measurements were conducted in the surrounding coal of the seal. Remote monitoring took place as the longwall face approached and then passed by the seal. Results indicate that as the longwall retreats past the seal, horizontal stress increases. The additional stress leads to an increase in confinement and a subsequent reduction in permeability.

INTRODUCTION

Dartbrook Mine is a modern longwall mine in the Upper Hunter Valley of New South Wales, Australia. The Wynn seam is currently being mined using retreating longwall methods. The longwall takes out a four metre thick working section from a coalescence of three seams. Enveloping the mine workings is coal of inferior quality. There is some 3-4m of coal in the floor and approximately 14m of coal in the roof. The mining environment is subject to a moderate gas content and a spontaneous combustion risk. The extra coal left in the goaf increases the risk of spontaneous combustion. The seam gas is predominantly Carbon Dioxide which assists with goaf

management by inertinisation. A gas and ventilation management plan is active at the mine. Significant ventilation velocities and pressures are present in the mine.

The Wynn seam is a low ash, low sulphur, and thermal export quality coal. It is a coal of variable strength, with laboratory UCS ranging from 10 to 40MPa. In general it is a fairly massive coal by nature.

A Segregation Barrier, i.e. solid coal barrier, between longwalls 4 and 5 was originally planned prior to the commencement of mining operations. The purpose of this barrier was to reduce the likelihood of spontaneous combustion in old goaves. To construct this barrier pillar an extra two heading panel would have to be developed at a very large cost to the operation. This cost was thought to be unnecessary and following four years of successful mining experience an alternative was desired.

For spontaneous combustion to occur, coal requires oxygen in certain volumes and percentages. A segregation barrier is considered a safe option to segregate old goaves because coal has a very low permeability. Thus not allowing sufficient oxygen into the goaf to initiate or sustain spontaneous combustion. The proposed alternative the Consolidated Barrier concept would only be successful if it could be shown to have a permeability rate that did not initiate or sustain spontaneous combustion.

To prove the suitability of the Consolidated Barrier, it would need to be demonstrated that the mega-seal would remain intact and that permeability would not permit the risk of spontaneous combustion.

The concept of a longwall seal

constructed large enough to maintain its integrity long after it has been left in the goaf was perceived. The expectation was that the broken and fractured coal surrounding the seal would suffer confinement around a stable seal. Previous laboratory and field work support the belief that confinement (increased stress) reduces permeability.

This confinement process would in turn reduce the level of permeability. Thus reducing the potential of spontaneous combustion.

For the Consolidated Barrier proposal to be successful it would need an area of relative stability for a seal to remain intact. This stability can be provided through an adequately sized pillar. A computer model was undertaken and it recommended the appropriate width of the panel, i.e. 50m centres at 300m depth of cover. (Gale, 1998 Company Report)

Sprayed concrete seals were in use at the mine and it was decided to use this technology for the trial seal. A 10m long semi-rigid seal is expected to provide a ventilation barrier that is as effective as intact coal barrier.

This paper reports an extensive underground monitoring program. Instrumentation was installed in the coal ribs and within the seal itself to measure permeability, stress changes and convergence. It should be noted that while the results support the original concept, the premature failure of the remote cabling did not allow for long term monitoring, as was first planned. Further testing of this proposal is envisaged.

PREVIOUS FIELD AND LABORATORY WORK

The sensitivity of coal permeability to confining pressure is well recognised and has been demonstrated in numerous published field and laboratory studies. A reduction in permeability of several orders of magnitude can be achieved with laboratory samples with relatively modest changes in confining pressure.

Indraratna et al (1998) conducted laboratory tests on intact and failed coal specimens using air and water. They report that the permeability of both intact coal and failed coal is sensitive to confining pressure. Their results are summarised in Figure 1.

For intact coal samples, they found that a change in confining pressure from 500kPa to 2000kPa caused a reduction in permeability of about two orders of magnitude. For fractured coal samples, they found a change in confining pressure

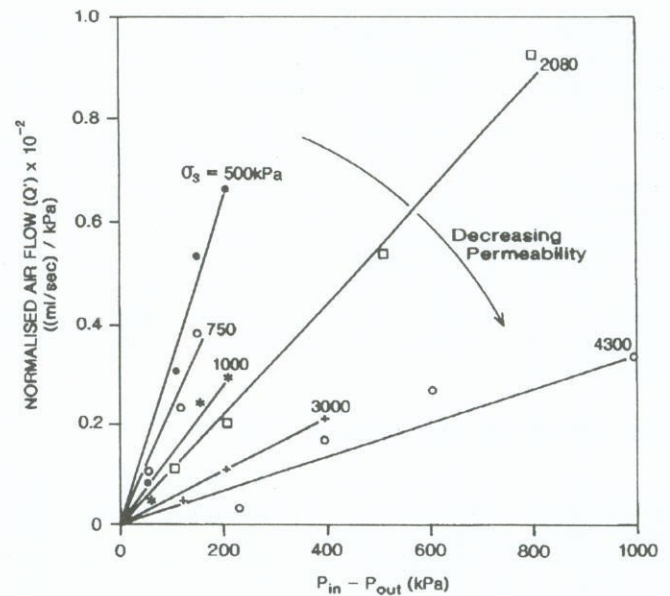


Figure 1 Variation in normalised flow rate (Q') against pressure difference for various confining pressures. After Gale, Indraratna and Haque 1998.

from 500kPa to 4300kPa caused a reduction in laboratory permeability of about one order of magnitude.

Figure 2 (Jeffrey et al, 1997) shows field results which indicate that a 7MPa increase in stress is likely to cause a reduction in permeability in intact coal of the order of 2 to 3 orders of magnitude.

Clearly, this phenomenon (confinement reducing permeability) can be used to reduce ventilation leakage around barrier seals if the seals are designed to generate sufficient confining pressure against the rib and roof strata. Reduced permeability in turn reduces the oxygen available to initiate or sustain spontaneous combustion.

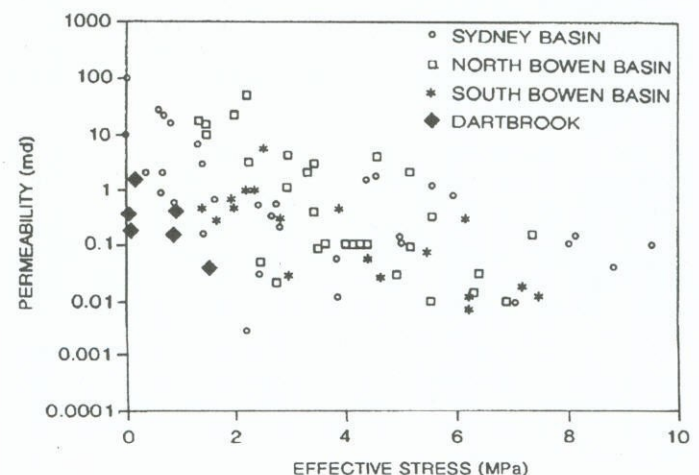


Figure 2 DARTBROOK MINE Dartbrook well test results compared to data from other coal basins in Australia.

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Balusu 1998 (Company Report), estimated values of permeability that would be associated with Spontaneous Combustion. His work is intuitive only, i.e. rule of thumb. He suggests that permeability levels of the order of 0.5D would be required for a serious spontaneous combustion risk. A value in the range of 1000mD would allow a low risk of a change to the gas environment around the immediate seal environment and not the deep goaf. Values measured during this monitoring program are presented in Table 4 by way of comparison.

SITE LOCATION AND PREPARATION

Figure 3 shows the mine layout and the test sites that were monitored. The main test site was located at 8 C/T in the tailgate of Longwall 2. The chain pillar is 30m wide, measured rib to rib. The depth of overburden at the site is approximately 225m. The overburden has a high proportion of coal giving a geophysically estimated average density of about 2.20t/m³.

Shortly after the face of Longwall 1 had retreated past the site, a Tekblend mini-seal was placed in the cut-through. Some additional rib bolts were installed prior to placement of this seal. At the completion of Longwall 1 and before Longwall 2 started, the ribs of the cut-throughs were treated with polyurethane to reduce the permeability of the coal ribs.

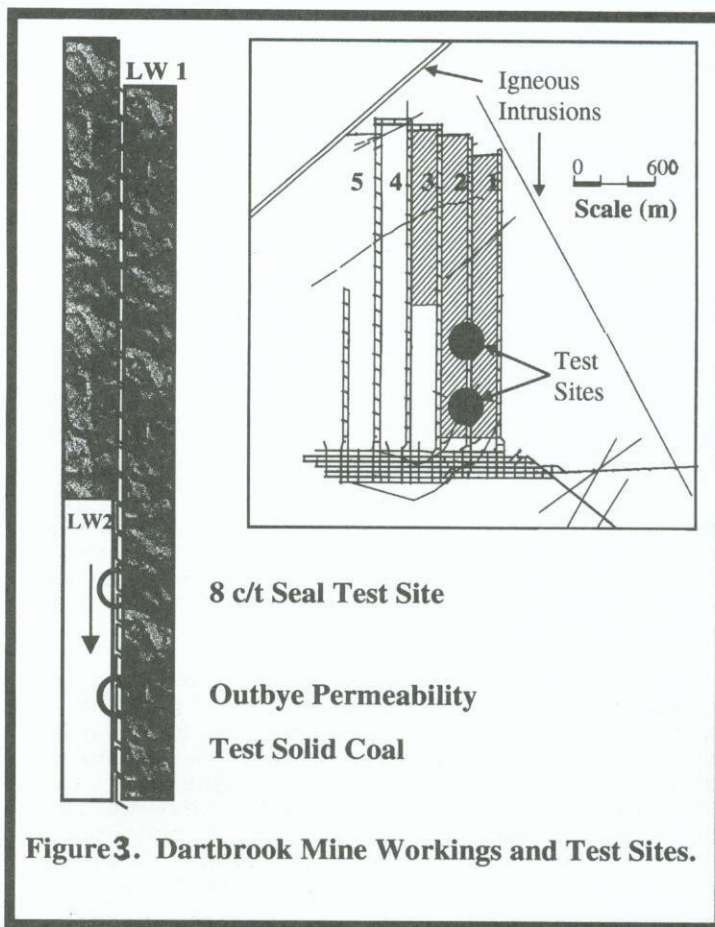
During extraction of Longwall 2 the face was approximately 300m away, the site was cleaned up in anticipation of placing a 5m wide sprayed concrete seal on the Longwall 2 side of the Tekblend mini-seal.

Details of the test site and the location of the various instruments are shown in Figure 4. Instruments were located in the coal rib midway along the concrete seal.

The concrete was delivered to the surface by commercial contractors and dropped to the underground workings by a steel lined bore hole. The cement is then transported from the underground borehole site and sprayed into place to create the seal. The concrete material was sprayed in layers with reinforcing mesh placed at 0.5-1.0m intervals. A timber crib was installed approximately 2m from the face of the concrete seal soon after the seal was formed. Cans were installed at the end of the cut-through.

As Longwall 2 approached the site, instrumentation cables were extended inside of protective pipes. Suspended in the top corner of the roadway the pipes were

extended to a location approximately 100m outbye of the test site. Unfortunately, the instrumentation was lost in stages as the various extension cables were crushed as Longwall 2 passed the site.



TESTING PROCEDURE & INSTRUMENTATION

Permeability tests were undertaken in the coal ribs adjacent to the seal at various stages, as the longwall face approached and then passed the monitoring site. At 8 c/t two drill holes were used to measure near surface (1-2m, outbye side) and surface (<1m, inbye side) permeability. By way of comparison a second series of tests were conducted at depth between 4 and 5 cut-through in intact coal using the same techniques.

Pressure transducers were located in the two holes at 8c/t with packed off sections of the hole to monitor pressure changes. Two air tubes were terminated in the same cavity allowing air to be circulated in one tube and out the other confirming the cavity was not blocked.

During pressurisation, the pressure in the cavity was measured directly using the pressure transducer. The volume of air flowing into the formation was monitored externally at the pressure supply.

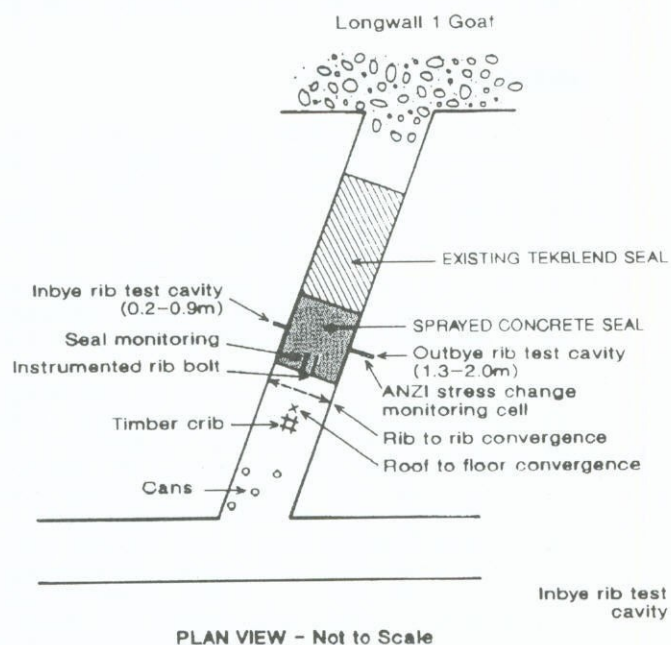
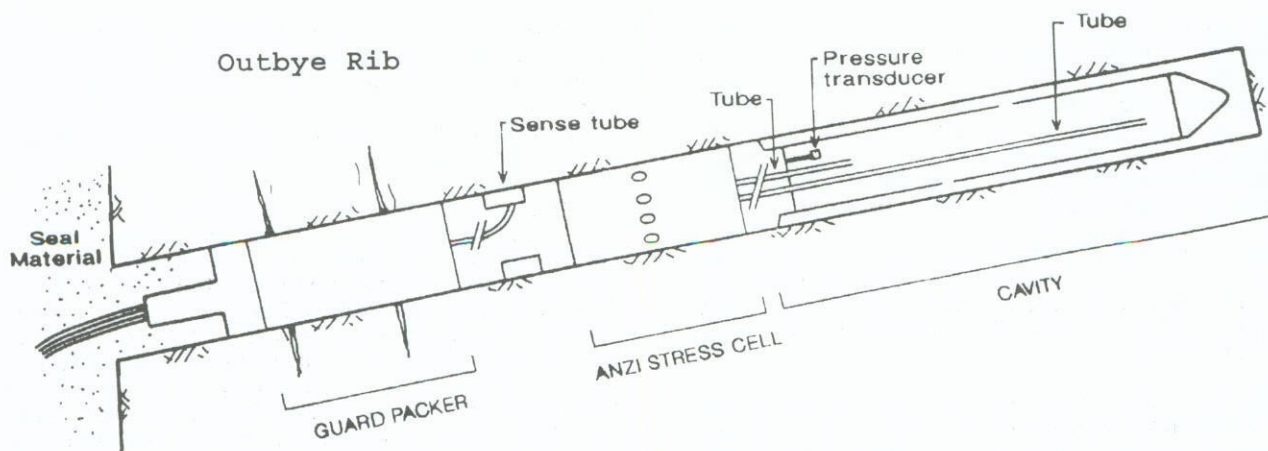
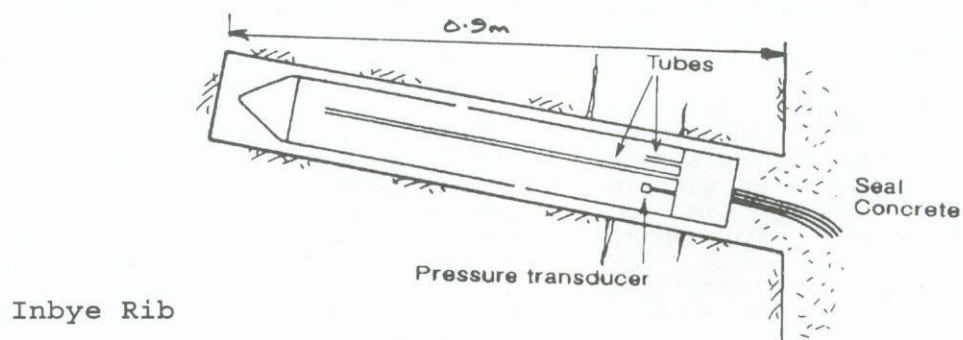
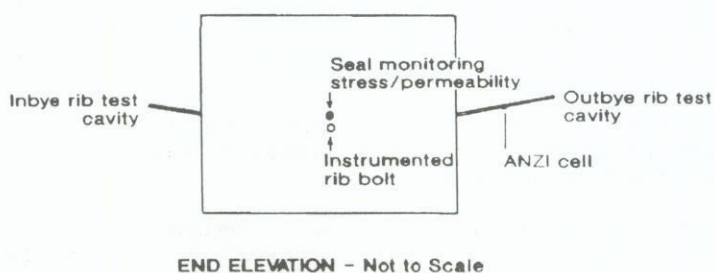


Figure 4

Instrumentation Layout and Monitoring Site



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Inflatable packers were cemented into the o/b rib hole. The inner packer was an ANZI stress cell (Mills 1997) capable of measuring stress changes. The outer packer acted as a guard cell to confirm that there was no leakage past the inner packer. A pressure sensing tube from the interval between the two packers confirmed there was no leakage. Two air tubes and a pressure transducer located in the cavity at the end of the hole allowed, first, circulation to be confirmed and, second, the cavity to be artificially pressurised to measure leakage into the surrounding strata.

Two types of permeability tests were undertaken at each site, constant flow tests and falling head tests.

The constant flow test provides a good indication of permeability in moderate to high permeability conditions, where the volume of air lost into the formation is sufficiently large to be measured in a reasonable timeframe. The constant flow test involves maintaining a steady flow at a constant pressure. Both the pressure and the flow rate are measured.

The falling head test gives a better result in moderate to low permeability conditions. It involves establishing an initial steady flow at constant pressure. The test starts by isolating the cavity and allowing the pressure to bleed away as flow occurs into the formation. The flow rate can be estimated from the pressure change.

Stress changes were measured using an ANZI stress cell. This instrument comprises of eighteen electrical resistance strain gauges located on the outer surface of the membrane. The strain gauges are cemented to the borehole wall using epoxy cement. After the cement has cured, the ANZI cell can be incrementally pressurised and depressurised to confirm the correct operation of the instrument and estimate the elastic modulus of the rock.

Movements of the rib, roof and seal face were monitored as the goaf of Longwall 2 loaded up the site. Rib and roof convergence was measured by a remote reading convergence measurement system. Outward movement of the face of the seal was measured using an instrumented rib bolt installed in the seal during construction.

An instrumented rib bolt was located in the centre of the seal. This instrument is capable of sensing very small deformations associated with outward movement of the face of the seal.

Monitoring of instruments was undertaken as the longwall approached the test site. Remote monitoring continued after the seal was passed by the longwall. Measurements

continued up until the different instrument lines were severed or crushed by the goaf.

PERMEABILITY TESTING

On the inbye side of the seal, the rib coal between 0.2 and 0.9m deep was tested. This test interval included a high proportion of open fractures up to 1cm wide accounting for an estimated 50-100mm of rib compliance. On the outbye rib, the interval between 1.3m and 2.0m was tested. Coal in this interval was essentially intact and beyond the immediate yield zone.

The results of the various permeability tests are compared in their raw form as flow rate versus cavity pressure. The permeability can be estimated directly from this data, but the values are sensitive to the boundary conditions assumed. The results are compared on the basis of flow rate and pressure and are summarised in Table 1 and Figure 5. The results of the falling head tests are shown in Figure 6. All flow rates are at atmospheric pressure.

When the longwall was still 70m inbye, two constant flow tests were conducted at different pressures. The results of the two tests fall on a straight line that passes through the origin (Figure 5). This recognises that proportionality exists between flow rate and pressure. Subsequent results are compared using a line that passes through the origin assuming such a proportionality relationship.

The initial permeability of the near surface interval tested on the inbye rib was approximately 200 times more permeable than the permeability of intact coal, tested 8m into the coal pillar at 4-5 C/T. As the longwall approached the seal site and then moved 20m past, the pressure required to maintain a constant flow rate increased 400%, representing a fourfold decrease in permeability.

The falling head test results indicate that when the longwall was 20m past the site, the permeability of the near surface interval tested on the inbye rib was approximately twenty times the permeability of intact coal.

The constant flow rate tests conducted in the outbye rib did not show an appreciable change in permeability until the longwall was within 40m of the site. The initial permeability of the interval tested in the outbye rib (1.3-2.0m in) was approximately 5-6 times more permeable than the intact coal. During the period when the longwall face had 40m to go and when it was 20m past the site, the permeability of the 1.3m-2.0m deep interval effectively halved.

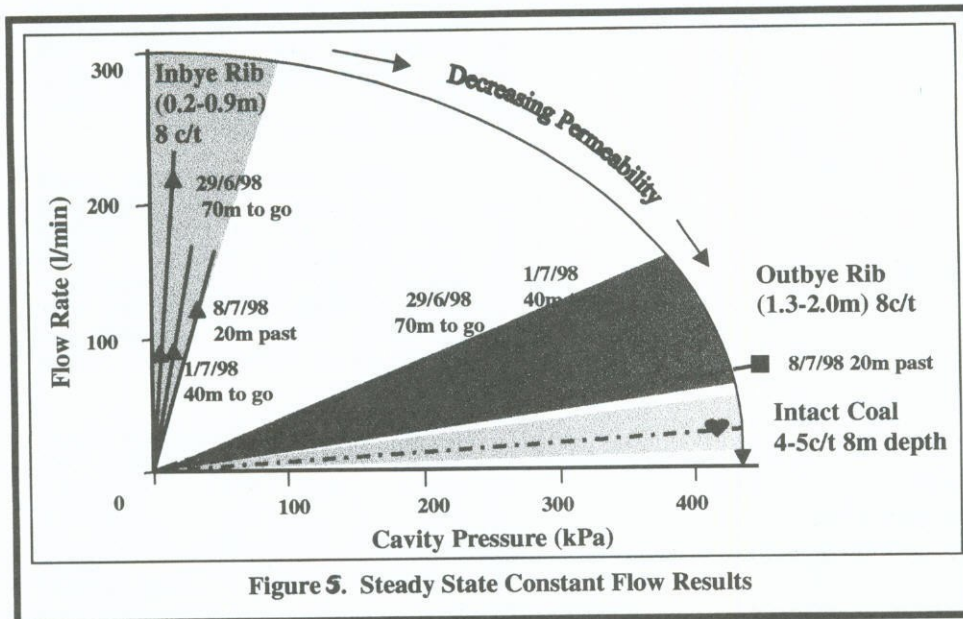


Figure 5. Steady State Constant Flow Results

The falling head tests at the end of the intact coal hole show essentially similar behaviour as the initial pressure is increased in stages from approximately 150kPa (Test 1), 300kPa (Test 2) and 475kPa (Test 4). There appears to be some residual pressure retained in the formation from the previous tests. A repeat of the first test with an initial pressure of approximately 150kPa (Test 5) shows an approximately 20kPa offset in the flow rate v pressure curves at low pressures as shown in Figure 7. The curves are otherwise similar.

Figure 8 shows the flow rates measured for each of the cavity lengths at an instantaneous cavity pressure of 200kPa for the first series of tests near the end of the borehole and the last two tests (Tests 7 & 8) conducted further outbye. The results show a more or less linear dependence of flow rate on cavity length as would be expected.

The equivalent flow rates measured during the various permeability tests are summarised in Table 2. In each case, the cavity is 0.7m long and 58mm in diameter and 5 l/min is the equivalent volume of air at atmospheric pressure.

The relationship between instantaneous pressure and flow rate is non-linear so the selection of flow rate will influence the ratios obtained. A flow rate of 5 l/min is chosen because it is representative of a low flow that is still within the range of data points available.

STRESS TESTING

Figure 9 shows the changes in circumferential and axial strains and

vertical and horizontal stresses measured on the ANZI cell. These strain changes are only meaningful when analysed in combination with each other; they indicate the gradual nature of the stress changes observed and the fundamental change in loading behaviour that occurs when the longwall is approximately 20m past the site.

The form of the results, the gradual nature of the changes observed, and the internal correlation of redundant gauges all indicate that a high degree of confidence can be placed in the stress changes measured. The calculated

stress changes in the outbye rib of the seal are summarised in Table 3.

The measurements indicate that the vertical stress increased gradually as the longwall face approached reaching 2.4MPa, when the face was 20m past. No significant change in the horizontal stress was measured during this period.

When the longwall face moved beyond 20m past the site, the loading behaviour changed. There was a very large increase in horizontal stress toward the seal accompanied by a small decrease in vertical stress.

The stress changes measured during the initial stage of loading show that the near vertical stress component is inclined at approximately 20° to vertical with a dip direction toward the goaf. This direction is consistent with the known stress environment.

CONVERGENCE MONITORING

Some 100-200mm of rib convergence and up to 100mm of roof convergence is estimated to have occurred prior to the instruments being installed. Most of this initial convergence would also have occurred prior to the sprayed concrete seal being placed.

The roof to floor convergence increased steadily over the period of monitoring reaching 250mm, by the time the longwall face was 60m past the test site. The roof to floor convergence was measured some 1.5-2.0m from the face of the concrete seal. It is expected that the actual convergence on the seal would have been less than that measured because of the physical resistance provided by the seal itself.

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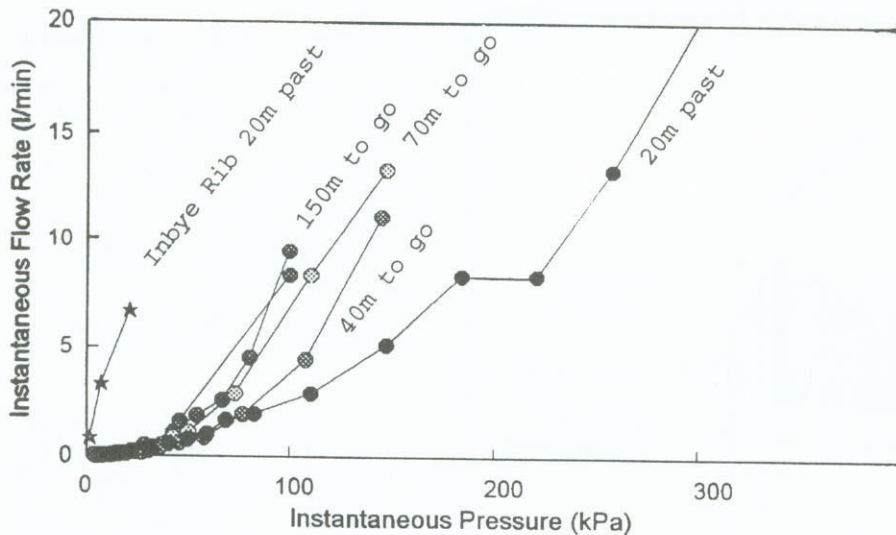


Figure 6. Falling Head Permeability Tests - 8c/t.

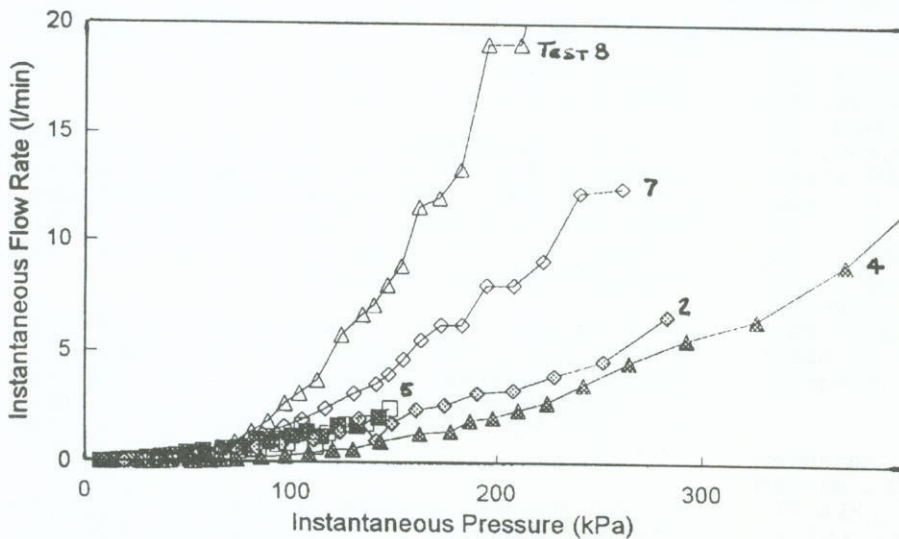


Figure 7. Falling Head Permeability - Intact Coal.

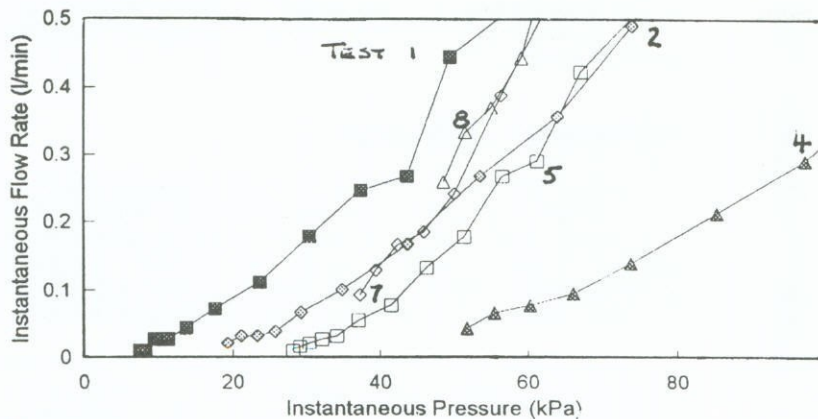


Figure 8. Falling Head Permeability - Low Flow Rates.

STRAIN GAUGE BOLT

The rib bolt located in the seal showed a small increase in load during the period

that the longwall face approached and passed the site. This instrument is very sensitive to small deformations in the seal.

A 20MPa increase in load was registered, representing an increase in the bolt length of approximately 0.1mm. This suggests that the seal was not heavily loaded at the time the instrument could no longer be read shortly after the face passed.

Core from the concrete seal material was recovered and the strength (17-34MPa), elastic modulus (11-20GPa), porosity (20%) and moisture (8%) of selected samples were determined in laboratory tests.

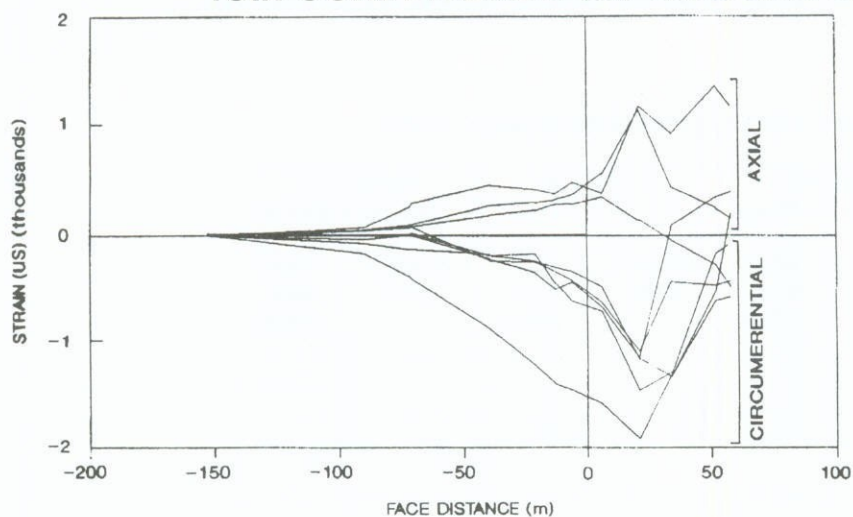
INTERPRETATION

The monitoring data suggests that the surrounding coal has converged on the seal more or less evenly as the longwall has approached and passed the seal. The permeability of the rib coal around the seal has decreased as the cracks in the ribs have closed as a result of this convergence/confinement. Once the cracks have closed completely, the horizontal stress has built up significantly.

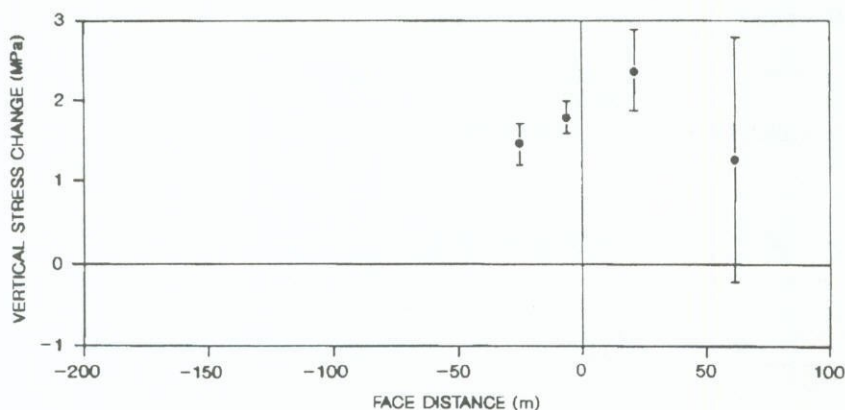
The stress monitoring results show that the stresses in the rib changed in two fairly distinct stages. The first stage coincided with the longwall face approaching and moving up to 20m or so past the site. During this stage the vertical stress increases gradually in a fashion consistent with the vertical load that would be expected about the corner of a longwall panel. There was a relatively small change in horizontal stress during this period.

Initially, the first metre or so of rib coal was fractured and approximately 200 times more permeable than intact coal, even though the rib had been treated with polyurethane.

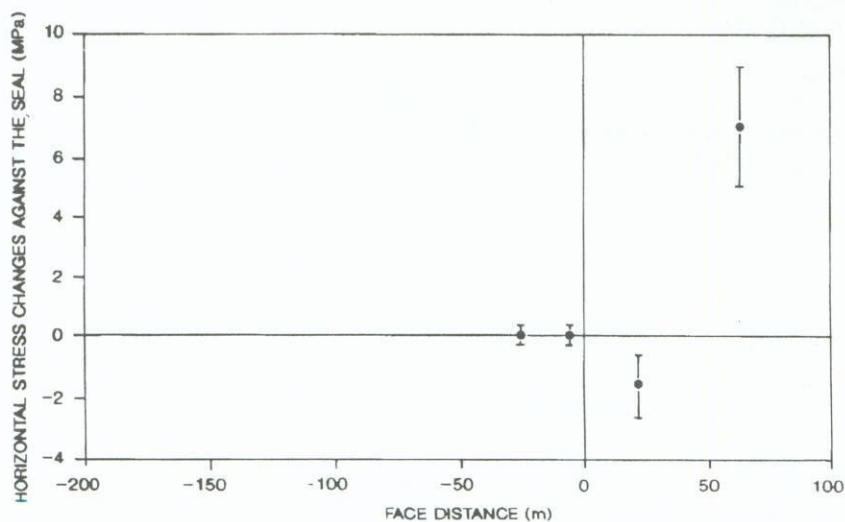
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a) Measured instrument strain.



b) Vertical stress change.



c) Horizontal stress change.

Figure 9 **DARTBROOK MINE**
Stress change measured on the outbye rib of the seal.

During the first stage, the cracks in the rib were still open but became narrower as the pillar converged onto the seal as evidenced by the reduced flows and the stress behaviour.

In the first stage, permeability in the near rib reduced by approximately 4-5 times and in the deeper rib 2 times. The reduction in permeability occurred even though the horizontal confining stresses in the rib remained very low and essentially unchanged, the still open cracks preventing the development of horizontal confining pressure. These permeability changes were measured directly.

The second stage coincided with the longwall face moving beyond 20m past the site. During this stage the horizontal stress increased rapidly. (The vertical stress during this period appears to have decreased slightly, but the error bands are too large to be confident of this.)

The second stage was characterised by the cracks closing completely, allowing very high confining pressures to be developed as the pillar continued to converge. When the cracks had closed completely, very high confining pressures (3-11MPa) were measured in the rib. At this time the permeability could not be measured directly. Relationships based on confining pressure suggest that the permeability of the coal rib would be reduced by one or more orders of magnitude, as a result of the confining pressure developed. This permeability would then be of the same order as that measured in intact coal.

During this period the convergence instruments indicate that the roof continued to converge at a more or less even rate. The rib convergence also appears to have occurred steadily.

The combined width of the open cracks is estimated from observation to have been of the order of 50-100mm in each rib. The total convergence required to close cracks on both ribs would be in the range 100-200mm. This level of convergence is consistent with the rib convergence that was measured (and extrapolated) to the point when the longwall face was 20m or so past the site.

The absence of any change in horizontal stress during the first period while there was a steady convergence of the pillar onto the seal is consistent with closure of the cracks in the first metre or so of rib. While the cracks are open,



TABLE 1: SUMMARY OF CONSTANT FLOW PERMEABILITY TESTS

LONGWALL FACE POSITION RELATIVE TO EACH SITE (m)	INBYE RIB (0.2-0.9M INTERVAL)		OUTBYE RIB (1.3-2.0M INTERVAL)		SOLID COAL (7.3-8.0M INTERVAL)	
	Pressure (kPa)	Flow Rate* (l/min)	Pressure (kPa)	Flow Rate* (l/min)	Pressure (kPa)	Flow Rate* (l/min)
70m to go	6	80	237	80		
	20	220				
40m to go	18	92	315	108		
20m past	35	120	570	99		
200m to go (outbye site)					420	24

TABLE 2: SUMMARY OF FLOW MEASUREMENT RESULTS

LOADING CONDITION	INTACT COAL 8M	IMMEDIATE RIB 0.2-0.9M	DEEPER RIB 1.3-2.0M
Equivalent Pressure (kPa) Required to Maintain Constant Flow Rate of 5 l/min*			
LW1 Side Abutment	87	0.4	14
LW2 40m to go	-	1	28
LW2 20m past	-	1.4	29
Equivalent Instantaneous Pressure (kPa) at 5 l/min in Falling Head Test**			
LW1 Side Abutment	280	-	82
LW2 70m to go	-	-	95
LW2 40m to go	-	-	110
LW2 20m past	-	14	150

TABLE 3: STRESS CHANGES MEASURED IN THE OUTBYE RIB
MIDWAY ALONG THE SEAL

Longwall Face Position (m)	Vertical Stress Change (MPa ±95% confidence)	Horizontal Confinement (MPa ±95% confidence)	Correlation Indicating Confidence
25m to go	1.5 ±0.5	0 ±0.6	0.982 on 5 dof
6m to go	1.8 ±0.4	0 ±0.5	0.991 on 5 dof
20m past	2.4 ±1	-1.6 ± 2	0.98 on 4 dof
60m past	1.3 ±3	7 ±4	0.92 on 3 dof

TABLE 4: PERMEABILITY VALUES MEASURED

Rib Sampling Depth (m)	Monitoring Points at 8 c/t		
	LW face 70m inbye of 8c/t	LW face 40m inbye of 8 c/t	LW face 20m outbye of 8c/t
0-0.9m	990(mD)	460(mD)	309(mD)
1.0-2.0m	35(mD)	31(mD)	16(mD)
Bore drilled in Solid Coal mid Pillar to determine in-situ permeability.			
8.0m	2.8(mD)		

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there can be no substantial horizontal stress transfer across them.

The onset of the change in loading behaviour and increase in horizontal stress after the longwall passed the site by 20m is consistent with the rib cracks having closed completely. The horizontal stress measured against the seal reached 7 ± 4 MPa before readings ceased because the cable was crushed. While the absolute magnitude of the horizontal stress change cannot be determined with a high degree of certainty, the result shows unequivocally that a significant increase in horizontal stress has occurred, as expected.

CONCLUSIONS

The monitoring program indicates that the sprayed concrete seal reduced the permeability of fractured coal in the rib around the seal. Based on the results obtained, it is reasonable to anticipate that a Consolidated Barrier would develop permeability of a similar order of magnitude to a solid coal segregation barrier.

The permeability testing shows that there is a zone of fractured coal and coal with higher permeability adjacent to the rib before the seal is installed. Most of the open fracturing is within the first metre or so. The coal in this zone is approximately 200 times more permeable than intact coal. In the next metre or so, the coal is less fractured but the permeability is still 6 times more permeable than intact coal.

The permeability appears to have decreased in two fairly distinct stages. In the first stage, the reduction in permeability appears to be associated with closure of the pre-existing rib cracks caused by convergence of the chain pillar onto the seal. The permeability changes measured during this stage are a gradual, but relatively modest, 2-3 times reduction in permeability.

In the second stage, the cracks have closed and further convergence of the coal pillar onto the seal causes a significant build up in horizontal stress in the rib. The magnitude of the stress increase measured suggests a reduction in permeability of about 2 orders of magnitude would be expected. No direct measurements of permeability change in this second stage were made due to the premature crushing of the tubes.

The results of the monitoring are sufficiently encouraging that further trialing of an improved seal design is

expected. With some small changes in the instrumentation system to provide greater security for long term monitoring so that the reduction in permeability anticipated could be directly measured.

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Ground Control in South African Coal Mines - A U.S. Perspective

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ABSTRACT

After the United States and Australia, South Africa has the largest, modern underground coal mining industry in the world. Historically, South Africa has been the cradle of many innovations in ground control, particularly in the area of pillar design. Today, South African mines are grappling with many of the same issues faced by their U.S. counterparts, including safety during pillar retreat operations, selecting the proper roof support, and maintaining long-term pillar stability.

The author recently visited seven underground mines in the Mpumalanga, Free State, and KwaZulu-Natal provinces. Mining methods included longwall, room-and-pillar with continuous miners, and drill-and-blast. The mines represented a cross-section of geology and ground support practices. At each mine, evaluations were made of the Coal Mine Roof Rating (CMRR), roof support practices, pillar design methods, and general ground control experience.

The coal seams observed were generally thick (2-5 m), the depths of cover moderate (less than 200 m), and the roof rocks relatively strong (CMRR greater than 45). Unfortunately, the South African roof fall fatality rate is approximately three times greater than in U.S., perhaps in part because roof bolts seem to be more widely spaced in most South African mines. Roof bolting equipment was generally outdated, and the quality of bolt installation was a major concern. Pillar design, on the other hand, is quite advanced, and pillar failures are rare. Several novel partial pillaring techniques are employed, including "checkerboard" mining in which every other pillar is removed, 3-cut systems, and high-extraction mining where engineered final stumps are left by design. The paper also offers observations on ground control management techniques (including recently introduced Codes of Practice), the role of the mine inspectorate, and the status of the research community.

ACKNOWLEDGMENTS

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INTRODUCTION

South Africa is the third largest coal exporting nation, after the U.S. and Australia. In 1996, nearly 106 million clean tonnes were produced from underground mines that employed approximately 28,000 workers (1, 2).

The mines are located primarily in the northeastern portion of the country (figure 1). The oldest mining areas are in KwaZulu-Natal, but today most production comes from the Witbank-Highveld area in Mpumalanga Province about 100 km east of Johannesburg. The three large mining "houses" (companies) of Amcoal, Ingwe, and Sasol together account for 80% of all coal production. A total of just 22 large mines account for nearly 90% of all production (3). Many of these are captive to power plants or the giant SASOL chemical works.

While the number of underground miners is approximately half of what it was just 10 years ago, underground production has nearly doubled, and productivity has increased by a factor of 3.6. Labor costs per tonne have been essentially constant, however, reflecting sharply higher wages and greater investments in human capital (3).

In 1996, 23 South African miners lost their lives in ground falls, about 50% of all underground fatalities (2). In recent years, the groundfall fatality rate has averaged about 0.6 per 1,000

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underground employees, a rate about 3 times greater than in the U.S (figure 2). Canbulat and Jack (2) present data showing that 35% of these fatalities occurred in the older KwaZulu-Natal coalfields which account for less than 10% of underground production (figure 3). The mining method was drill-and-blast in 67% of all South African ground control fatalities.

The most modern mining methods are found in Mpumalanga province, which accounted for 80% of underground production but just 40% of the groundfall fatalities. Kritzing's (4) detailed analysis of Mpumalanga accident statistics found that between 1997 and 1998, 50% of fatalities occurred under supported roof, 77% occurred in the face area, and 77% occurred on continuous miner sections. Injuries to roof support personnel (roof bolt machine operators, rock drill crews, and pinch bar users) accounted for 32% of serious groundfall injuries. Another 25% of injuries were to continuous miner operators, and 14% were to operators of coal cutting or loading machines on conventional sections.

Nationwide, approximately 40% of the fatalities occur during the process of pillar extraction, though only about 25% of underground production is from "stooping" sections. Also, the thickness of the rock was less than 0.5 m in 63% of the fatal

accidents, and less than 1.0m in 81% of the accidents. Sandstone, shale, and coal roofs were each cited in approximately equal frequency in the fatalities where reports were available (2).

GEOLOGY

South Africa is blessed with excellent mining conditions. Coal seams are almost always at least 2 m thick, and often 4-5 m thick. The depth of cover is less than 300 m, and the coal ribs are usually strong and relatively uncleated. The floor is usually competent and the seams are level. The roof is typically comprised of quite competent sandstone or siltstone as well. I found the Coal Mine Roof Rating (5) greater than 65 in 5 mines (3 in the Highveld and 2 in KwaZulu-Natal), and greater than 50 in 3 other Highveld observations. The weakest roof observed was in the 3 Seam in the Sasolburg area, in the Free State. Here the CMRR was still about 45 (in the U.S., weak roof is often CMRR=35 to 40). Where the roof is a weaker shale, the mines often leave 0.5 m of the strong coal in the top. Other mines leave 10 cm of top coal to reduce frictional ignitions. There seemed to be a reasonable correlation between the CMRR and the entry width (figure 4).

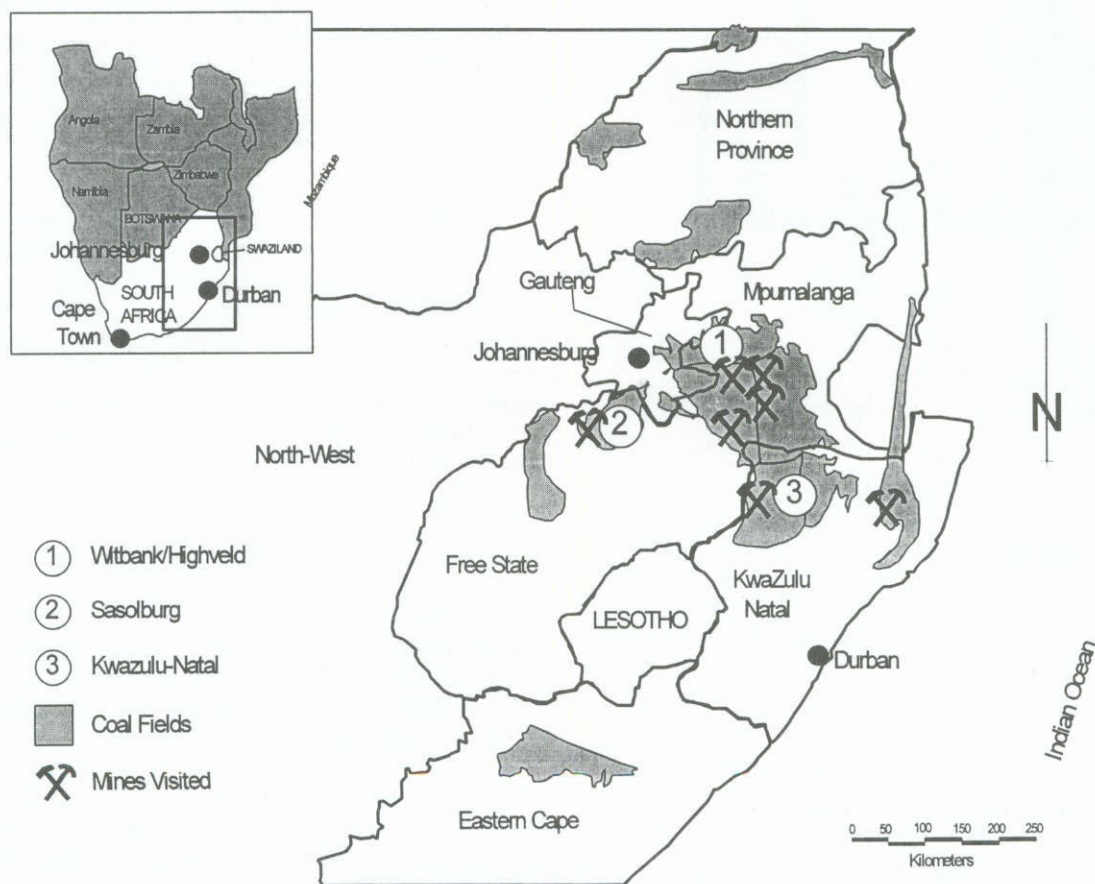


Figure 1. Location of the South African coalfields and the mines visited for this report.

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Figure 2. Number of employees and groundfall fatality rate for the South African coal mining industry.

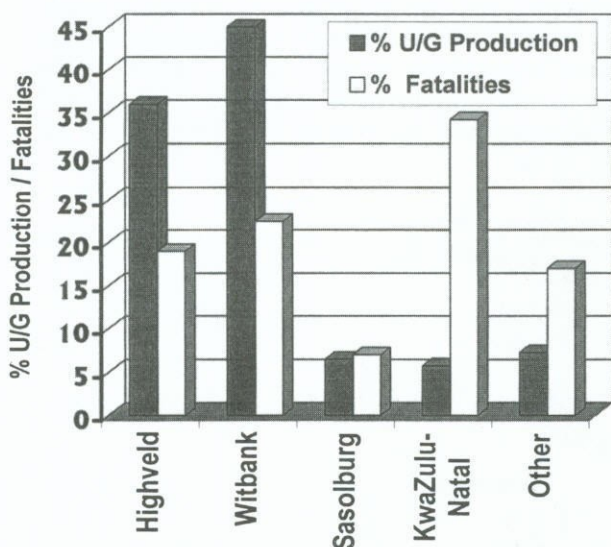


Figure 3. Underground production and groundfall fatalities compared by location.

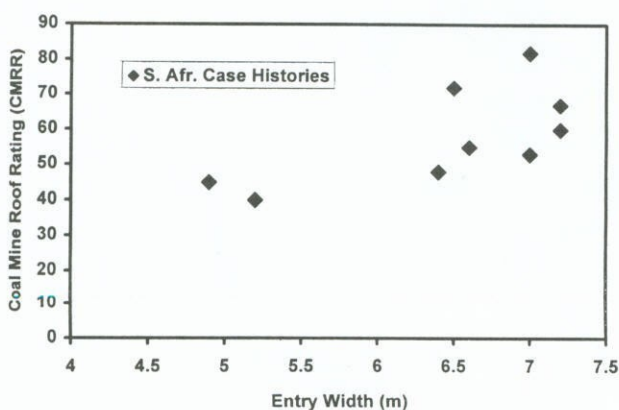


Figure 4. Entry widths and Coal Mine Roof Ratings (CMRR) for mines that were visited.

Horizontal stress was not considered an issue in South Africa until very recently. Clear evidence of horizontal stress could be observed in some of the weaker roof types. A study at the Brandspruit Colliery in the Highveld area also indicated that most of their fatal accidents have occurred beneath stream valleys (6). According to the world stress map project, and confirmed by stress measurements in deep hard rock mines, the major principle stress is oriented NNW (7). Several of the visited mines reported that EW entries were the most troublesome, which would result from an approximately NS regional principal stress.

Igneous dykes appear to be the most significant large-scale geologic features affecting ground stability. The dykes often cut through the coal seam, affecting mine layouts. At Syferfontein Colliery, in the Highveld, the dykes devolatilize the coal, weakening it and ruining it as top coal. The shale roof is much more prone to failure without top coal, so the entry widths are reduced from 6.6 to 4.9 m near dykes, and extra support is installed. Faulting is also a significant problem in some mines.

Dolerite sills are another igneous feature that affects ground control in many South African mines. Dolerite is extremely strong, and the sills may be 50 m thick. The loading experienced by pillars in a retreating section can be much higher when the sill is intact and bridges the panel. Therefore, panels are often sized to exceed the "critical width" of the dolerite, so that it will cave and transfer the load to the gob (8, 9). Where the required width is excessive, two retreat panels may be worked simultaneously side-by-side.

In general, however, ground conditions are relatively benign. In an extensive study reported by Canbulat and Jack (2), 8 m long sonic extensometers were installed at 29 sites in 5 collieries. Though the mines represented a variety of geologic environments and mining methods, the maximum total roof movement measured at any intersection site was just 12 mm. In roadways, the maximum deflection was only 3 mm! Apparently, the combination of strong roof, light cover, and moderate horizontal stress mean that massive roof falls are relatively rare.

MINING METHODS

More than 90% of the underground coal is mined using room-and-pillar techniques. In 1992, more coal was still being mined by conventional drill-and-blast than by continuous miners (10). All the mines I visited employed continuous miners, except Zululand Anthracite Colliery.

Longwall mining has never been popular in South Africa, due to the faults and dykes, subsidence concerns, and the excellent conditions for room-and-pillar mining (10). Longwalls currently account for less than 5% of underground production.

New Denmark Colliery is one of the two large South African longwall mines (Ingwe's Matla Colliery is the other one). Attached to a large power station, it is one of the deepest

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operating collieries (200 m) and is in a thin seam about 2 m). As many as four longwall faces may be operating at any one time.

Pillar design at New Denmark has been driven by the need for uniform surface subsidence. The first longwalls used a two-entry system, with a 25 m pillar that was reduced to 9 m by the longwall. The blind cut-out was less than satisfactory, and was replaced by a 3-entry system where one pillar was extracted. The mine has now returned to 2-entry gates with 9-12 m "crush pillars." New Denmark also has considerable experience extracting pillars and mid-gates with longwalls (figure 5), with some mixed success (11).

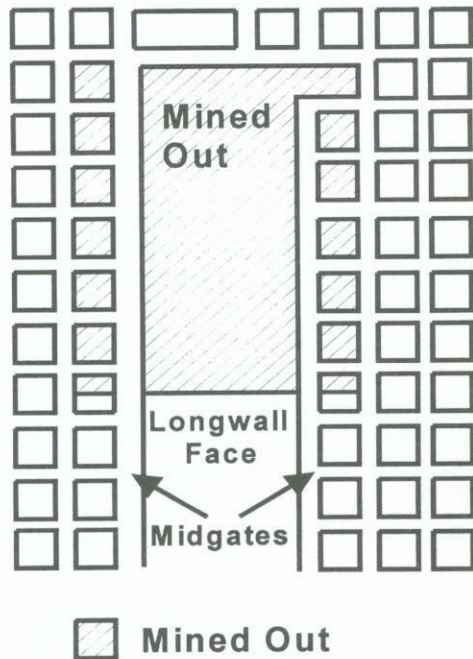


Figure 5. Midgates in a longwall panel at New Denmark Colliery.

New Denmark's roof is a competent sandstone, with a CMRR estimated at 72. No bolts are installed during development. When the longwall face is 60-70 m away, contractors install rows of 3-4 bolts every 1.5-2 m in the 6.5 m wide gate entries. Cable bolts are also installed in the intersections, but no standing support is used in the tailgate.

Another unique mining system was observed at Syferfontein Colliery. There an ABM30 miner-bolter is used in conjunction with a continuous haulage system. Two bolts are installed for each meter of advance in the 4.8 m high entry. The entries are mined on 24 m centers, with the crosscuts angled at 60°. A strong, flat floor is essential to the success of the system, which averages 70 m of advance in a 10-hour shift. Smaller ABM20s are used in the weak roof areas near dykes.

ROOF SUPPORT

One striking feature of South African coal mines is the relative lack of roof bolts. The Primary Support Rating (PSUP) averaged 0.08 for mines with a CMRR between 50 and 70, which is less than half the support that mines in similar conditions in the U.S. would use (12). The biggest difference is the bolt density. Where the spacing between bolts in the U.S. is limited to 1.5 m, and 4 bolts per row is standard, in South Africa a typical pattern is 3 bolts every 2-3 m. In fact, 9% of South Africa's collieries use no bolts at all (2). Figure 6 shows that PSUP observed in South African mines essentially falls on the lower bound of the U.S. data. Interestingly, the figure also shows that the regression equation derived from Australian data (13) basically defines the upper bound of the U.S. data.

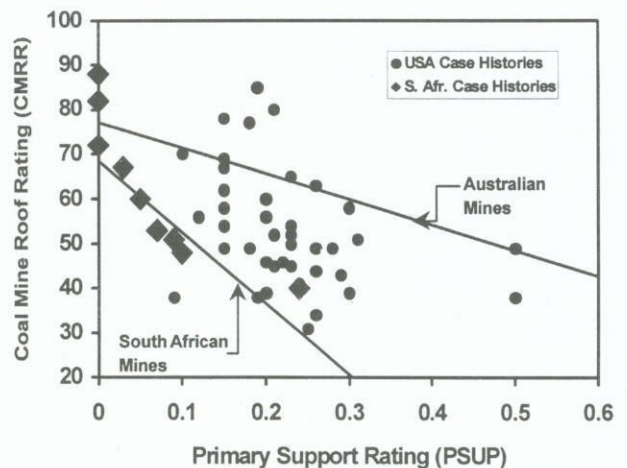


Figure 6. Primary support ratings (PSUP) for South African, U.S. and Australian coal mines.

The South African "Guidelines for Compiling a Mandatory Code of Practice to Combat Rockfall Accidents in Collieries" (14) classifies roof into three types:

- ◆ *Strong roof* requiring no systematic support;
- ◆ *Suspension-type roof* where a thin layer of weak material is overlain by a stronger layer, and;
- ◆ *Beam-creation type roof* consisting of a thick layer of weak material.

The Guidelines also mention discontinuities, road width, burnt coal, ribside support, and water/gas pressure, and "the potential for failure between support elements."

While large roof falls do seem to be unusual in South Africa, most of the injuries and fatalities are being caused by relatively small pieces of rock. The wide spacing between bolts may be an important contributing factor. Some South African rock mechanics specialists have argued that installing shorter bolts on a tighter pattern could be effective in preventing these small rock falls. The mines seem reluctant to adopt this approach, and roof bolting equipment may be part of the reason. The roof bolters I saw were all single-boom machines, some with

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scissors-type drill masts, mounted on small rubber tires. Automated temporary roof support (ATRS) was either inadequate or non-existent, so temporary jacks are required. It is considered a good shift when two men install 50 bolts with this equipment.

The quality of roof bolt installation is also a source of controversy. Studies conducted at the Sasol mines found that between 30 and 70% of the installations were inadequate. Sasol has recently implemented a program where roof bolts, drill rods, and resin cartridges are all color-coded to ensure that they match. Timing lights have been installed on the roof bolters to help the operators time the resin spin cycle. After the improvements were implemented, a follow-up study found that 98% of the installations were good, and the Sasol approach is now spreading through the industry.

There is no equivalent of the ASTM standards for roof support materials that are used in the U.S., so maintaining quality of roof bolt consumables is another problem. Ingwe mines require tight quality specifications in their contracts with roof support manufacturers. Samples from every batch of supports are tested in Ingwe's own rock mechanics lab, and batches that fail are rejected. At Sigma, the Code of Practice requires that roof support materials shall be supplied by ISO 9002 rated companies, with independent quality checks carried out on all support elements (15).

PILLAR DESIGN

Pillar design has been a defining issue in South Africa ever since the 1960 Coalbrook disaster in which over 400 miners were killed in a massive pillar collapse. The Salamon-Munro (16) formula, which was statistically derived from back-analysis of 125 case histories, is still the basis for most pillar design in the country. The Guidelines for Codes of Practice (14) state that "pillars shall be designed to a suitable safety factor using the Salamon-Munro (16) formula." The Guidelines allow that "any other formula with proven applicability" may be used, but "deviations from the Salamon-Munro (16) formula must be approved by the Chief Inspector."

A nearly universal "deviation" appears to be the Salamon (17) "squat" formula which was developed for pillars with width-to-height (w/h) ratios greater than 5. Typically, main entries are designed to SF=1.6 with the Salamon formulae. Where pillar extraction is anticipated, SF of 1.8-2.0. are used.

Some local adjustments have been necessary. In the Vaal Basin (which includes the Sasolburg Coalfield), experience has shown that the in situ coal strength is 4.5 MPa, not the 7.2 MPa used elsewhere in South Africa (18). At Zululand Anthracite Colliery, where the coal is highly sheared, a pillar squeeze occurred despite a SF of 1.7 on development. The SF for all subsequent developments was increased to 2.0.

Madden and Canbulat (19) evaluated the case history data and found that 34 pillar collapses have occurred when the SF exceeded 1.6. They determined that nearly all of these failures could be explained by four factors:

- ◆ Slender pillars at shallow depth;
- ◆ Weak coal in the Vaal Basin;
- ◆ Soft floor, and;
- ◆ Slips and joints in some KwaZulu-Natal Coals.

They concluded that to prevent failures at shallow depth, w/h ratios should be greater than 2.0, extraction ratios should not exceed 75%, and no pillar should be less than 5 m wide. Most recently, van der Merwe (20) re-analyzed the case histories and derived a new pillar strength formula that reduces the strength of slender pillars but predicts greater strength for squat pillars.

PILLAR RECOVERY

Pillar recovery operations account for a disproportionate share of the groundfall fatalities in South Africa. The mines I visited had adopted a variety of techniques to improve upon this record. A common theme seems to be the use of "high extraction" techniques that leave small remnants of pre-determined size in planned positions (21). The goal is that the "snooks" (remnants) are large enough to support the roof above the intersection as the pillar is being extracted, yet small enough to fail once they are in the gob (15). The "high extraction" concept has largely replaced the older philosophy that emphasized complete pillar recovery (22, 8).

Sasol's Middlebult Colliery is reportedly typical of the large mines in Mpumalanga. The roof is quite competent, with a CMRR of 67, and the seam is 3-5 m high. In areas where pillars will be recovered ("stooped"), they are initially sized with an SF of 2.0. A typical pillar is 20 by 20 m. The pillars are then recovered by split-and fender. Only a single row of breaker posts, and no bolts, are employed. However, there is no attempt to recover a pushout. Instead, the final stump, measuring approximately 4 by 6 m, is left for roof support.

Sigma Colliery, in the Free State, is another Sasol operation. The roof is significantly weaker, with the CMRR ranging from 40-50. Under the CMRR=50 roof, a rib pillar technique (similar to the U.S. outside lift, or "Old Ben" technique (8)) was quite successful (23). In the weaker top, however, roof falls frequently trapped the continuous miner. A partial recovery technique, called "pillar robbing" but similar to our Virginia three-cut, has proved much more successful. One of the advantages of pillar robbing is that it gives the crew more flexibility to respond to changing geologic conditions.

The roof at Zululand Anthracite Colliery (ZAC) is such a competent sandstone that no roof bolts are installed. During full pillar recovery, the roof has a tendency to hang up for tens of meters and then cave suddenly. These large falls could "feather

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edge" into the working areas with occasionally fatal consequences.

ZAC has now adopted a partial pillaring technique called "checkerboarding." Pillars originally measure approximately 12 m square in the 1.3 m seam. Half the pillars are completely recovered, while every other pillar is reduced to a stump measuring 8 by 8 m (figure 7). The goal is to provide roof support, while at the same time preventing a massive collapse that might occur if smaller stumps (with width-to-height ratios less than three) were left. Worker confidence has so improved with this system that overall coal recovery is said to have remained unchanged. One problem I observed, however, is a tendency to reduce the "checker" pillars too much.

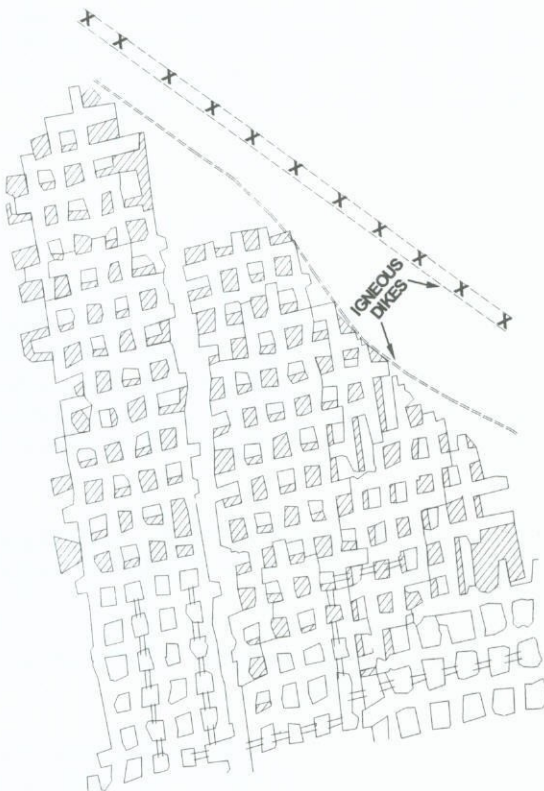


Figure 7. Checkerboard pillar extraction at Zululand Anthracite Colliery.

With extraction heights often approaching 5 m, setting breaker posts is a serious problem. Concerns about the size of the timber posts led to the development of the Voest-Alpine Breakerline Supports (BLS) at Middlebult Colliery in the mid-1980's. The BLS are no longer used, primarily because of the high capital costs. The recent trend has apparently been towards roof bolt breakerlines. Currently at Middlebult, for example, breakerlines consist of a single row of 1.2 m fully-grouted bolts.

INSPECTORATE, CODES OF PRACTICE, AND REGULATORY ISSUES

The role of the mine inspectorate in South Africa is currently evolving. On the one hand, it has much greater responsibility than it did in the past. The resources available to it have not increased proportionately, however. In Mpumalanga province, for example, the Principal Inspector is responsible for 70,000 miners in coal, gold, and chromium mines. He has just 23 inspectors (including specialists) for this huge job. The inspector/worker ratio is about 10 times greater than it is in the U.S.

Three years ago the inspectorate began a major tripartite initiative to reduce groundfalls through Codes of Practice (COP), a concept borrowed from Australia. A COP is drafted by a committee established by the mine manager in consultation with the mine's Health and Safety Committee, and approved by the Chief Inspector. Once approved, it has the status of a legal document and non-compliance is an offense. It is reviewed annually, and covers the geology, five-year historical rockfall analysis, panel design, roof support, and procedures for "special areas" (14).

The COP reflects the philosophy that "due to the complexity and variability of conditions at mines pertaining to the design, geometry, and support requirements, rigid and prescriptive guidelines would not be in the interest of safety" (14). Instead, the COP attempts to apply general principles combined with local expertise, experience, and knowledge. Indeed, even the COP is a general "strategy" document. For example, Sigma's COP contains statements like "ribside support will be installed where it is required in the opinion of the responsible official" (15). The real function of the COP is to guide the formulation of specific mine manager's rules, called the "standards and procedures." These include details like bolt pattern, cut sequences, etc.

The inspectors and rock mechanics specialists I spoke with expressed some disillusion with the COP approach. Apparently, three years on the COPs have not had the desired effect in reducing groundfall accidents. The emphasis is now shifting to implementation, through better training, improved roof support equipment and consumables, and higher quality roof support installations. Hazard recognition is considered a critical training element, in light of the relative inexperience of many roof fall accident victims.

Another controversial issue is accident investigations. Despite significant improvements since the issue was addressed by the Commission of Inquiry into the Safety and Health in the Mining Industry (24), there is still a lack of solid technical information on accidents. However, the inspectorate cannot provide the rock mechanics expertise to investigate every groundfall fatality with its small staff. Fortunately, a research project has recently been approved to address this issue.

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LABOR ISSUES

Labor issues are important because of the emphasis that is now being placed on "tripartism" to combat rock related accidents (14). Many of the labor issues faced by the South African mining community are a legacy of the old apartheid regime. Though black workers made up the big majority of the workforce, they were considered transient migrant laborers, not career coal miners. Black workers were housed in single-sex hostels at the mines, and were not allowed to bring their families. They were also barred from all but the least responsible jobs. Skilled white miners earned ten times as much as their unskilled black co-workers.

Recent years have seen dramatic improvements. Unions were legalized twenty years ago, and the National Union of Mineworkers has won large wage increases for its members. Blacks are now free to live where they please, and mines are increasingly staffed by a permanent, local workforce. Unfortunately, blacks formerly received so little education and training that it will take some time before the professional and management ranks are finally integrated. In the meantime, the racial gap can exacerbate the usual difficulties in labor-management communications.

ROCK MECHANICS COMMUNITY AND RESEARCH

Rock mechanics has a formal and important role in the South African mining industry, perhaps because of its central place in the gold mines. There is a difficult Rock Mechanics Certificate, similar to the U.S. Professional Engineering Certification. In addition, a "suitably qualified rock engineering practitioner" must participate in the development of Codes of Practice, and must approve any plans for "special areas" (14). There is some ambiguity about the definition of a "suitably qualified rock engineering practitioner", however.

The coal mining rock mechanics community consists of three parts. The three big mining houses have their own rock mechanics departments, each with a central office and field staff at the mines. There are consultants who carry out rock mechanics duties for the small mining companies and special projects for the large ones. And there is CSIR Miningtek, the government research agency which some years ago merged with the Chamber of Mines research organization. Communication between these three groups is formalized by their active participation in the Coalfields Branch of the South African National Group on Rock Mechanics (SANGORM). Currently there seems to be little involvement from academia in coal mine ground control.

Research is funded by a mechanism similar to the Australian model. As in Australia, a tax is collected from the mining companies, but the levy is based primarily on a company's accident rate rather than its production. The funds are administered by the Safety in Mines Research Advisory Committee (SIMRAC), a tripartite committee. SIMRAC solicits

competitive research proposals from all sectors of the mining community, on critical health and safety topics in the gold, coal and other mines. A total of 34 million Rand (about \$7 million) was spent on research projects in 1999, with 73% going to gold research and 18% going to coal. CSIR Miningtek performs much of the research. In coal mining, nine projects are addressing the issues of explosions, fires, face ventilation, and frictional ignitions. Another three projects are on rock mechanics topics, two on pillar design and one on roof support. While the emphasis on health is currently increasing, in 1997 there were apparently only two generic health projects for the entire industry, one addressing dust sampling and the other noise hazards.

SIMRAC also places a great emphasis on technology transfer. Symposia are held each year in the mining areas, and final reports are made available over the internet and on CD disk. Future planning is based both on surveillance data and stakeholder input expressed in strategic workshops.

CONCLUSIONS

During the last two decades, coal industries around the world have experienced wrenching changes. In South Africa, the coal industry has had to contend not just with economic dislocation, but with dramatic social and political changes as well. Thus far, the mining community (like South African society as a whole) has demonstrated a remarkable resilience and adaptability.

Much of the South African coal industry today resembles the room-and-pillar segment of the U.S. industry. A continuous miner section in the Highveld already looks much like a Coalburg seam operation in southern West Virginia. The increasing emphasis on health and safety in South Africa will surely increase the similarities.

South Africa has a long tradition of world leadership in mining rock mechanics. With its excellent connections to the industry and strong research orientation, the coal rock mechanics community should continue to deliver innovative solutions. U.S. roof bolting concepts and technology may also have quite a bit to offer South Africa. The coming years should see much valuable interaction.

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Rock Mechanics Issues in the Trona Patch

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ABSTRACT

The trona mines in the Green River Basin, commonly known as the "trona patch," present an interesting set of rock mechanics issues stemming from mining in a unique underground environment. Like other minerals occurring in tabular deposits, trona is mined using high-productivity room-and-pillar and longwall methods. However, trona does not behave quite like other evaporites and behaves far differently than coal.

In this paper, the following key issues in trona patch rock mechanics are discussed: material properties, *in situ* stress field, water and gas pressure effects, creep characteristics, pillar behavior, roof span stability, floor stability, underground tailings storage, longwall mining, two-seam mining, and solution mining.

Many of the existing trona producers are entering potentially more difficult mining conditions, and much consideration is being given to "new" technologies such as longwall mining and solution mining. In analyzing 20 years of experience with trona patch rock mechanics issues, the authors have identified key comparisons and contrasts with conventional tabular deposit rock mechanics that will help illuminate what has been learned as the trona patch enters the 21st century.

INTRODUCTION

The region encompassing the trona deposits near Green River, in southwestern Wyoming, has come to be known in mining circles as the "trona patch" because it accounts for most of the North American production. Trona is an industrial mineral which is refined into soda ash, used in the manufacture of glass, fiberglass, and many other industrial and consumer products. Approximately 25% of the world's soda ash production comes from trona mined in the Green River Basin (1). The trona is currently produced by five mines: (1) OCI Wyoming, L.P., (2) General Chemical Corporation, (3) Solvay Minerals, Inc., (4) Tg soda ash, inc., and (5) FMC Corporation (Figure 1). Four of

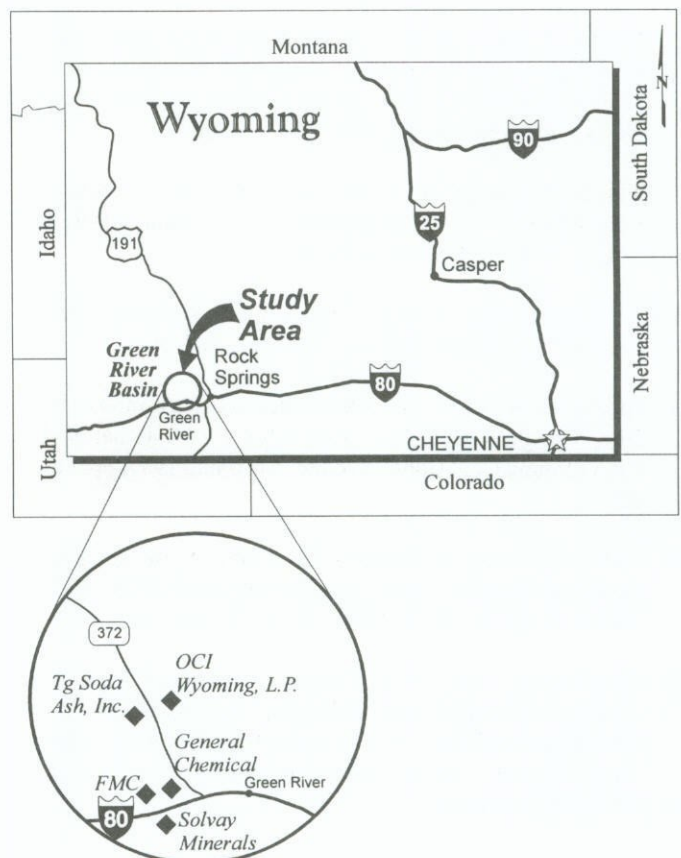


Figure 1. Location of Current Trona Mines in Green River, Wyoming, USA

these exclusively use room-and-pillar methods, while the fifth is primarily a longwall mine.

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This paper discusses rock mechanics issues in the trona patch, focusing on those unique to the extraction of trona in the Green River Basin.

MINING ENVIRONMENT

Geology

The Wilkins Peak Member of the Green River Formation contains approximately 25 important trona beds that represent deposits from evaporation of a large inland lake during Eocene time (Figure 2). These are numbered in stratigraphic sequence from the oldest (Bed 1). The most important beds currently being mined are Beds 17, 19, 20, 24, and 25. Bed 17 is the most productive to date, hosting the Solvay, FMC, and General Chemical operations. Tg is operating in Beds 19 and 20, while OCI mines Beds 24 and 25.

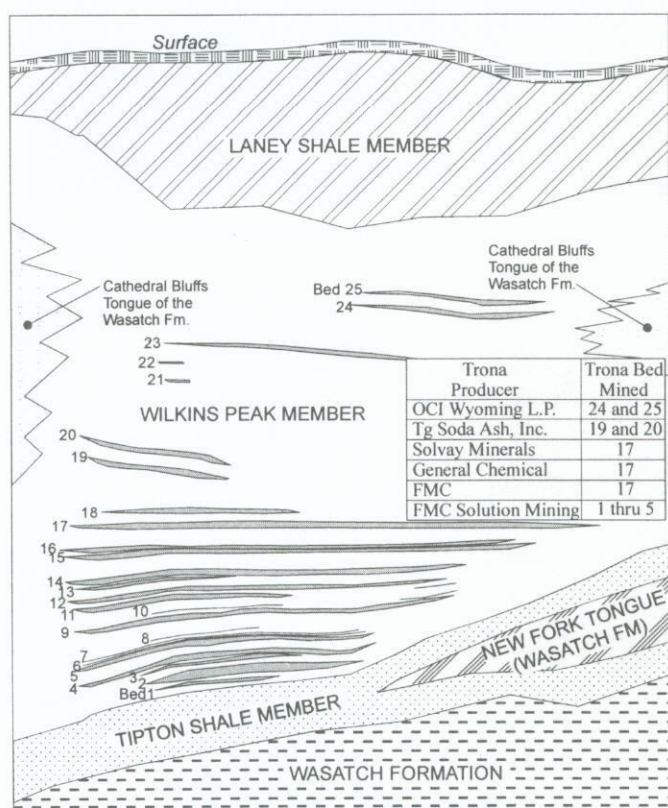


Figure 2. Generalized Geologic Column (after 1)

The trona occurs in a diverse bedded sequence that includes shales, oil shales, marlstones, sandstones, siltstones, limestones, and tuffs.

Material Properties

Trona is generally much stronger and stiffer than the immediate roof and floor rocks, with the degree of contrast varying with different beds and locations. Table 1 provides some typical values.

Table 1. Typical Nominal Values for Intact Rock Properties (true values vary with bed and location)

Horizon	Unconfined Compressive Strength (psi)	Elastic Modulus (psi)
Immediate Roof	6,000	1,000,000
Trona	8,000	3,000,000
Immediate Floor	4,000	500,000

Trona shows some time-dependent behavior, but few laboratory creep tests have been performed. Creep tests on model pillars (2, 3) show a much slower creep rate than salt. Much of the time-dependency exhibited in the field may in fact represent creep of weaker roof and floor strata rather than the trona bed itself.

Stress Environment

Repeated overcoring stress measurements by both the U.S. Bureau of Mines (USBM) (4, 5, 6) and Agapito Associates, Inc. (AAI) (7) mostly suggest a vertical stress gradient of 1.3 psi/ft depth and a horizontal:vertical stress ratio of approximately 1:1. Measured horizontal stresses are not sufficiently high to impact mine design; however, the high vertical stress gradient may be an issue.

Figure 3 shows close agreement between overcored vertical stress measurements in the rib of one room-and-pillar mine and a 1.3 psi/ft of depth vertical stress gradient. The model accounts for the stress effects induced by the local mining geometry and the stiffness of the roof, trona bed, and floor.

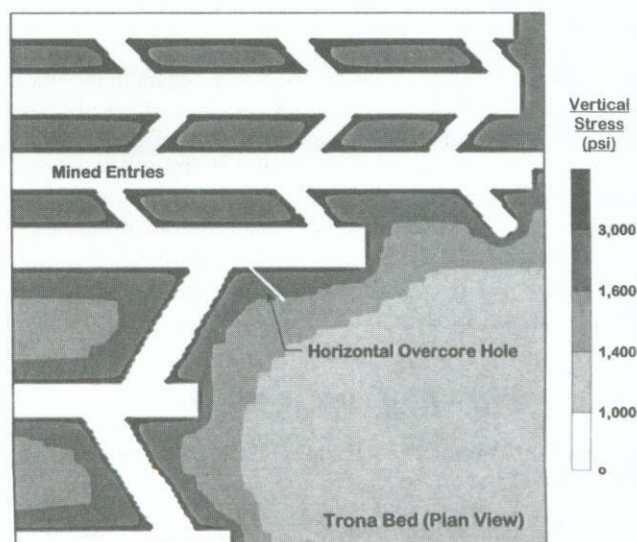


Figure 3a. Modeled Vertical Stresses at an Actual Overcore Stress Measurement Location, Based on a 1.3 psi/ft of Depth Vertical Stress Gradient

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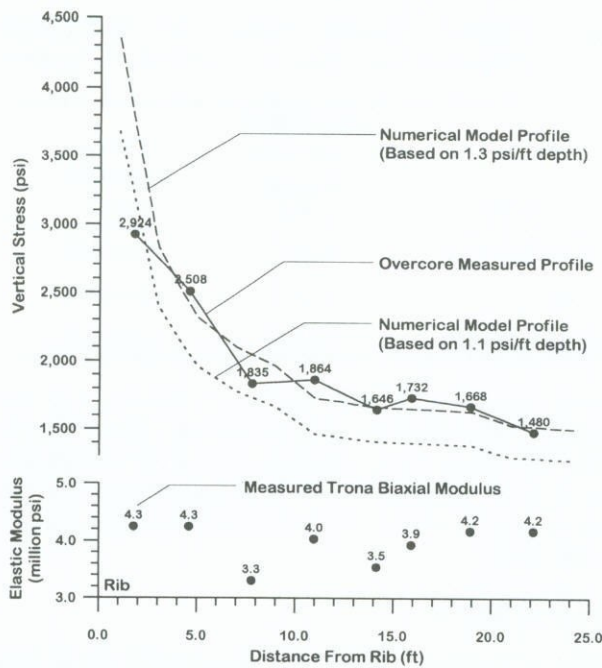


Figure 3b. Comparison of the Modeled versus Measured Vertical Stress Profile from Figure 3a

The back-calculated vertical stress gradient reflected in Figure 3b is somewhat higher than the 1.1 psi/ft calculated from gravity loading, although either fall within the scatter of relatively shallow vertical stress measurements elsewhere (8). Using the higher stress gradient *can* significantly change the results of design analysis, where the objective is to optimize the extraction ratio in the presence of weak roof and floor strata. More research is needed to resolve uncertainties regarding the stress field. In the interest of conservative design practice, we believe the higher stress gradient should not be dismissed arbitrarily.

Although trona mines are often dry, water occurs in the floor at some locations. Water can have a weakening effect of up to 50% or more on both the trona and the roof and floor rocks. Methane, ammonia, and other gases occur in the trona formations, with methane particularly abundant in the associated oil shales. Gas under pressure has been reported at several operations. Weller (9) reports gas cavities in Bed 17 at Solvay up to 25 ft × 15 ft × 1 ft in dimensions that contained sufficient gas pressure to eject rocks up to several hundred feet when intersected by mining.

ROOM-AND-PILLAR MINING

Key elements of rock mechanics for room-and-pillar trona mining include behavior of:

1. Roof/pillar/floor system
2. Roof spans
3. Yield pillars
4. Barrier pillars
5. Roof support

Despite the abundance of attention given to design of stable pillars, few instances of actual pillar failures have been reported in the trona patch because of the aforementioned contrast between the strength of the trona and the surrounding rocks. Upon closer inspection, many cases of "pillar failure" turn out to be pillar *foundation* failures (Figure 4). This is not just an issue of semantics because designs based on pillar strength alone without considering punching mechanisms are inadequate, much like high-rise building designs that ignore the foundation loading.

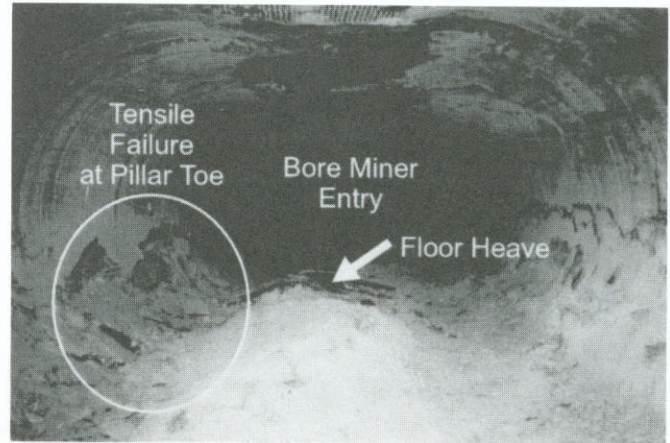


Figure 4a. Initial Rib Slabbing and Floor Heave in Response to Yielding of Pillar Foundation

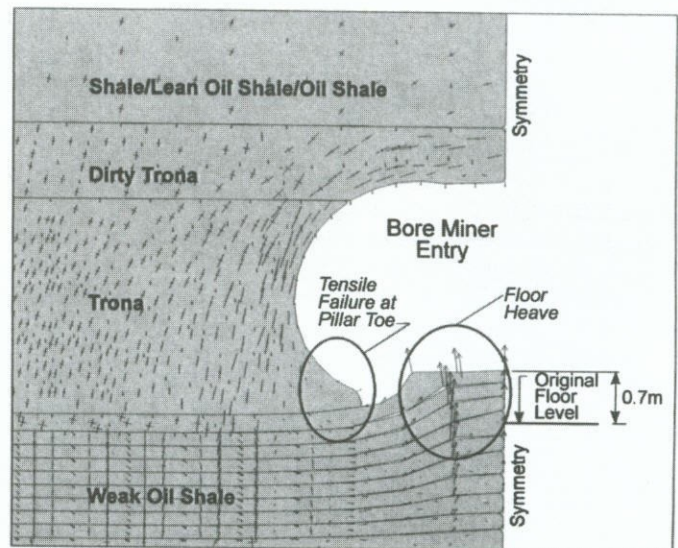


Figure 4b. Numerical Model of the Pillar Foundation Yielding Mechanism Pictured in Figure 4a (above)

The initial behavior shown in Figure 4 can ultimately result in pillar failure by progressive rib slabbing in response to tensile stresses induced by lateral deformation of the foundation strata, as shown in Figure 5. Loading of the weak roof and floor rocks can cause them to yield, weakening the pillar by developing tensile stresses in the trona. This behavior can be readily

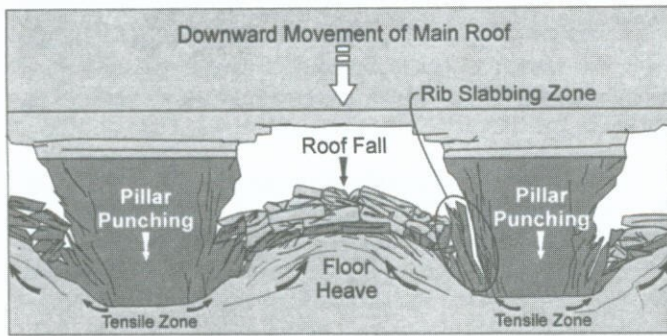


Figure 5. Ultimate Pillar and Roof Failure Resulting from Foundation Failure

demonstrated in the laboratory by placing a soft material in the specimen-platen interface prior to testing. Slabbing of the pillar edges is common even in conservatively sized rigid pillars and is not in itself an indication of instability. Further, core discing often occurs near the pillar edges, which is actually a manifestation of elastic pillar behavior.

Yield pillars are commonly employed in the mining industry to minimize floor heave and roof instability in areas subjected to high gravity or abutment loading. However, yield pillars in trona are problematic because of the characteristics of the roof/pillar/floor system. Rather than yielding by progressive rubblization like typical 30-ft-wide coal pillars, trona pillars remain relatively elastic (i.e., unyielding) even when they are quite thin (15-ft wide or less). Any yielding that does occur in the pillar is likely to be in the form of slabbing, which is difficult to control. If the pillar width is further decreased in an attempt to promote yielding, the stress ultimately becomes sufficient to result in foundation failure described previously (10).

Pillar widths may be reduced somewhat to induce a limited amount of pillar/floor yield in a panel to optimize resource recovery (7). Large arrays of thin pillars must be separated by barriers of sufficient width to isolate yield zones and promote stress transfer. Extraction ratios within the panel must be carefully selected, in some cases, to permit the panel to be completed before excessive closure limits access. Bore miners are particularly sensitive to this issue.

Roof span behavior in the trona patch is variable and depends on depth and local geology. Spans of 15 ft are common in Bed 17, and have been extended to 29 ft in the shallower cover conditions of Beds 24 and 25 with good success. Areas where pillar loading is high can result in "cutters" and increased risk of roof fallouts. This is particularly evident in Bed 20 where poor roof conditions are induced by yielding floor strata.

Smooth dome-shaped roof falls, some quite large, are sometimes initiated by high gas pressures in laminated roof strata (Figure 6). These tend to occur first in intersections where spans are greatest. A postulated mechanism involves (1) stress relief by roof relaxation above the intersection during mining; (2) concurrent increase in roof stress over the newly formed



Figure 6a. Smooth Dome-shaped Roof Fall Across Laminated Strata in an Intersection



Figure 6b. View of Dome-shaped Roof Fall from Crosscut

pillars; (3) migration of gas down-gradient from above the pillars to the intersection roof; and (4) roof span deflection leading to ultimate failure from gas pressure, as shown in Figure 7. All of the mines have resorted to drilling gas pressure relief holes in the roof, either routinely or on a spot basis, to prevent blowouts.

Typical primary roof support in the trona patch consists of 5- to 8-ft fully-grouted resin bolts on 2- to 5-ft centers. One shallow operator uses 54-inch mechanical anchor bolts instead with historical success. Bolts alone are generally sufficient to maintain good roof conditions. Occasionally mats or mesh are supplemented on a local basis. Rarely is more substantial support required, except in Bed 20 where particularly poor roof conditions are induced by yielding floor strata. Under these circumstances, heavy ground support consisting of grouted bolts on 1-ft row spacing, cable bolts (up to 20-ft long), cable slings, and mats is oftentimes required to control the roof (Figure 8).

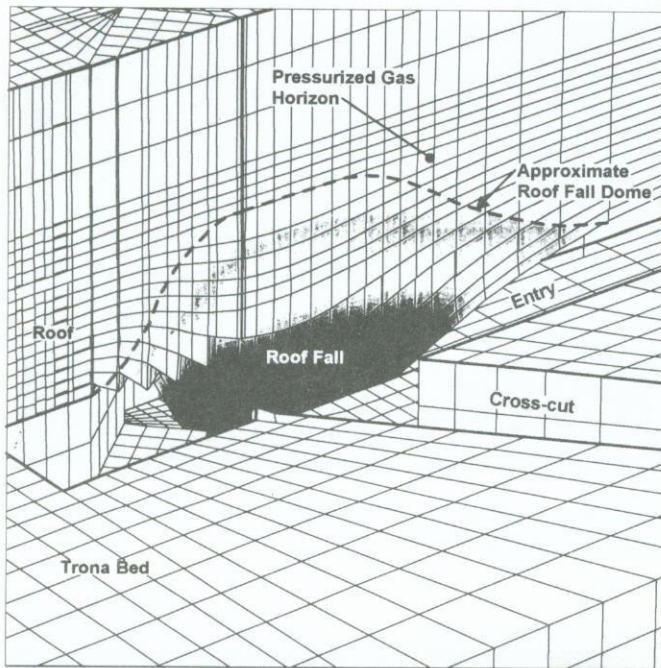


Figure 7. Numerical Model of the Domed Fall Initiated by Gas Pressure in the Roof Pictured in Figure 6

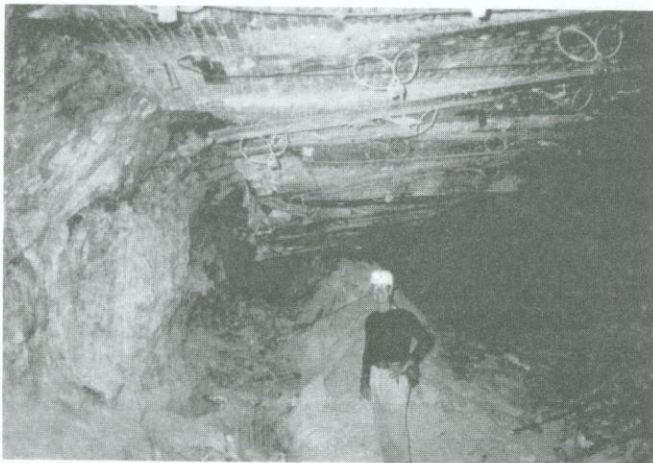


Figure 8. Heavy Roof Support Required Where Pillar/Rib Stresses Exceed the Strength of the Floor Strata (*note the use of cable bolts with head safety loops, mats, and cable slings*)

LONGWALL/SHORTWALL MINING

To date, two local operators have had experience with longwall mining of trona, both in Bed 17 (11, 12), and one has employed shortwall mining in Bed 20 (13). The first longwall was installed at General Chemical at a cover depth of 1600 ft. Many equipment-related problems were encountered due to the inadequacy of the contemporary longwall equipment, which was designed for cutting coal. Perhaps the primary rock mechanics problem was slabbing at the face. The larger slabs were

18-inches thick and up to 15-ft long, requiring secondary fragmentation prior to belt haulage. Other geotechnical issues included poor caving resulting in high abutment stresses leading to floor heave, and stress-induced fracturing and water inflow at the face. The longwall method was abandoned in the late 1970s.

Tg experimented with shortwall mining starting in 1982. The face support line consisted of 34 Dowty four-legged shields with a 660-T yield capacity (Figure 9). Extendable forepoles and a 5.5-ft stroke on the advancing rams allowed an 11-ft advance of the support tips. The last-used face length was 184 ft. Several ground control difficulties were encountered in the first panels. These included stress relief instabilities requiring heavy support in the tailgates and in development areas within the abutment zone of previously-mined panels. Average closure rates of 3 inches per day or more were reported in the front abutment area, requiring frequent cleaning of the floors. Floor heave experienced in early tailgates resulted in nonparallel roof and floor lines that created severe deformation in the shield bases and lemniscate links after repeated setting. Shortwall mining was discontinued in the mid-1990s.



Figure 9. Tg Shortwall Face Supported by 660-T Dowty Four-legged Shields

FMC started longwall mining in 1981, essentially contemporaneous with termination of the longwall experiment at General Chemical. Today FMC successfully operates the only non-coal longwall in the western hemisphere. Progress was slow in the initial panels, with productivity limited by slabbing at the face and slow cutting rates of the shearer, which was designed for cutting coal. Steady improvements in layouts, equipment, and operating practice that have come with experience have resulted in production rates as high as 5300 TPS (12). In their quest for longwall success, FMC was aided by improvements in durability of equipment such as longwall shearers, but their own innovations were crucial. These included a specialized shearer drum and other equipment modifications.

From a ground control perspective there are several important lessons to be learned from these experiences: (1) floor heave from abutment loading is a major problem in longwall and shortwall gateroads and can only be controlled by large rigid



pillars—yield pillars do not provide controlled entry closure; (2) in beds with an exceptionally sensitive floor, large barriers may be required to completely separate active panels from abutment zones from previous panels; and (3) where bore miners can be used, superior ground control is achieved.

CURRENT ISSUES

Longwall Mining

The success of FMC in Bed 17 has inspired other operators to look at longwall mining. We see no reason why longwall mining cannot be successful in other beds providing (1) subsidence can be tolerated, (2) the resource is sufficient for longwall panels, and (3) the roof and floor rocks have sufficient strength for efficient layouts.

More Difficult Conditions

It is likely that much of the resource will be more difficult to mine using mechanical mining techniques because of insufficient bed height, soft floor conditions, or depth. More difficult conditions have also been reported near the margins of individual beds because of dessication structures, gas, and other geologic features. Estimates of 500 years of production commonly cited (1), based on reserve estimates and current production rates, may be optimistic.

Depth

Although economic reserves exist in Beds 12 and 14, and possibly others below Bed 17 (1), trona has not been mined conventionally deeper than 1800 ft. Additional stress-related problems are expected at a depth of 2000 ft where a typical 50% extraction ratio produces an average pillar stress level of 5200 psi. A 5200-psi bearing stress is likely to exceed the strength of the pillar foundation in many of the beds. While there is substantial resource at shallower depths, future extraction of deeper beds is not expected to be economical with conventional mining techniques and will require more innovative methods, such as solution mining. FMC currently operates a solution mine pilot facility extracting brines from Beds 1 through 5 (14).

Multi-seam Mining

The existence of 25 important trona beds makes it a certainty that multi-seam mining will be required in the future. Beds 12, 14, and 19 contain economic reserves directly below some existing working in Beds 17 and 20 (1). To date there has been very limited experience with multi-seam mining of trona (15, 16). OCI has experience mining two overlying panels 35 ft apart in Beds 24 and 25 at less than 850 ft of depth. Indications from the OCI experience are that multi-seam mining is possible (17). As operating depths deepen, multi-seam mining will require added attention to panel alignment and sequencing, and provisions for increased ground support.

Underground Tailings Storage

Underground tailings storage is practiced at four of the five trona mines because of its long-term cost savings and environmental advantages. FMC and Tg both use non-managed placement systems that inject tailings directly into unmined workings from surface holes (18). Solvay and OCI use managed systems where low-water-content slurries are delivered via a pipeline from surface and through the mine to abandoned panels. Dams are used to control placement and decant water from settled tailings. OCI returns approximately 90% of tailings underground. In considering the long-term effects of the method, the weakening effect of water on trona measure rocks previously mentioned should be considered in the design of underground tailings storage areas, especially if subsidence is a concern. Subsidence monitoring should be established to assure performance.

Continuous Miner versus Bore Miner Entries

Bore miners are being used successfully at two trona mines in conjunction with the more common ripper-type continuous mining machines used by all five operators. The ovaloid-shaped entry produced with the bore miner has proven to be intrinsically more stable than the conventional rectangular opening produced with the ripper miner, principally by virtue of its rounder shape. This is illustrated in Figure 10, which compares the rock yield zone surrounding both types of entry shapes for equivalent entry dimensions and the same pre-mining stress state. The figure shows considerably more yielding in the roof and floor with the rectangular opening. The smaller yield zone shown for the ovaloid opening is borne out in practice in the form of more stable roof conditions, reduced floor heave, and less rib slabbing. At FMC, the dramatically superior entry stability realized with the bore miner versus the ripper miner allowed the primary and secondary longwall gateroad support systems to be streamlined, resulting in reduced costs and increased productivity (12). As mining conditions advance to greater depths and multi-seam mining becomes more prevalent, the inherent ground control advantages of the ovaloid entry must be weighed against the disadvantages of the bore miner, including the increased likelihood of becoming trapped in squeezing ground and the inability to regulate cutting height in response to changes in bed thickness.

SUMMARY

Ground response to mining in the trona patch is quite different from other evaporite deposits and coal. This is due to unique trona material properties, its higher strength in relation to roof and floor strata, and the geological environment.

More difficult mining conditions are anticipated in the future due to increased depths and geological features near the margins of the beds. The homogeneity in dips, bed thicknesses, and near-perfect ground conditions anticipated in the 1960s did not materialize, and ground control problems have occurred in some areas of all the mines. Estimates of several hundred years of production at current rates seem optimistic.

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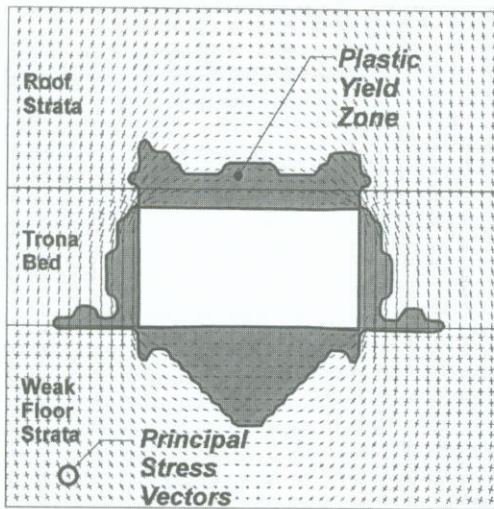


Figure 10a. Extent of Yield Zone Around a Rectangular Continuous Miner Entry

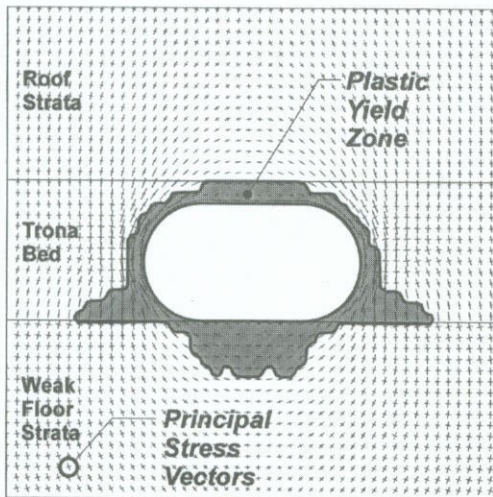


Figure 10b. Extent of Yield Zone Around an Ovaloid Bore Miner Entry

Room-and-pillar mining has been used efficiently to extract most of the trona to the present. However, it is likely that longwall mining will be used more efficiently in the future to achieve better productivity and resource recovery at depth. Solution mining offers the possibility to allow extraction in deeper and thinner beds and compliments the other mining methods.

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Highwall Augering in Ultra-thick Western Coal Reserves: Unique Geotechnical and Operational Challenges

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ABSTRACT

The Pittsburgh & Midway Coal Mining Co.'s Kemmerer Mine is one of the deepest surface coal operations in the world, with the highwall extending to approximately 1,000 ft above the pit floor. To increase recovery, auger mining is being evaluated beneath the highwalls of several seams representing the current economic limit of pit development. The primary augered seam is over 50 ft thick, can approach 100 ft thick, and dips into the highwall at an average of 17 degrees. The seam thickness and dip present many geotechnical design and operational challenges for multiple (up to five) lift augering. This paper presents the design criteria and methodology applied to determine appropriate web and septum thicknesses for the various cover depths anticipated, as well as operational measures that can be taken to increase overall coal recovery.

The primary tool used in the design was the finite-difference code FLAC. Models incorporating typical strata sequences associated with the coal seam were run at several different cover depths corresponding to the maximum anticipated auger penetration. Web and septum thicknesses were varied until the minimum thickness satisfying a given safety factor criterion was determined. Using this procedure, design curves were developed for web and septum thickness versus cover depth.

Operationally, several recommendations were developed to ensure that overall coal recovery is maximized. By augering across the seam dip, auger penetration per hole can be increased dramatically, and by carefully planning maximum planned auger penetration versus hole spacing, increased coal recovery can be safely attained.

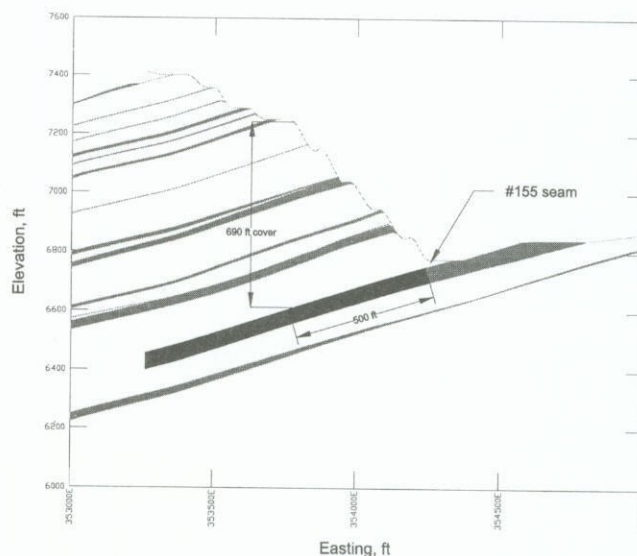


Figure 1. Typical profile of the #155 Seam.

INTRODUCTION

To increase reserve recovery at its Kemmerer Mine near Kemmerer, Wyoming, The Pittsburgh & Midway Coal Mining Co. (P&M) is investigating the feasibility of auger mining several seams beneath highwalls representing the current economic limit of surface mining. The Kemmerer Mine is one of the deepest surface coal mines in the world, with the highwall extending to approximately 1,000 ft above the pit floor. The primary augered seam (the #155 Seam, Figure 1) is over 50 ft thick, can approach 100 ft thick, and dips into the highwall at 17 degrees. In an effort to optimize auger coal recovery while leaving highwall stability unaffected, P&M retained NSA Engineering, Inc. (NSA), to develop augering designs for each seam under consideration.

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Table 1.- Comparison of augering designs in Australia and the United States.

Location	Australia	Australia	Australia	Texas	New Mexico	Wyoming	Wyoming
No. Lifts	2	4	2	1	2	1	1
Design Approach	Numerical	Numerical	Numerical	Empirical	Emp./Num.	Empirical	Emp./Num.
WEB PILLAR GEOMETRY							
Auger Diameter (ft)	4.25	4.25	4.25	6.00	6.00	6.00	5.00
Maximum Cover (ft)	290	140	300	180	490	220	200
In Situ Coal Strength (psi)	754	310	464	600	725	619	755
Web Pillar Width (in)	31	20	26	72	108	36	40
WEB PILLAR SAFETY FACTOR							
Salamon-Munro	1.01	0.53	0.50	2.02	1.29	0.83	1.57
Obert-Duvall	0.90	0.54	0.47	1.67	0.99	0.83	1.40
Bieniawski	0.84	0.49	0.43	1.67	1.05	0.77	1.33
Holland-Gaddy	0.77	0.39	0.37	1.67	1.09	0.66	1.23
Mark-Bieniawski	0.95	0.53	0.48	1.97	1.29	0.85	1.51
Average	0.89	0.49	0.45	1.80	1.14	0.79	1.41

The seam thickness and dip present many geotechnical design and operational challenges for multiple-lift (up to five) augering. Multiple-lift augering is not common in the United States, and where it has been practiced, it has been limited to two lifts. Since its introduction to Australia in the early 1990's, however, multiple-lift augering has become the norm (1). This paper compares design approaches previously applied in Australia and the United States, and details NSA's approach to the multiple-lift augering design at the Kemmerer Mine.

REVIEW OF APPLICABLE DESIGN METHODS

The goal of highwall augering design is to ensure that highwall stability is unaffected by augering. To accomplish this, web pillars left laterally between holes must remain stable, as must the coal left between lifts (the septum). Because the multiple-lift augering considered for the Kemmerer Mine is unprecedented in the United States, standard design methods have not been established. To determine the design approach most applicable to auger mining at the Kemmerer Mine, NSA reviewed design approaches previously applied in Australia and the western United States. Past augering designs have employed either numerical modeling, empirical pillar design formulas developed for underground coal mining, or a combination.

Table 1 presents several augering designs developed within the past 10 years for western United States and Australian mines. For comparison, web pillar safety factors were calculated using tributary area loading and various empirical pillar design equations (see Mark (2), for a summary of these pillar design equations).

Although they do not account for site-specific conditions, calculated web pillar safety factors shown in Table 1 suggest that wide disparity exists between designs, with Australian designs generally more aggressive than those of the western United States. Table 1 reflects that most Australian augering applications involve multiple lifts, perhaps explaining why numerical modeling methods are preferred there over empirical

methods. Empirical methods are not well suited to multiple-lift augering design for several reasons, including (1) they were developed for rectangular openings, which result in higher stress concentrations around the opening as compared to circular auger holes; this effect is more pronounced when openings are close together, as is the case with auger mining; (2) they relate pillar strength to width-to-height ratio and were developed for much higher width-to-height ratios than are encountered in auger mining; (3) with the exception of the Mark-Bieniawski formula (3), they do not account for pillar strength increase with length; this effect is significant in auger mining as web pillars are as long as the auger penetration into the highwall. Perhaps most importantly, the empirical equations were developed for mining on a single level and, therefore, are fundamentally inappropriate for multiple-lift mining (unless septum thickness is great enough to isolate each lift—an unattractive option due to decreased seam recovery).

Differences between web pillar designs based on empirical equations (rectangular openings) and numerical modeling (which can account for circular openings) are illustrated in Figure 2 using the FLAC finite-difference code (4). Strain-softening elements were used to account for the effects of stress transfer from yielded coal elements near the periphery of the opening. All cases are for 400 ft of cover, with an in situ coal strength of 1,100 psi. According to the empirical design equations in Table 1, a web pillar 6 ft wide is required to satisfy a 1.5 safety factor criterion for a rectangular opening 6 ft wide by 6 ft high. Figure 2a shows this configuration modeled with FLAC. Mohr-Coulomb safety factors in the core of the web pillar are at least 1.5. The same web pillar width next to a circular opening (6 ft diameter) is shown in Figure 2b; Mohr-Coulomb safety factors throughout the core of the pillar are in excess of 3.0. Similarly, a web pillar width of 30 in satisfies a 1.5 Mohr-Coulomb safety criteria adjacent to a circular opening (Figure 2c), whereas the same width pillar next to a rectangular opening (with a safety factor based on empirical equations of 0.64) is shown to fail (Figure 2d). So, even though there is good correlation between safety factors from numerical modeling and the empirical design equations for rectangular openings, NSA

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concluded that numerical modeling is more suited to augering design at the Kemmerer Mine because of its ability to account for circular openings and the effect of multiple lifts.

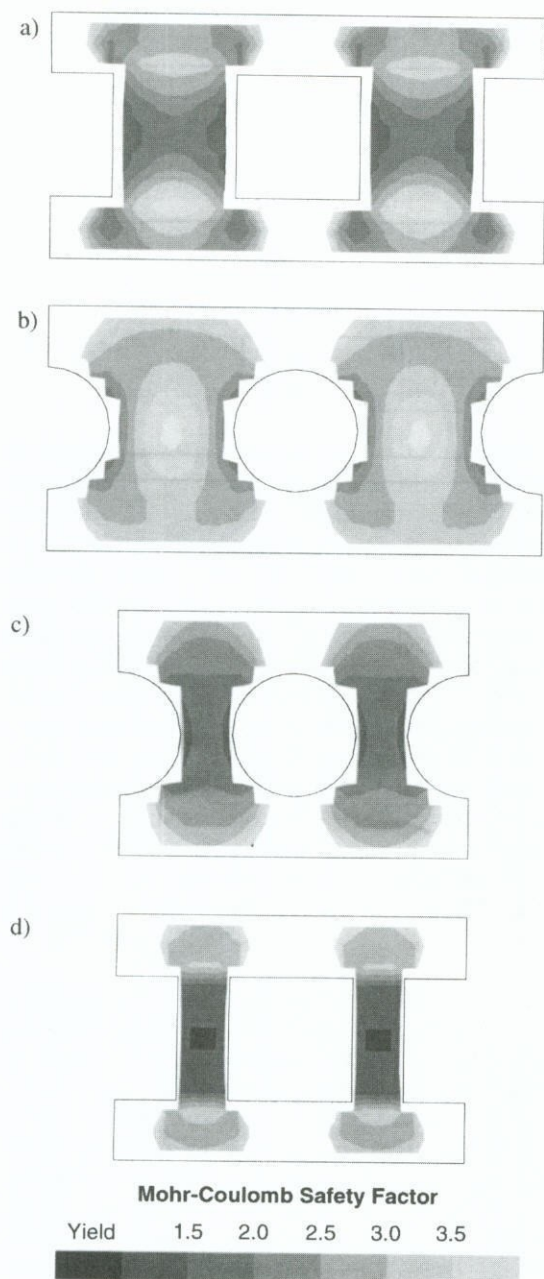


Figure 2.- Comparison of stable web pillar designs using empirical equations and numerical modeling. (a) 6-ft web pillar adjacent to a 6- by 6-ft rectangular opening, predicted by empirical formulas to satisfy a 1.5 safety factor criterion, correlates with Mohr-Coulomb safety factors of at least 1.5 in the web pillar core; (b) 6-ft web pillar width next to a 6-ft auger hole has Mohr-Coulomb safety factors of at least 3.0 throughout the web pillar core; (c) 30-in web pillar next to a 6-ft auger hole satisfies a 1.5 Mohr-Coulomb safety criterion; (d) 30-in web pillar adjacent to a 6- by 6-ft rectangular opening is predicted to fail using empirical formulas (safety factor of 0.64); numerical modeling results show yielding throughout the web pillar core.

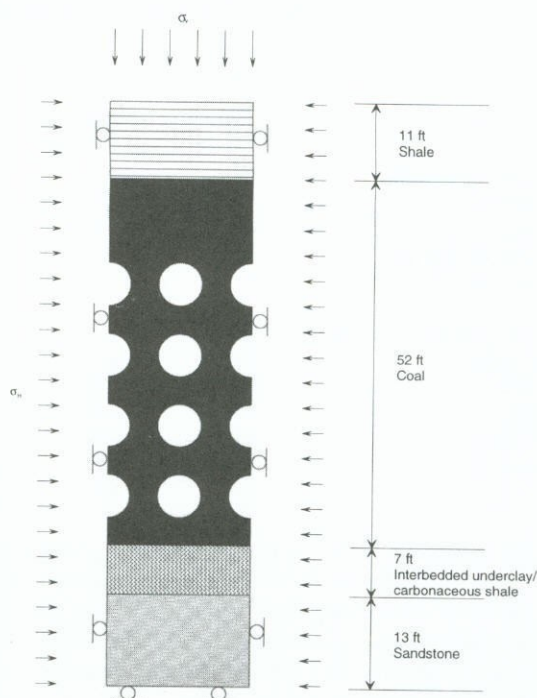


Figure 3. FLAC model geometry for the #155 Seam showing stratigraphy and loading conditions.

NSA DESIGN APPROACH

As part of its design for augering at the Kemmerer Mine, NSA performed several tasks, including (1) a site visit in which each augering area and seam was inspected, relevant information from previous geotechnical studies was gathered, deficiencies in the available data were identified, and arrangements were made for the collection of additional data; (2) assessment of the structural characteristics and intact rock physical properties of the highwall strata bounding the seam horizons to be augered; (3) numerical modeling evaluations of various hole spacings for both single and multiple lifts, to arrive at optimum spacings as a function of depth and auger hole diameter; and (4) the development of operational measures to increase efficiency and promote safety.

The pattern of auger holes (stacked versus staggered) and recommended auger hole spacing as a function of cover depth were determined using two-dimensional FLAC models of various auger hole arrangements (and resulting web and septum thicknesses). Appropriate input parameters were determined through on-site observations, review of data provided by P&M, and core sample testing.

Based on borehole logs and seam thickness contour maps provided by P&M, a typical column for the #155 Seam was developed (Figure 3), including the coal seam and strata immediately above and below the seam. In FLAC, planes of symmetry were included on the right and left hand sides of the

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models so that the models simulate a seam of infinite lateral extent. Including the strata above and below the seam is important in that the behavior of these layers affects the stress distribution in the coal seam. Also, if weak layers bound the coal, failure may occur through the mechanism of coal webs punching into these layers.

To determine appropriate input for the numerical modeling analyses, geotechnical studies provided by P&M containing physical property data were reviewed and compiled, including results of unconfined compressive strength (UCS) tests, triaxial strength tests, tensile strength tests, direct shear tests, and point load tests. Table 2 presents the resulting physical property parameters used in the FLAC modeling analyses.

Table 2.- Physical property parameters used in FLAC modeling analyses.

Rock Type	Friction angle (ϕ), degrees	Cohesion (c), psi	Tensile Strength (σ_t), psi	Young's Modulus (E), psi	Poisson's Ratio (ν)
Sandstone	37.7	1,060	350	1,900,000	0.25
Underclay	27.0	200	65	100,000	0.35
Carb. Shale	30.0	700	300	1,500,000	0.25
#155 Coal	34.8	290	100	300,000	0.35

To check whether or not the strength values shown in Table 2 are reasonable, the geometry of past underground mining at Kemmerer was modeled. Unconventional underground coal mining methods have been tried on the property in the past, including "slusher" mining (a hard-rock mining method) in a 100-ft-thick seam. This type of mining resulted in large openings (35 ft wide by 42 ft high), with 50-ft pillars between openings. Indications are that the coal ribs and roof stood up very well, with no bolting (this mining was done prior to the Federal Coal Mine Health and Safety Act of 1969). Figure 4 shows safety factors from FLAC modeling of this geometry at 600 ft of depth, using the physical properties from Table 2. Stability is maintained throughout most of the model, with small areas of yielding in the coal ribs. Because actual mining experience is that coal ribs stood for many years, these results suggest that the physical property values given in Table 2 are somewhat conservative, yet reasonable.

In addition to the stratigraphic columns and physical properties associated with the various seams, the assumed in situ stress field has a large effect on the stress distributions and safety factors that result from augering. Measurements of horizontal stresses throughout the world have shown that horizontal stresses tend to exceed vertical stresses under the cover depths that will be encountered during augering at the Kemmerer Mine (5). However, creation of the highwall relieves horizontal stresses, and horizontal stresses near the highwall are more likely to be from self-loading (Poisson's effect, commonly resulting in a 3:1 vertical-to-horizontal stress ratio). The actual vertical-to-horizontal stress ratio at the Kemmerer Mine is unknown, although it is likely to be in the range of 1:1 (at the limit of auger penetration) to 3:1 (near the highwall). To examine the effect of assumed loading conditions on auger hole

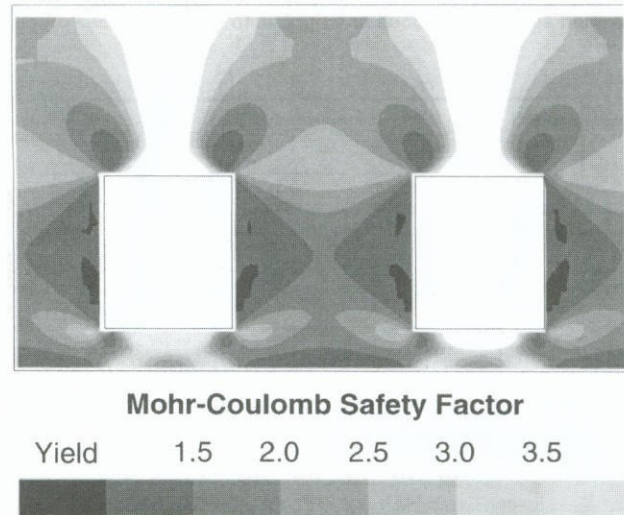


Figure 4. Model of past underground mining at the Kemmerer property confirmed that physical properties used for the #155 Seam were reasonable.

stability, FLAC models were run with varying vertical-to-horizontal stress ratios. These models demonstrated that web stability decreases with increasing vertical-to-horizontal stress ratio. Septum stability was maintained throughout, and increased with vertical-to-horizontal stress ratio. In all the models used to determine recommended web and septum spacing, a 3:1 vertical-to-horizontal stress ratio was assumed. This assumption builds a certain degree of conservatism into the design if the actual vertical-to-horizontal stress ratio is lower (down to 1:1), which may very well be the case under deeper cover.

EFFECTS OF AUGERING GEOMETRY VARIATION

Basic effects of auger hole geometry were examined, specifically the effects of vertically stacked versus vertically staggered holes, the effect of auger hole size, and the relationship between the number of stacked holes and the horizontal hole spacing. These effects were examined so they could be understood and eliminated as variables when determining vertical and horizontal hole spacing, thereby greatly simplifying the analysis.

Vertically Stacked versus Vertically Staggered Arrangements - The most basic question to be answered involved which arrangement provided greater stability for multiple lifts, vertically stacked (auger hole centers aligned) or vertically staggered (auger hole center aligned with the center of the web of the adjacent lift). To answer this question, a two-lift arrangement was modeled, using a 6-ft hole diameter. Vertical and horizontal spacings were the same in each model, and an 800-ft cover load was assumed. Results are presented in Figure 5. Clearly, the vertically stacked arrangement provides greater stability, as the vertically staggered arrangement has low safety factors in the septum between lifts. It was therefore

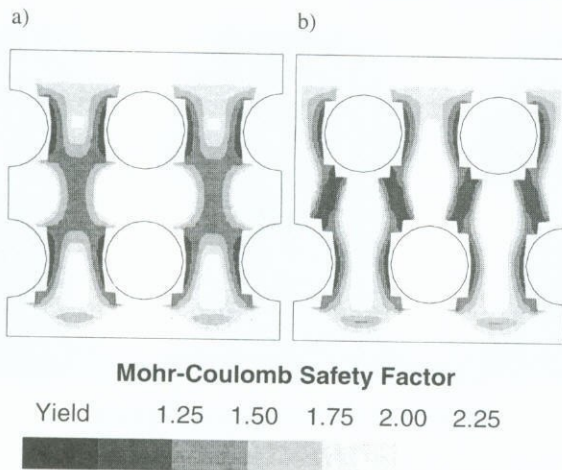


Figure 5.- Comparison of vertically stacked (a) versus vertically staggered (b) augering arrangements. For a given hole spacing, the vertically stacked arrangement has a higher stability; however, for conservatism, recommended hole spacings were based on the staggered arrangement.

recommended to P&M that every effort be made to vertically stack holes. However, because it is not possible to control wandering of the auger within the hole, which increases with auger penetration, NSA based its web and septum designs on the vertically staggered arrangement to account for worst case conditions at the back of the hole.

Ratio of Web-to-Septum Thickness - FLAC modeling demonstrated that the optimum ratio of web-to-septum thickness is close to 1.0; for simplicity in field application, NSA recommended to P&M that identical septum and web thicknesses be used.

Effect of Auger Hole Size - Theoretically, the stress distribution around a circular opening in an infinite medium is independent of hole size, that is, stresses are identical at a given distance (in terms of hole diameter) into the web. Therefore, a design based on 6-ft holes, for example, should apply to smaller or larger holes, so long as the required distance between holes is expressed as a function of hole diameter. Because the effect of bounding strata may affect the validity of this assumption, models were run for the #155 Seam using 3-ft-diameter holes, with the same web pillar-to-hole diameter ratio that was found to be adequate for 6-ft holes. It was concluded that for the #155 Seam, the effect of the bounding strata was negligible.

Effect of Stacking on Required Vertical Spacing - Because of the basic differences in the induced stress field associated with a single lift of holes versus stacked holes, it was expected that the required web thicknesses for multiple-lift designs would have to be greater than for single-lift designs. Comparative models of single, two-lift, and four-lift designs are shown in Figure 6. These results show that the horizontal spacing required for stacked holes is greater than that for a single row of holes. However, this effect does not increase with the number of lifts (required horizontal spacing for two stacked holes (Figure

6b) is essentially the same as that for four stacked holes (Figure 6c)).

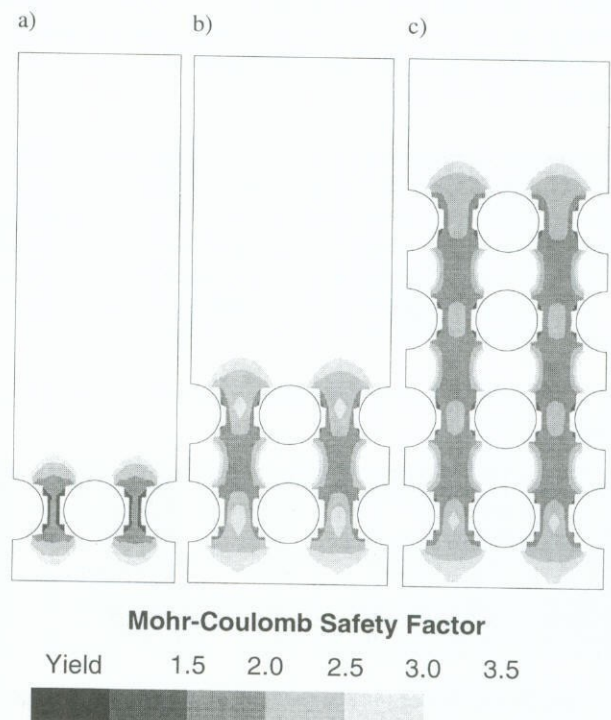


Figure 6.- Comparison of single-lift (a), two-lift (b), and four-lift (c) augering arrangements. Required web and septum thicknesses increases going from a single-lift to a two-lift arrangement, however, the required spacing does not increase for additional lifts.

HOLE SPACING RECOMMENDATIONS

With the effects of hole pattern, hole size, and multiple lifts addressed, the process of determining recommended hole spacing as a function of cover depth was greatly simplified. It was recognized that a numerical model can never perfectly represent a rock mass. For example, attempting to model the coal cleat in this case would introduce more uncertainty than insight, due to the lack of cleat properties and field verification opportunities. Therefore, several conservative assumptions were incorporated into the design, including:

- 1) Hole spacing was based on worst-case "back of the hole" cover loading. Due to the 17 degree dip of the coal seam and the 45 degree slope of the highwall, cover depth decreases rapidly from the back of the hole to the highwall. Designing for the deepest cover load, therefore, results in rapidly increasing factors of safety toward the highwall.
- 2) As previously mentioned, designs were based on the worst case stress distribution from a staggered hole arrangement, even though every attempt will be made to stack holes from successive lifts.

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- 3) The 3:1 vertical-to-horizontal stress ratio assumed is likely conservative, especially for the deeper cover loads upon which the spacing recommendations are based.
- 4) For multiple lifts, the 1.5 safety factor criterion is the *minimum* allowable safety factor in the web core or septum; the *average* safety factor in the web or septum is generally much higher.

Using these assumptions, several models were run at cover depths of 300, 400, 600, and 800 ft, in web and septum spacing intervals of 0.25 ft, to determine the minimum web and septum spacing that satisfied the 1.5 safety factor criterion for each depth. Figure 7 illustrates this procedure. Figure 7a shows the multiple lift arrangement at 800 ft of cover and an 8.5-ft web and septum spacing. The 1.5 safety factor criterion is satisfied, as the cores of the webs have minimum safety factors in the 1.5 to 2.0 range. When web and septum spacing is decreased to 8.25 ft (Figure 7b), safety factors less than 1.5 are evident in the cores of the septa between holes. Therefore, the 8.25 ft spacing was rejected, and the 8.5 ft spacing was recommended.

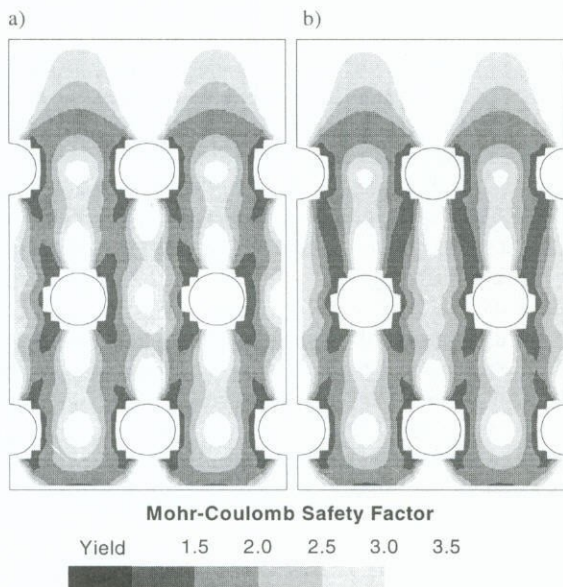


Figure 7. (a) Example of a hole spacing that satisfies a 1.5 safety factor criterion. (b) Decreasing web and septum thicknesses by 0.25 ft, safety factors fall below 1.5 between holes, and the criterion is not satisfied.

Using this procedure, the design curve shown in Figure 8 was developed. Center spacings were normalized to the hole diameter.

OPERATIONAL CONSIDERATIONS

The design curve presented in Figure 8 provides P&M with a rational starting point for auger hole spacing in the #155 Seam. By using this design curve to determine minimum hole spacing and adjusting hole spacing as conditions warrant, maximum

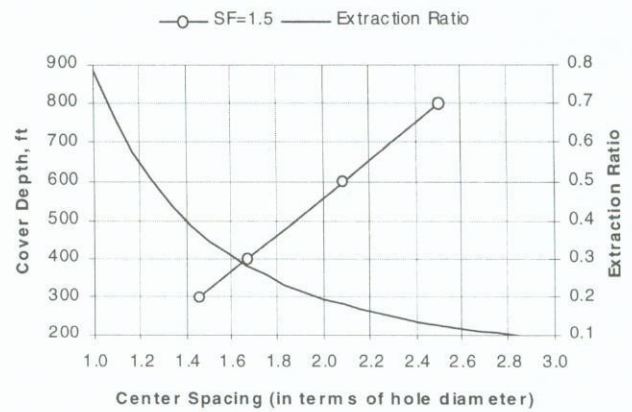


Figure 8.- Auger hole spacing as a function of cover depth, #155 Seam.

resource recovery (from a hole spacing standpoint) can be safely attained. In addition, the following operational recommendations were developed to help ensure that maximum overall recovery can be attained for each of the seams under consideration.

Augering at an Angle off Perpendicular - The highwalls at the Kemmerer Mine are generally along the strike of the seam, meaning that if the seams are augered perpendicular to the highwall, as is normally the case, augering will be along the dip. However, the seam dips, which vary between 14 and 24 degrees, may exceed the capabilities (both physically and with regard to horsepower) of the augering machine. To minimize the angle at which augering will be accomplished, NSA recommended to P&M that auger holes be oriented at an angle off the perpendicular.

Based on examination of the Brydet augering machine that is anticipated to be used and on conversations with the augering contractor, an angle off the perpendicular of up to 30 degrees is practical. Table 3 shows the average effective dip both perpendicular to the highwall and 30 degrees off the perpendicular.

Table 3. Average effective dip perpendicular to and 30 degrees off perpendicular to the highwall.

Area	Effective dip	
	Perpendicular	30° off perpendicular
#155 Seam	17	14
#255 Seam	24	20
#355 Seam	20	14
#450 Seam, north	20	17
#450 Seam, south	16	13

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As is shown in Table 3, orienting the augering holes 30 degrees off the perpendicular significantly reduces the effective dip. This reduction is especially evident for the #255 Seam. Figure 9 illustrates the improvement in recovery that may be attained by angling the holes off the perpendicular in the #255 Seam. In Figure 9a, augering is perpendicular to the highwall, along the dip of the seam, which is 24 degrees. Assuming the maximum angle that the augering machine can operate from horizontal is 18 degrees (as indicated by the contractor), two 6-ft holes can be augered; the lower hole will penetrate 180 ft before hitting the top of the seam, whereas the upper hole will penetrate only 110 ft. This gives an average hole depth of 145 ft and yields 45 tons per linear ft of highwall. By augering at 30 degrees off the perpendicular, the effective dip is reduced to 20 degrees (Figure 9b). The bottom hole can reach more than 500 ft, likely beyond the horsepower limit of the augering machine. The top hole can penetrate 470 ft before it hits the top of the seam, for an average hole depth of at least 485 ft. This converts to 82 tons per linear ft of highwall, an 82% increase over augering directly down dip.

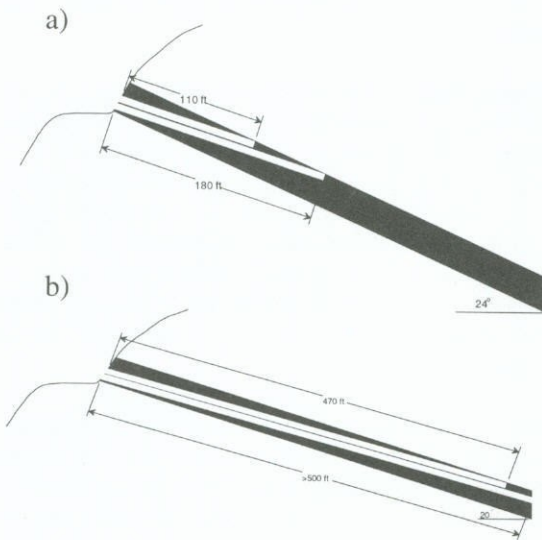


Figure 9. Auger hole penetration in the #255 Seam (a) perpendicular to the highwall, and (b) 30 degrees off perpendicular. In this case, augering across the dip results in an 82% increase in recovery.

It is important to note that if holes are augered at an angle from perpendicular, spacing along the highwall will have to be increased by the inverse of the cosine of the angle off perpendicular in order to maintain the proper spacing between holes.

Strategies for Hole Spacing Based on Maximum Anticipated Penetration - To apply the design curve in Figure 8, the cover depth at the back of the hole must be determined; this, in turn, is a function of how deep augering will penetrate the highwall. Intuitively, it would appear that every effort should be made to

auger as far as possible into the highwall, up to the limit of the machine. However, because of the combined effects of the seam dip (14 to 24 degrees) and the overall slope of the highwall (45 degrees), cover depth increases rapidly with penetration into the highwall. Under certain circumstances, reserve recovery may be increased by limiting penetration so that hole spacing can be tightened both vertically and horizontally. To illustrate this point, consider the case of the #155 Seam, with an average thickness of 50 ft. Table 4 shows recoverable tons per linear ft of highwall.

Table 4. Tons/ft of highwall, #155 Seam, 50-ft seam height.

Hole length	Cover depth	Center spacing	Number of lifts	Tons/ft of highwall
300	328	8.49	5 (+5 ft hole)	233 (205 + 28)
350	383	9.45	5	215
400	438	10.27	4 (+4 ft hole)	201 (181 + 20)
450	493	11.03	4	189
500	547	11.77	4	197

For each given hole length, a cover depth was calculated based on a 20 degree seam dip and an average highwall angle of 45 degrees. For the resulting cover depth, the required hole spacing for a 1.5 safety factor was taken from Figure 8. Finally, the number of lifts that can be fit into the 50-ft seam was calculated for 6-ft holes, adding any remainder that could be augered using a smaller diameter hole. Total tons for each set of holes was then calculated by multiplying the number of holes by their area and the depth of hole. Tons/ft of highwall was then calculated by dividing by the center spacing. The resulting recovery is plotted against depth of hole in Figure 10, which shows that for the #155 Seam, recovery actually decreases with increasing hole penetration, due to the requirement for leaving thicker webs and septa. The requirement to leave thicker septa results, at some point, in entire augering rows being dropped.

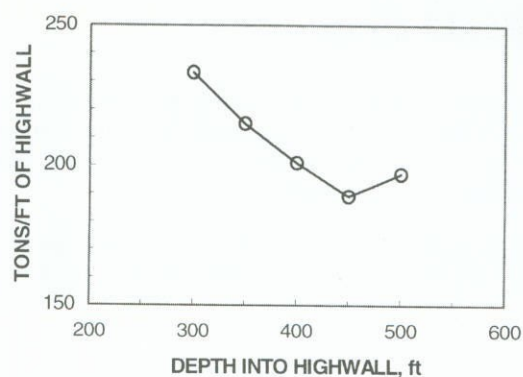


Figure 10.- Coal recovery versus depth of auger penetration for the #155 Seam. Because of the rapidly increasing cover from the highwall, required web and septum spacing also increases rapidly, resulting in a general decrease in recovery with planned auger penetration.

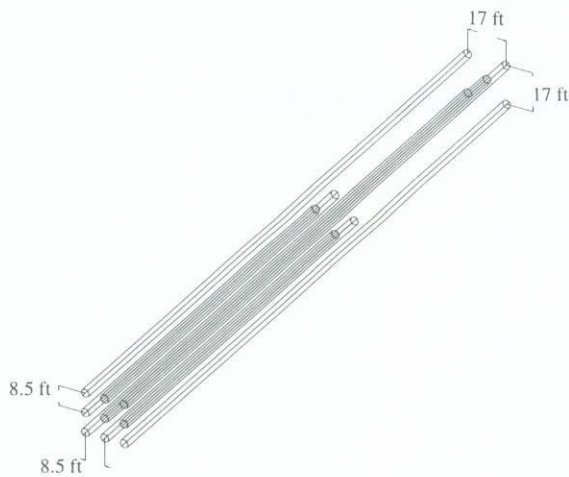


Figure 11. Augering concept wherein hole spacing is based on 300 ft of hole penetration. Alternate holes can be taken back as far as the horsepower of the machine will allow while still satisfying spacing requirements.

In addition to the relationship shown in Figure 10, it makes sense to design for shorter holes, because the likelihood of completing a short hole is greater than the likelihood of completing a long hole. With a hole spacing set (for example, based on 300 ft of penetration), it will be possible to drill every other hole as far as augering will allow. Figure 11 illustrates this concept, where holes are laid out on 8.5-ft centers, appropriate for the cover depth associated with 300 ft of penetration. If every other hole is taken back 500 ft, the actual center spacing of these deep holes is 17 ft, or twice the spacing for the shallow holes. This results in a center spacing, in terms of hole diameter, of 2.8, which is far in excess of that required even for a cover depth of 800 ft. For this example, augering every other hole to 500 ft satisfies spacing requirements while increasing recovery by 33%.

SUMMARY AND CONCLUSIONS

The highwall augering design developed for the Kemmerer Mine is unique in the United States in that it calls for up to five augering lifts. Empirical pillar design methods commonly used

for augering design in the western United States were found to be inadequate in this instance because they do not account for the stress distribution in the web pillar associated with circular openings, nor do they account for the effects of multiple augering lifts. Instead, the FLAC finite-difference numerical modeling code was chosen for augering design at the Kemmerer Mine. By first determining the effects of variations in basic auger hole geometry, these effects were eliminated from the analysis of web and septum spacing versus cover depth, greatly simplifying the design procedure.

In addition, minimizing auger hole spacing while maintaining highwall stability, operational measures can be taken to ensure maximum overall coal recovery. These measures include augering at an angle off the perpendicular, and basing hole spacing on a specific, optimized auger penetration.

The highwall augering design developed provides P&M with a rational starting point for augering at the Kemmerer Mine. As with any design, field performance should be evaluated on an ongoing basis, with necessary adjustments made to increase safety and productivity.

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Seismic Events Due to Underground Mining Activities

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ABSTRACT

In order to investigate locations, source parameters, and source mechanisms of mining induced seismic events, a local network of seismic stations was installed at a coal mine in the eastern part of the Ruhr District (Germany). The recorded events were located with the help of an anisotropic velocity model, which accounts for the fine layering of the carboniferous strata. Most of the events took place about 50 m above the working Luise 76-10. Their locations correlated with the advancing working face, and the number of events increased when pillar edges were undercut. The source mechanism, i.e., the orientation of the fracture plane and the direction of displacement on this plane, could be determined for several seismic events. The majority of them was characterized by a fracture plane parallel to the working face and an almost vertical dislocation on this plane towards the goaf: a mechanism resembling the breaking down of the hanging wall. The fracture plane had a dimension of about 100 m. Probably the fracturing of a sandstone layer, located approximately 40 m above seam Luise, was responsible for the observed seismicity.

INTRODUCTION

Seismic events are a worldwide observed concomitant phenomenon of mining. In the Ruhr District (Germany) these events are recorded since the early 20th century. Nowadays, thousands of events with local magnitudes up to 3 are being recorded each year.

Although the knowledge that mining may induce seismicity is known for a long time, even today the exact relation between mining and seismic events is unknown. In order to determine locations and source mechanisms of seismic events, a network of three underground and five surface based seismic stations was installed at a coal mine in the east part of the Ruhr District. This network was in operation from April 1994 to May 1996.

LOCATION

A seismic event is a procedure of fracturing taking place in a confined area. The fracturing does not occur instantly in a whole area but starts at a single point. This point is called location of the seismic event or in seismological terms, the hypocenter. It is the point where the emission of seismic energy starts.

The location of events is computed by means of the travel time differences of seismic waves recorded at the seismic stations. These travel time differences depend, apart from event location and station position, on the seismic velocities of the underground.

Anisotropic velocity model

It was found that the velocity model used for the Ruhr District so far resulted in erroneous locations. Especially the determined depths of the events were unrealistic, located several 100 m below the current workings (1). The reason was the neglect of the fine cyclical layering of the carboniferous strata. Since the period of a cycle is, with approx. 10 m, much smaller than the wavelength of the recorded elastic waves (more than 100 m), the medium acts like an anisotropic medium with hexagonal symmetry (often also called transverse isotropic medium) (2). Figure 1 shows the seismic velocity in the carboniferous Ruhr District as a function of the propagation direction. Shown are the velocities for the three different types of elastic waves which propagate through a transverse isotropic medium. The shear-waves with vertical (qSV) and horizontal (SH) polarization respectively and the compressional qP-wave.

Location of the events

In the time from April 1994 to May 1996 more than 1000 events were recorded. 50 of them, which were recorded at three or more seismic stations, could be located. Figure 2 shows the

event locations together with the location of the seismic stations and the current mining operations of the mine Heinrich Robert.

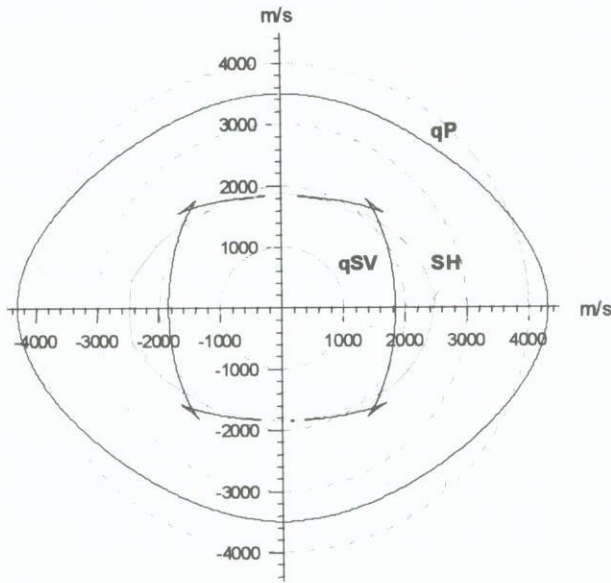


Figure 1: Velocity of seismic waves as a function of their propagation direction.

Using the anisotropic velocity model, the events were located in a depth of about -900 m. The location errors due to uncertainties in the velocity model and errors in the determined onset times were estimated at about ± 150 m in the horizontal and about ± 200 m in the vertical direction.

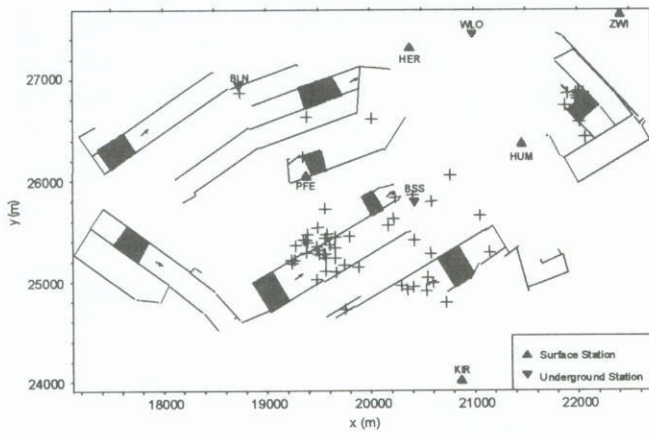


Figure 2: Location of events

Most of the events took place in the vicinity of longwall Luise 76-10, which was executed in a depth of -950 m. The localization accuracy was not sufficient to decide whether the locations of the events were directly dependent on the advancing working face of Luise 76-10, or whether the events represent a more global response of the rockmass surrounding the working. In the following, this paper will be confined to the events located in the vicinity of Luise 76-10.

Using S to P onset differences

The working face of Luise 76-10 was moving towards the underground station BSS. At this station several events exhibited strong and undisturbed S and P onsets. Although the amplitudes of these events varied over several decades, their waveform was remarkable similar. It was therefore possible to accurately determine the S to P onset differences ($t_s - t_p$) by comparing the recordings. As an example, Figure 3 shows the recordings of two events. It is evident that although the waveforms of the events are very similar, the S to P onset difference of the first event (04.11.94) is greater than the difference of the second one (01.12.94).

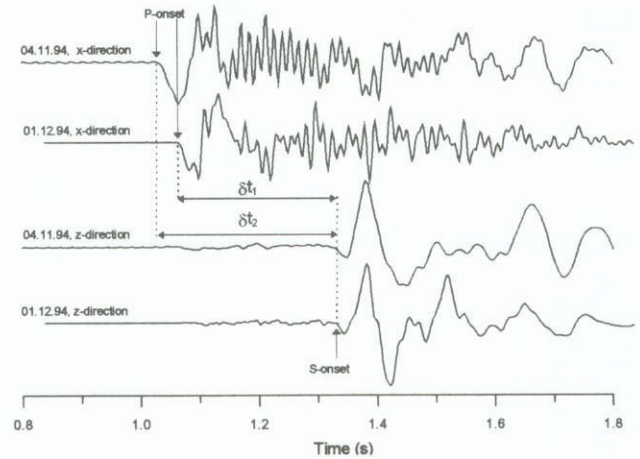


Figure 3: Determining the S to P onset differences of similar recordings.

With the assumption of constant S and P wave velocities (v_s and v_p , respectively) between the station BSS and the event location, the distance s of an event from BSS can be directly calculated via

$$s = (t_s - t_p) \frac{v_p v_s}{v_p - v_s} \quad (1)$$

Figure 4 shows the distances of the events together with the distance of the working face of Luise 76-10 from BSS versus time. The Figure demonstrates the strong correlation between face position and event location and, hence, the direct connection between the occurrence of events and the work in Luise 76-10. Due to the great accuracy in the determination of the onset differences, the error in calculating the relative distances between single events is very small (about ± 16 m). The absolute distance between an event and BSS, however, depends decisively on the used velocities. It is therefore not clear whether the events are located in front of or behind the working face.

Relation between mining parameters and events

The reason why the working Luise 76-10 was accompanied by a relatively high number of seismic events, unlike other



workings being worked on the same time, is not clear up to now.

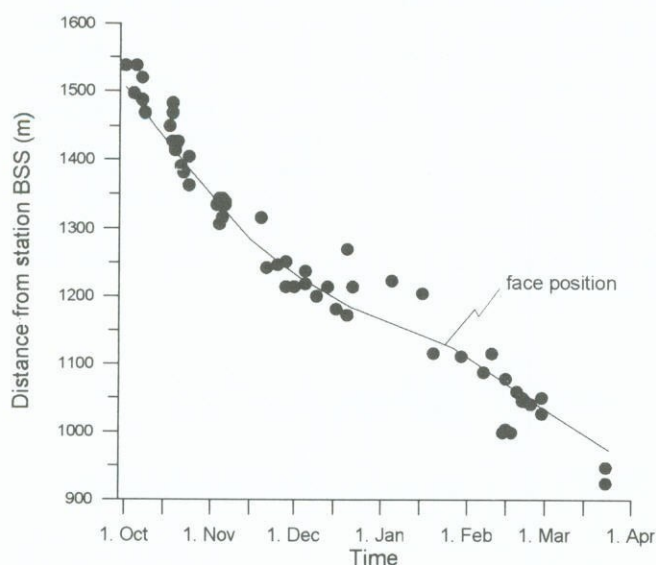


Figure 4: Relation between face position and seismic events.

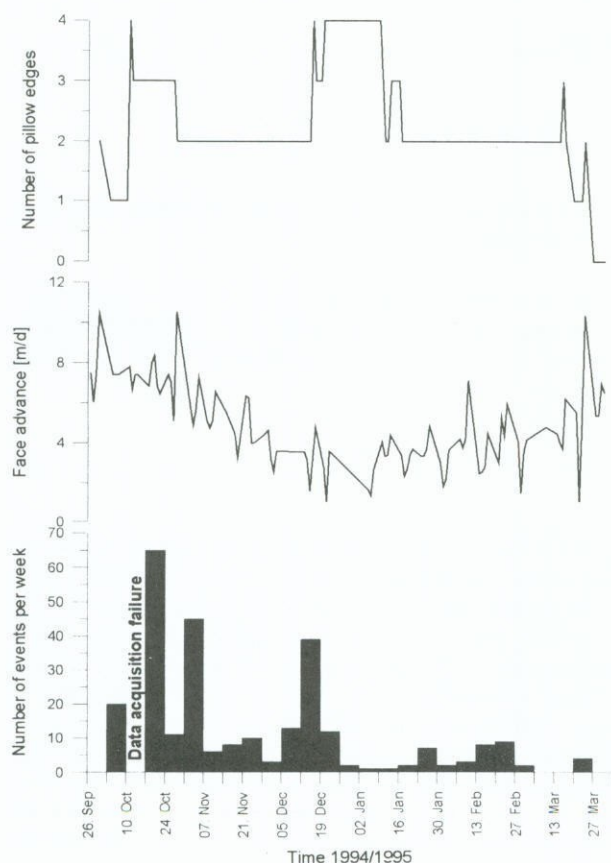


Figure 5: Comparison of mining parameters with the number of recorded events.

It is presently investigated whether the events around Luise 76-10 are dependent on geological parameters, such as seam thickness or stratigraphy, and/or on parameters of the working

geometry, such as pillar edges, face width, and the direction of mining. Figure 5 shows, as a first result, a comparison between the number of recorded events, the number of pillar edges, and the rate of face advance.

The pillar edges are caused by previous workings in seams located above Luise (e.g. Johann and Helene, cf. Figure 12). Both the increase of events in the middle of October and the increase in the middle of December came along with a greater number of undercut pillar edges.

There is no evident correlation between the number of recorded events and the rate of face advance.

FAULT PLANE SOLUTION

The radiation of seismic waves from mining induced events is in general explainable by shear failures on a fault plane (3). The determination of the orientation of a fault plane and the direction of slip motion on it is called fault plane solution.

The radiation pattern of the shear failure (considered as a point source) is mathematically equivalent to the pattern radiated from two crossing force dipoles, referred as double couple. The space around such a source can be divided into four quadrants with respect to the sense of first motion (the polarization) of the P waves. Two quadrants are compressional and two quadrants are dilatational. Figure 6 shows a double couple and the corresponding radiation pattern of P waves which also corresponds to two possible fault planes: One plane with a normal parallel to one couple and a direction of slip motion parallel to the other couple, and the other plane with normal and direction of slip motion interchanged. It is consequently not possible to distinguish between these two planes from the observed radiation alone.

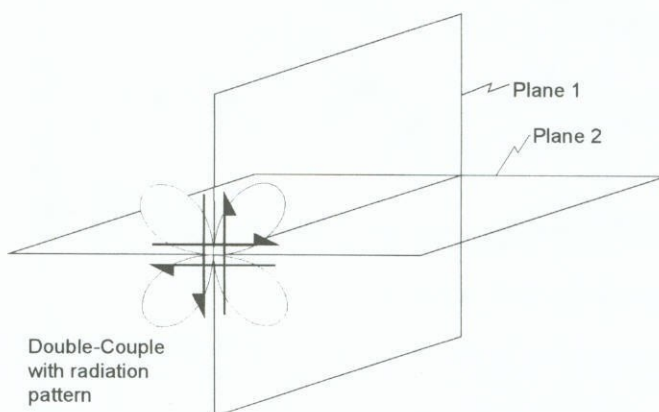


Figure 6: Equivalence between a double couple and a shear failure. In the dark areas of the displayed radiation pattern the P wave exhibits a positive polarization.

The classical fault plane solution uses the polarization of P waves, by comparing the sense of first motions on different recordings of one event. This method, however, requires much



more than four or five recordings, which is the limit in our case, to determine the orientation of the fault plane and the direction of slip motion. For this reason we used, in addition to the polarization of the P wave, the SV to P amplitude ratio, and the polarization angle of the S wave. The search of possible fault plane solutions was carried out systematically by varying the angles of fault orientation and slip in discrete steps of 5° . The values of P polarization, SV to P amplitude ratio, and S polarization were then calculated for each station. A fault plane solution was accepted as possible, when calculated and recorded values were equal within predefined limits. Figure 7 shows the possible fault plane solutions for one event. The two solutions – which can not be distinguished from radiation alone – are characterized by a steep plane (medium dip approx. 80°) and an almost flat plane (medium dip approx. 10°), respectively. Laboratory fracture experiments, which modeled the exploitation of a coal seam, exhibited fractures with steep planes above the seam. It is therefore probable that the solutions with the steep planes correspond to the actual orientations of the fault plane. These solutions will in the following be designated as the fault plane solutions.

It was possible to determine the fault plane solution of 20 events. Except for two, all events could be subdivided into two types. Type 1 is a shear fracture on a steep plane with downthrowing character (cf. Figure 7). Type 2 is also characterized by a steep plane, but the direction of slip motion is in the opposite direction. The orientation of both the fault plane of type 1 and the plane of type 2 are approximately parallel to the working face of Luise 76-10.

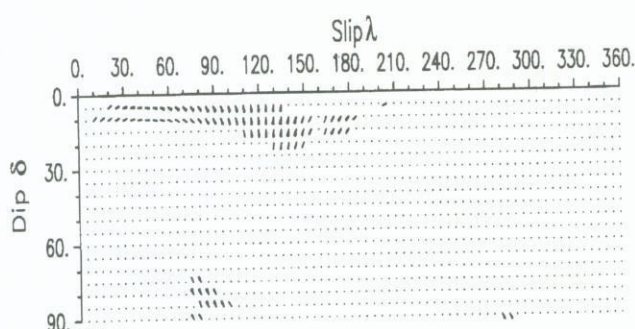


Figure 7: Possible fault plane solutions of one event. The small strokes indicate the solutions in the dip-slip coordinate system. Their orientation corresponds to the direction of strike.

Possible explanations of the source mechanisms

The mechanism of type 1, with the fault plane parallel to the working face and an almost vertical dislocation on this plane towards the goaf, resembles the mechanism of the breaking down of the hanging wall. How the mechanism of the events of type 2 can be explained is presently not clear. One possible explanation is that the events are located behind the working face and are caused by tensions remained after the working face has passed. Another is, that they are located below the coal

seam and are caused by relaxations towards the abandoned working (cf. Figure 8).

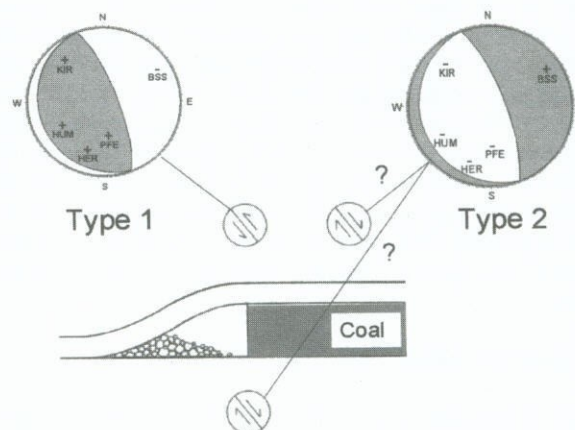


Figure 8: Possible explanations of the observed source mechanisms. Also shown are the observed P wave polarization in an equal-area projection.

With the knowledge of the source mechanism the P and T axes, i.e., the directions of compression and tension, can be calculated. Figure 9 shows the position of these axes together with the direction of face advance for the events of type 1. If these events are located in front of the working face, then the direction of highest compression is located before and the direction of tension behind the face. This result is in accordance with the known stress distribution around a working face. The distribution of the P and T axes for the events of type 2 looks similar to Figure 9 with P and T axes interchanged.

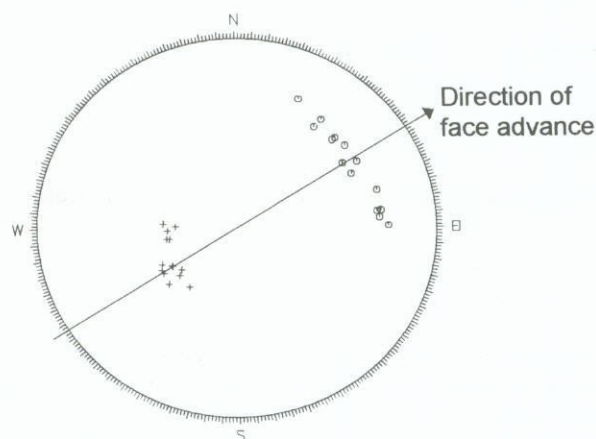


Figure 9: Equal-area projection of the axes of compression (o) and tension (+).

SOURCE PARAMETERS

We have considered a seismic event as a point source and have neglected the dynamics of the fracture and its finite extent so far. A simple model which describes the dynamics of a fracture is the dislocation model of Madariaga (4). Here the

seismic source is represented by a circular fracture plane. The dislocation starts at one point (at the center of the circle) where the static shear strength is exceeded. It extends with a constant rupture velocity until the governing stresses are falling below the dynamic shear strength. Attempts to apply the source model of Madariaga to seismic events induced by mining have shown that it can be used successfully to interpret some characteristics of the recorded seismic waves. This interpretation is normally done in the frequency domain. Figure 10 shows, as an example, the spectrum of a typical seismic event recorded at the underground station BSS. The spectrum is relatively constant at low frequencies and falls off rapidly after the so-called corner frequency f_c .

Seismic moment, source radius and stress drop

The low-frequency level is directly related to the seismic moment M_0 which is defined as the product of shear strength μ , the area of the shear fault A , and the average dislocation on it $\bar{u}_0$:

$$M_0 = \mu A \bar{u}_0. \quad (2)$$

The corner frequency f_c is related to the radius of the circular fault r_c :

$$r_c = \frac{k v}{f_c} \quad (3)$$

where v is the velocity of the considered seismic wave (P or S wave velocity) and k is a constant depending on the source mechanism.

Whereas the estimation of the source size is strongly model-dependent, the calculated seismic moment is independent of the applied source model. It is a general measure of the strength of a seismic event.

From seismic moment and source radius the stress drop

$$\Delta\sigma = \frac{7 M_0}{16 r_c^3} \quad (4)$$

can be estimated. In equation 4 $\Delta\sigma$ represents the uniform reduction in shear stress causing the seismic slip over the circular fault. Figure 11 shows the seismic moments as a function of the source radii for events recorded at the station BSS. Also indicated in Figure 11 are lines of constant stress drop.

The seismic moment ranges from $4.6 \cdot 10^{11}$ Nm to $1.2 \cdot 10^{13}$ Nm and the source radius from 59 m to 144 m. The average displacement on the shear fault, calculated from equation (2), is approximately 20 mm. The stress drop of the events is in the range of 10^5 Pa to 10^7 Pa.

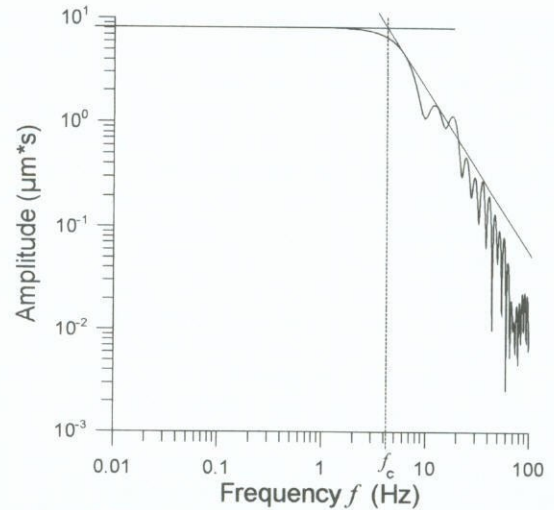


Figure 10: Spectrum of seismic events. The straight lines indicate the low and high frequency asymptotes used to determine the corner frequency f_c .

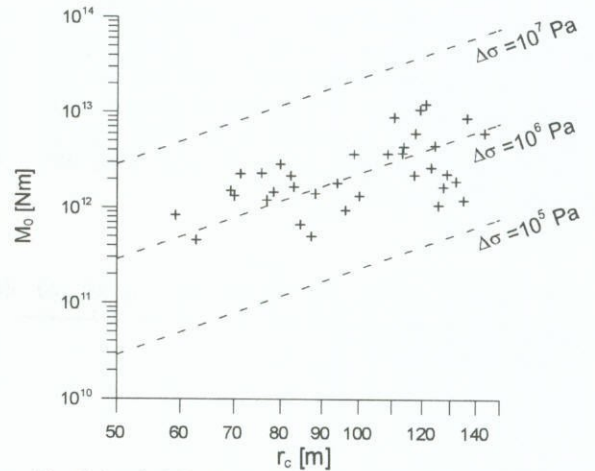


Figure 11: Seismic Moment versus radius of the fault plane. Also drawn are lines of constant stress drop.

DISCUSSION OF THE RESULTS

The locations of the seismic events occurring in the vicinity of Luise 76-10 were moving with the advancing working face. The fault planes of the events were parallel to the working face and the axes of compression and tension pointed in the direction of face advance. These events were therefore directly connected with the work in Luise 76-10. At least the events of type 1 are located above Luise 76-10. The number of events increased when pillow edges were undercut. A possible explanation of these events is the fracture of the sandstone layer located approximately 50 m above seam Luise (cf. Figure 12).

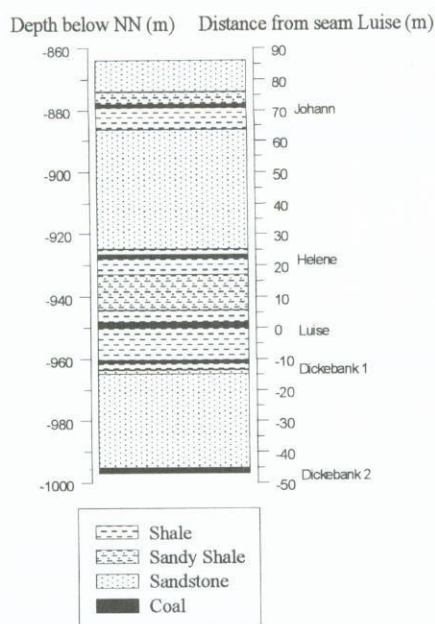


Figure 12: Geological situation around seam Luise.

This layer is situated between the seams Helene and Johann and possibly due to former work in these seams under increased pressure. Additional pressures, caused by the work in seam Luise, led to the fracture of the layer.

The calculated dimensions of the fracture planes (radii up to 144 m) are much greater than the thickness of the sandstone layer (approximately 40 m). Due to the steepness of the planes, these dimensions correspond approximately to the thickness of the breaking rock strata. In addition to the sandstone layer, other layers are therefore also fractured. These are possibly layers which are supported by the sandstone layer and thus originally intact. They fracture after losing their support. It has to be considered, however, that the source dimension was calculated with a relatively simple model, which assumed for example a circular fault and a constant rupture velocity. It is therefore possible that the dimensions of the actual fault on the one hand are smaller, and the average dislocations on the other hand are, due to the independence of the seismic moment from the applied model (cf. equation 2), bigger.

The range of 10^5 Pa to 10^7 Pa, found for the stress drop of the events, is typical not only for mining induced events but also for earthquakes in general.

Further measurements have to be done to check whether the results found for the seismic events in the area of the mine Heinrich-Robert are typical for mining induced events in the Ruhr Area. In order to reduce the number and strength of mining induced events future work should be concentrated on the question, how the generation of seismic events is connected with special geological and/or mining related conditions.

ACKNOWLEDGMENTS

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Control Technology for Roof Drill Operators

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ABSTRACT

The roof drill operator productivity can be enhanced by the introduction of control technology. The operators control station can be improved by considering human factors such as handle forces and control location. The smaller electrical controls can be located much easier than the traditional hands on valves.

The operators control can be improved by using processors with programmable subroutines that assist the operator in the decision making process. For example to increase feed rate – to slow rotation – to avoid clogged steels – etc. The overall affect is to improve drilling and bolting consistency.

The control system uses a dedicated processor packaged as an intrinsically safe control station. The roof drill operator is assisted in the drilling and bolting by a closed loop feedback system. The processor monitors the drilling parameters such as feed force, feed rate, rotation torque, RPM, drill position, and vacuum condition. Based on the drilling parameters the processor executes pre-determined condition statements intended to enhance drilling. The drill operator has an active display to alert him to the real time machine status. For example if the feed were to slow or delay due to a high vacuum level the display would show "high vacuum".

The dedicated processor is available on the arm feed style machines. The prototype machines used programmable logic controllers to develop the feedback relations. The ability to record drilling parameters was extremely helpful in this development project. The extended use of the recording feature will aid operators in mapping the mine roofs. Interpreting the recorded drilling parameters can reveal a void in the top or hard competent drilling. The prototype machines also showed the importance of vacuum monitoring. This feature is extremely valuable in softer materials.

The benefits of the control system range from efficient use of the consumable bolting materials to uniform operator performance.

Control Technology for Roof Drill Operators

The roof drill operator's productivity can be improved by the introduction of electronic control technology. The traditional valve handle control is dependent on valve size and hosing requirements in relation to the design of the operator's area. The smaller "hands on" components found in electrical control can be mounted on a roof drill in a more protected manner. The control stations can also be mounted in a more ergonomic fashion allowing improved operator position relative to the machine canopy and control levers. The electronic control station features lower handle forces and real time read outs providing the operator with drilling information. Additional safety interlocks and operational limits can be introduced in the electronic system.

The control technology extends beyond the operators control area into the drilling decision making area. The electrical control allows easy customized closed loop feedback for drilling. The feedback systems take into consideration not only feed rates and forces but rotation torque, RPM, and vacuum or water flushing levels. All of these drilling parameters are monitored and subroutines are executed to maintain pre-determined relations between the values. Maintaining these values to the subroutine instruction improves drilling consistency and bit life. The time spent changing drill bits is reduced as is the time spent clearing clogged drill steels. This keeps the machine in the actual drilling cycle and reduces the in cycle down time.

The control technology is also used in the bolt cycle to improve consistency. The control station will allow the operator to select subroutines that relate to specific resin suppliers' installation instructions.

Currently three resin types are programmed in the controller. The machine will regulate the spin time, and the setting thrust, and the hold time for the selected resin in service (Fig. 1). The resin subroutine assists the operator decision making so bolt insertion consistency is no longer dependent on the operator's perception of time. A change in

Spin Time

Complete mixing of the cartridge contents is vital to obtain the maximum anchorage strength. **Spin Time** is the amount of time needed to properly mix the cartridge components by bolt rotation. Complete mixing of the components normally requires 30-35 revolutions of the bolt.

Hold Time

After the cartridge is thoroughly mixed, the bolt must be held in place until the resin sets. The amount of time necessary after mixing stops until the bolt is firmly anchored is referred to as **Hold Time**.

Gel Time

The amount of time necessary until the resin begins to harden is referred to as **Gel Time**. Practically speaking, Gel Time is the sum of Spin and Hold Times, and is inversely related to temperature.

Cartridge Speed	Spin Time (Seconds)	Hold Time
50	5 - 8	20
35	5 - 8	10
20	3 - 6	3 - 6

Note: Spin and Hold Times are dependent upon bolt rotation speed and cartridge, coil and rock temperature. Field trials are recommended.

Standard Applications	Super Fast (20)	Fast (35, 30)	Medium (45, 50)	Slow (60)	Lokset PA	Resin Cup
Fully Grouted Repair	X	X	X			
Tension Resin: Come Nuts/Shear Pins	X	X				
Tension Resin: Two-Speed Resin Combination	X	X	X	X		
Resin-Enhanced Mechanical Anchor					X	
Resin-Enhanced Mechanical Anchor on Rebar					X	
Competition Resin Rebar/Mechanics	X	X				
Special Applications						
Covering		X	X			
Steeper Installation		X	X			
Pneumatic Cartridge Insertion						X

("X" denotes proper combinations)

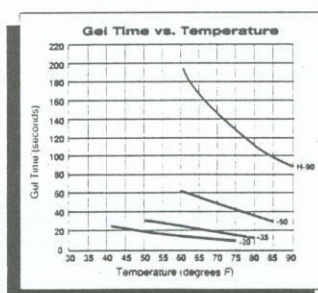


Figure 1. Resin cartridge application and installation terminology.

resin type will not require the operator to re-think his spin and hold times. The operator will respond by setting the control station to the new resin type and allowing the control to spin, thrust, and hold the bolt.

The installation of an expansion shell bolt can be regulated accurately by monitoring the torque directly by the strain gauges located in the drill head trunnion. The significance of this type of load cell is it removes the efficiency rating of gear sets, hydraulic motors, and hydraulic pressure drops from the torque measurement system.

The control system itself is made up of a dedicated microprocessor mounted in an intrinsically safe control station (Fig. 2). The control station has a proportional joystick for feed and rotation control. The joystick has side or thumb buttons that act as a feed and rotation enable. The side or thumb button has to be depressed before the feed or rotation will be active. The side or thumb button acts directly to eliminate inadvertent activation. Two additional control buttons are found on the joystick. The mechanical bolt and resin up buttons are located so the first and second finger can easily reach them. The mechanical bolt button sets the feed rate, feed force, and rotational torque to pre-selected values for the installation and torquing of expansion shell bolts. The resin up button is used with the fast feed button (located on the side of the control station) to initiate the bolt up subroutine. This will set the mix time, thrust, and hold time for the

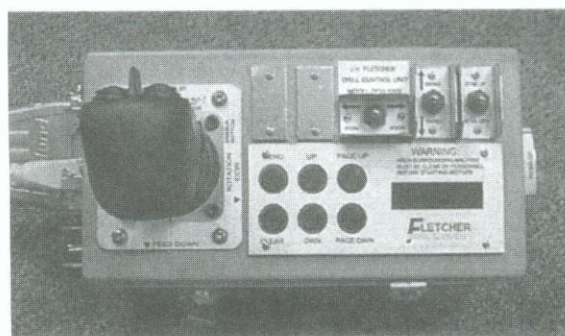


Figure 2. Feedback control station.

selected resin type. The control station also has toggle switch (on/off) control for pump start, canopy raise, boom swing, boom extend, and stab foot. The fast feed button is located on the side of the control station and is used with the joystick and resin up button to increase the feed rate beyond the usable feed rate for drilling. The six membrane style pushbuttons make up the remaining switches on the control unit. These switches are used to select the drilling parameter settings; for example, the feed back curve or the type of resin subroutine.

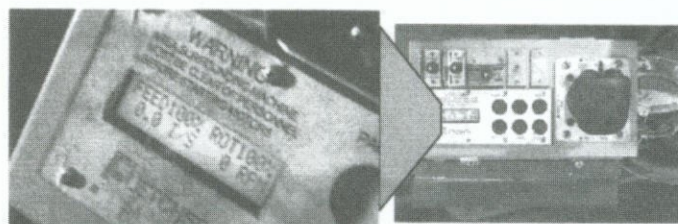


Figure 3. Feedback control station with exploded view of setting display.

The settings are viewed on the two-line display located at the side of the membrane switches. The menu button is pressed allowing the operator to scroll "page up" or "page down" to find the parameter to be changed. The parameter value is then changed by the up or down membrane switch. In addition to changing machine parameter settings the display is a real time indicator of machine status. If, for example the feed slows or stops in response to a clogging drill steel the display will read "high vacuum". If, for example the feed force and torque both drop the feed rate will go to a pre-determined feed rate with a fast RPM and the display will read "void". A number of these condition statements are included in the control subroutines to enhance drilling decision-making.

The dedicated microprocessor offers an easily expandable control due to the nature of the programmable system. Additional subroutines and custom sequences can be added to meet specific needs such as controlling the boom swing and extend to move the drill directly to the next bolt position. The system has drill boom diagnostic capability and

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can be expanded to include specific machine diagnostic features. The system currently checks to see that the sensors are in a realistic range and display which sensor is potentially at fault. The system will be expanded to display when to change the pressure filters, display the oil temperature, and oil level.

The fact that the control system monitors drilling parameters allows it to also record the parameters. The recordings take place on a removable "record key" located in the Structured Mining Systems control station. This "record key" will currently record approximately two hours of drilling information. The recording aspect of the original Service Machine Company prototype PLC was used extensively to refine the feedback relations. The recording feature can now be expanded to map the mine roof.

The improved, predictable control is based on closing control loops for the feed and rotation. The sensors input information to the controller where condition statements and subroutines act on the information to regulate the output to the control valve solenoids. Knowing how much to change a value is obviously critical in improving a control system. The closed loop system allows accurate control signal modification. Reference has been made to various sensors. The control system on the arm feed machines accepts information from a feed force/drill torque load cell (Fig. 4) located in the drill head trunnion, a rotary position/velocity transducer (Fig. 5) located on the lower end of the boom arm, an RPM counter located in the drill head, and a vacuum transducer located on the valve mount tray. With the exception of the RPM counter (which is a pulse counter) all the sensors are intrinsically safe and have a 4-20ma output. The mast feed machines use a displacement transducer mounted in the feed cylinder for drill position and velocity. The feed thrust and torque sensor on the mast feed is different only in packaging. The other sensors do not change from mast feed to arm feed machines.

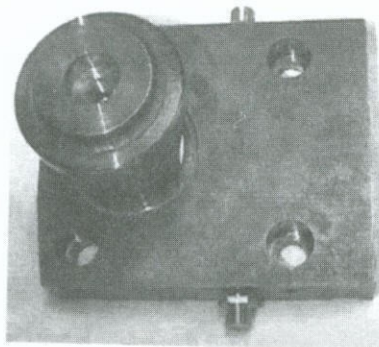


Figure 4. Thrust and torque sensor.

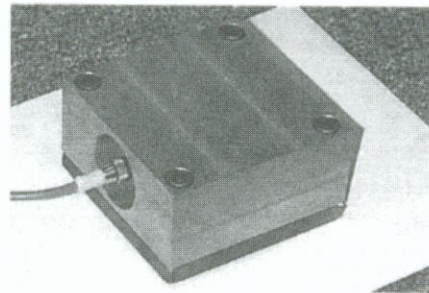


Figure 5. Position and velocity sensor.

The use of intrinsically safe components reduces change out time of any sensor or connecting cable (Fig. 6). The sensors are connected to the control station by cord sets with multi-pin screw on connections.

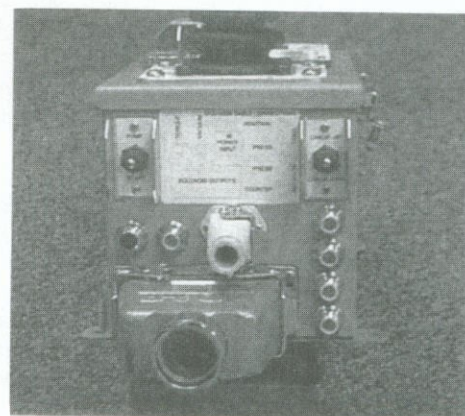
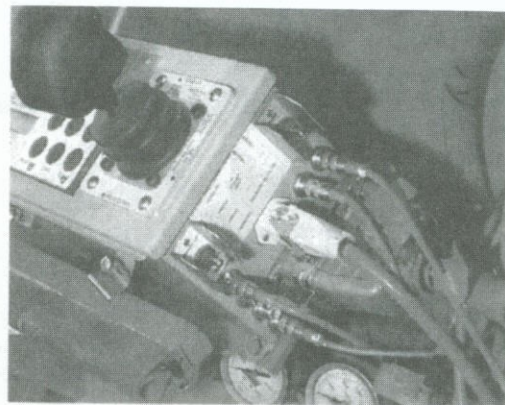


Figure 6. View of cable connector ports.

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The control stations input power is provided by a multi-pin plug and locking socket cable. This cable connects the control station to an I.S. power supply located in the explosion proof starter box (Fig. 7).

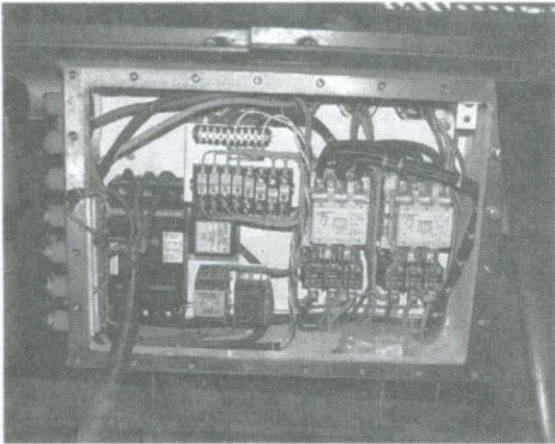


Figure 7. IS power supply in an explosion proof starter box.

Additional conductors in this cable connect to the I.S. start, start relay in the starter box. This one cable per boom is all that is required to run back to the existing X/P starter box. The output cable to the control valve solenoids is also connected to the control station by a multi-pin plug and locking socket connector. The solenoids are in turn connected by "O"volumen connectors to a molded junction block. In short, all cable connections can be replaced without the need of tools. The benefits of the intrinsically safe approach are apparent in the size of the control station and in the ease that it can be replaced if damaged.

The feedback system was first prototyped on a single head remote machine with limited automation (Fig 8). This particular machine had carousels to hold eight resin tubes and eight bolt assemblies.

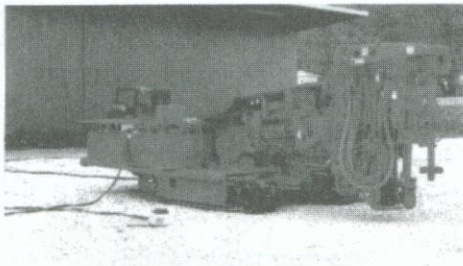


Figure 8. Single head remote prototype machine.

The operator used a multi conductor pendant to control the drill module. The next application was on a multi head machine where two operators simultaneously control four drill heads (Fig 9).



Figure 9. Four head prototype machine.

Both projects used programmable logic controllers and were instrumental in developing not only the feedback but safety systems. Both projects used mast feed drills each with two drill guides. The operator was located close to but not at a convenient reach to the drill heads and ran the drills remotely thus reducing exposure to industrial hazards.

The programmable logic controller worked well in these applications allowing various on site changes in the latter logic with a laptop. The PLC and laptop allowed the drilling parameters to be recorded (Fig 10) and reviewed to fine tune the feedback. The main disadvantage of the PLC is its physical size and number of wire terminations. The intrinsically safe system eliminates wire connections and reduces the component size.

Data Table Report			PLC-5/30		Addr 0		Thu Jul 9, 1998 Page 1			
							Data Table File N57:0			
Address	0	1	2	3	4	5	6	7	8	9
N57:0	571	298	400	937	626	660	9	1	1	0
N57:10	543	320	400	932	627	660	9	1	1	0
N57:20	733	340	400	913	638	660	9	1	1	0
N57:30	1210	384	400	1105	631	660	9	1	1	0
N57:40	1482	372	363	1267	633	660	9	1	1	0

Fig. 10 Sample information from drill parameter recorder

The main disadvantage of the PLC is its physical size and number of wire terminations. The intrinsically safe system eliminates wire connections and reduces the component size.

The vacuum sensor was also tested on these prototype machines. The vacuum sensor helped reduce the number of clogged steels in the sandstone drilling but was only required at high feed rates. In trona, however, the vacuum sensor allowed reliable drilling with a vacuum dust collection system in an application dominated by wet drilling. The vacuum feedback reduces the down time required to correct clogged steels and reduces damaged drill steels as well as drill bits.

Early in the project it became apparent the recording feature had benefits beyond fine tuning the feedback system.

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The recorded information could reveal voids in the mine top or hard competent drilling. The interpretation of available drilling information is now a project of its own. J.H. Fletcher will be working with The Department of Mining Engineering at West Virginia University on the new project. The goal is to relate the recorded drill information to the actual strata being drilled. This feature can reduce the cost of consumable materials in mines that are using bolts and resin in a longer length than required to provide sound bolt anchorage. The recorded information would indicate trends in strata change with the operator noting the immediate change. The feature can also alert the operator to increase the bolt length due to changing mine strata. The recording feature could allow study of the operator's productivity relative to changing seam height and relative to hours worked. The steps leading to an accident would also be recorded and available for review.

The drilling time is not a major portion of the cycle time in roof control drills. The benefits of the control system will keep the roof drill in productive uses with less in cycle delays. The closed loop feed and rotation control will match feed rates to mine strata while maintaining the proper drill RPM. This feature alone should reduce drill steel and drill bit costs. The addition of the vacuum feedback will further reduce drill steel and drill bit cost. At the same time as the steel and bit consumption is being reduced the time required to correct clogged steels and to change worn drill bits is reduced. The result to the drill operator is a reduction in fatigue and a more productive day. The result to the machine is a consistent level of operation with fewer overload conditions and improved component life.

The machine has completed the in plant testing. The machine has drilled blocks of concrete containing limestone, sandstone, and basketballs (Figs. 11, 12, and 13). The mixture of materials demonstrated the control systems ability to adjust to changing drilling conditions. After completing these tests the machine drilled in "blue sandstone" from Stoney River Mine at Mount Storm WV. This test proved the control systems ability to match the thrust and RPM to harder drilling conditions.

The in plant testing also allowed the operator's comfort and ease of control to be observed while drilling. All of the in plant testing can only partially prepare a new system for the mine test. The first stop for the RR2000 was ELK RUN's Black Knight Mine (Fig. 14).

The mine management provided three experienced drill operators for our test (Fig. 15).



Figure 11. Test drilling into sandstone.

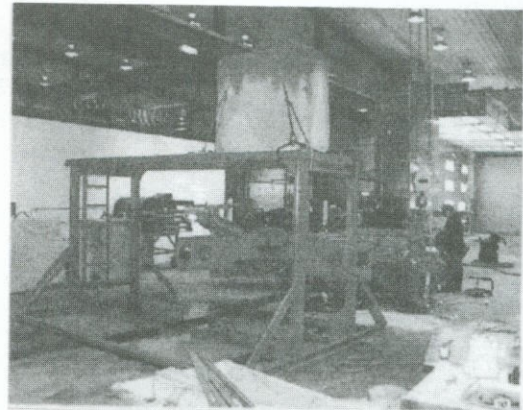


Figure 12. Test drilling into concrete.

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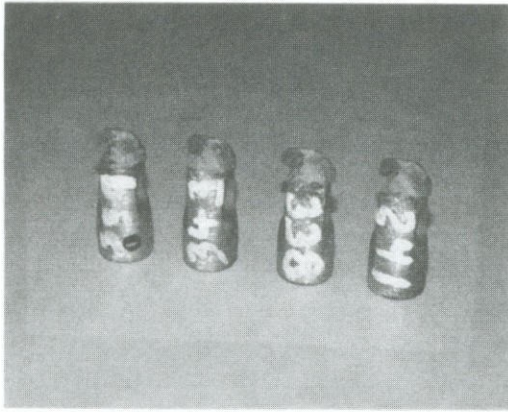


Figure 13. Test bits with data file numbers.

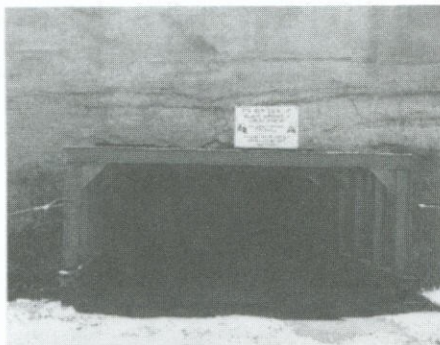


Figure 14. Black Knight Mine portal.



Figure 15. In-mine testing at Black Knight Mine.

The control system met with very favorable test results and comments from the drill operators. The top conditions were fairly constant with little hard drilling however several voids were detected. The conditions were not so severe as to show an extreme improvement in bit life but the operators comments on comfort and ease of control were rewarding. The machine has three other prototype stops before being released for production.



Resin Annulus Size Effects on Rebar Bolt Pull Strength and Resin Loss to Fractured Rock

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Abstract

One of the critical variables affecting resin bolt performance is the size of the resin annulus, defined as the difference between the hole and bolt radius. The vast majority of the millions of resin roof bolts installed each year in the United States employ an annulus between 0.125 and 0.188 inch. As the annulus size decreases the resin flow around the bolt becomes more difficult, increasing the resin pressure induced by bolt insertion. As the resin pressure increases the loss of resin into bedding planes and vertical fractures in the rock increases. As the annulus size increases the mixing of the resin paste and mastic becomes less efficient. Thus, the lower and upper limits of the optimum annulus size is determined by insertion pressure and mixing considerations, respectively. Previous U.S. Bureau of Mines researchers have concluded that a 0.125 inch annulus is optimal. However, others have recently reported that a 0.095 inch annulus with special rolled deformation (SRD) bolts provide superior anchorage to standard rebar with a very large 0.250 inch annulus, attributing SRD resin loss in weak ground to oversized holes.

The effect of annulus size on unit length pull strength and resin loss to fractured rock was investigated. Short encapsulation pull tests with resin annuluses of 0.125 and 0.095 inch resisted an average of 2.25 tons per grouted inch, with no significant variation. However, annulus size has shown a dramatic effect on the magnitude of resin lost to simulated rock fractures. No. 5 standard, No. 6 standard, and No. 7 J-Bar rebar were inserted into two different viscosity resin cartridges designed for 4 foot bolt encapsulation into a one inch inside diameter metal tube. To simulate a rock fracture, a 0.250 inch diameter orifice was located near the capped end of the tube. Resin losses increased as annulus size decreased, with a minimum loss of 35% and a maximum loss of 60% associated with 0.125 and 0.095 inch annuluses, respectively.

Introduction

Fosroc is the world's leading producer of polyester resin cartridges for rock bolt anchoring, with production facilities in the United States, Australia, Poland, South Africa, and Brazil. We manufacture cartridges to suit the performance and cost requirements of the local customers. Fosroc introduced polyester resin cartridges to the United States market over twenty-five years ago. Production facilities in Kentucky, West Virginia, and Colorado, combined with four company owned warehouses enable Fosroc USA's timely and efficient service throughout the United States and North America. Fosroc's product development and marketing is customer driven and inclusive of mine regulators and researchers. By listening to our customers and reacting to their needs, we have continuously improved the polyester resin cartridge performance. Fosroc considers itself to be the most technically competent resin producer, with significant personnel assets in both cartridge research development and ground control for underground mining and tunneling.

The effectiveness of resin anchored roof bolting is dependent upon a complex interaction of many variables. A quality polyester resin provides approximately 2 tons per grouted inch of resistance to pull. However, the cartridge cannot be viewed in isolation. The most effective evaluation methodology is in situ performance of the bolt and cartridge as installed in a specific size hole in a specific rock type. To this end the most valid test of field performance is pull strength and speed of set. To further complicate the issue, the pull testing of full column resin bolts is not a valid indication of resin strength, as the resin anchorage is stronger than the tensile strength of the bolt. A laboratory approximation of resin pull strength for a specific bolt, hole, and cartridge combination may be obtained through short encapsulation pull testing. While this methodology does not measure bolt strength or take geologic variation into account, it has been proven to be

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an effective indicator of resin strength.

One of the critical variables affecting resin bolt performance is the size of the resin annulus, calculated as the difference between the hole and bolt radius. The vast majority of the millions of resin roof bolts installed each year in the United States employ an annulus between 0.125 and 0.188 inch. As the annulus size decreases the resin flow around the bolt becomes more difficult, increasing the resin pressure induced by bolt insertion. As the resin pressure increases the loss of resin into bedding planes and vertical fractures in the rock increases. As the annulus size increases the mixing of the resin paste and mastic becomes less efficient. Thus, the lower and upper limits of the optimum annulus size is determined by insertion pressure and mixing considerations, respectively.

A review of prior resin annulus research found significant ill-defined areas in the literature. Specifically lacking was a direct comparison of very tight annuluses relative to the generally used standard resin annuluses. Three research efforts were found to be especially relevant:

- Research conducted by the U.S. Bureau of Mines and reported in 1987 concluded that: *"Observations made after accidental splits of the test blocks indicated that resin forced into a crack is uncured, uncatalyzed resin that will not bond broken or cracked rock together. Therefore, resin forced out into fissured rock strata from a cartridge resin bolt will probably not bond the broken strata together, and may actually precipitate splitting"* (1).
- Research conducted by the U.S. Bureau of Mines and reported in 1989 concluded that: *"Resin-grouted roof bolt integrity is directly related to annulus thickness. The results of this investigation indicate that, for a 3/4 in diameter roof bolt installed in competent rock, optimum annulus thickness is 1/8 in (1-in-diameter drill hole). As annulus thickness increases from the optimum, grout mix quality, effective grout ratio, and axial stiffness will decrease"* (2).
- Tadolini recently compared 7/8-SRD bolts installed in 1 in holes (0.095 in annulus) to No. 7 rebar installed in 1 3/8 holes (0.250 in annulus) (3). He concluded that: *"The system stiffness and shear strength of the reduced annulus bolts were larger in both the laboratory and field experiments."* He also concluded that the installation of the reduced annulus bolt: *"does require higher installation forces."* Relative to resin loss he concluded: *"The hypothesis for the resin losses experienced during field installations is that the hole diameter is becoming larger when weak roof areas are encountered."*

Upon review of the above research, Fosroc undertook an evaluation of the effect of No. 7 J-Bar (0.810 in diameter) bolts installed in 1 in holes (0.095 in annulus)

against No. 5 and No. 6 standard rebar in 1 in holes (0.125 and 0.188 in annulus). This complements the above reported research by inclusion of the annulus employed in the installation of the vast majority of polyester resin roof bolts.

Pull Tests

A series of short encapsulation pull tests in a concrete block were performed with a single resin formulation and two different bolt types (Table 1). Standard block pullout tests were conducted using four No. 6 headed rebar bolts and four 0.810 in diameter J-Bar bolts. Encapsulation was 9.0 inches in a 1.0 inch diameter hole (Fig. 1). Insertion spin was timed at five seconds. Block temperature was taken to be the ambient room air temperature of 64 degrees Fahrenheit. The cartridge and bolt temperature were both set to 55 degrees Fahrenheit by a one hour immersion in water. The cartridges used were 23 mm diameter, low viscosity, polyester resin. The first bolt was pulled ten minutes after installation and all eight were pulled in less than one hour. Figure 2 displays the very consistent maximum pull out force per grouted inch resisted by the eight individual bolts. The average maximum pull out force resisted was only 1.4% or 60 lbs higher for the J-Bar. This indicates equivalent pull strength performance by the two systems.

Resin Loss

Three different diameter roof bolts types were hydraulically inserted into 23 mm diameter, 4 foot equivalent grouted length, resin cartridges of two different viscosities (Table 2). The mine roof was simulated with a 1.5-in outside diameter, 0.25-in wall, by 48.75 in long steel tube (Fig. 3). The bolts were each cut to 53.5 inches in length to provide a 48.0-in encapsulation with 5.5 inches to compensate for fixture/plating offset on the Fosroc bore hole test machine (Fig. 4). To simulate a fracture, a hole was drilled and tapped and a 0.25-in inside diameter straight nipple installed in each tube, 12 inches from the top. A plastic container attached to the nipple caught the material extruded (resin loss) during insertion and was weighed for comparison.

Low and Standard viscosity resin was compared in the simulated crack testing. Fosroc has developed the low viscosity resin to facilitate the insertion of passive and tension cable bolts, resin enhanced mechanically bolts, and combination bolts. Figure 5 graphically displays the relative viscosity of Fosroc's resin with changes in temperature.

In an attempt to make the tests directly comparable, the time of insertion was held between 5 and 6 seconds for all the tests (Fig. 6). The force required to insert the bolt was measured for closed tube insertion tests with



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standard viscosity resin. The insertion pressure increased dramatically with reduction in annulus size (Fig. 7). Resin loses increased as annulus size decreased, with a minimum loss of 35% and a maximum loss of 60% associated with 0.125 and 0.095 inch annuluses, respectively (Fig. 8). The relative resin loss for standard and low viscosity resin was not distinguishable.

Conclusions

- The best compromise of both pullout forces and resin loss variables is achieved with a resin annulus of 0.125 in, confirming previous U.S. Bureau of Mines research.
- Direct comparison of 0.125 and 0.095 inch resin annuluses revealed no significant difference in resin pull out strength.
- The resin insertion pressure increased dramatically with reduction in annulus size.

- Resin loses increased as annulus size decreased. This is especially important, considering previous U.S. Bureau of Mines research found that resin lost to fractures is not reacted and does not assist in bolt anchorage.

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2. Ulrich, B. F., J. Wuest, and R. M. Stateham, Relationships Between Annulus Thickness and the Integrity of Resin -Grouted Roof Bolts, U.S. Bureau of Mines, RI-9253, 1989, 13 pp.
3. Tadolini, S., The Effects of Reduced Annulus in Roof Bolting Performance, 17th International Conference on Ground Control in Mining, Morgantown, WV, 1998, pp. 231-236.

Table 1. Short encapsulation pull test variables.

Bolt diameter, in	Steel grade	Rebar Type	Hole diameter, in	Annulus, in
0.810	75	J Bar	1	0.095
0.750	60	Standard	1	0.125

Table 2. Resin loss test variables.

Bolt diameter, in	Rebar Type	Annulus, in	Resin viscosity
0.810	J Bar	0.095	standard
			low
0.750	Standard No. 6 Rebar	0.125	standard
			low
0.625	Standard No. 5 Rebar	0.188	standard
			low

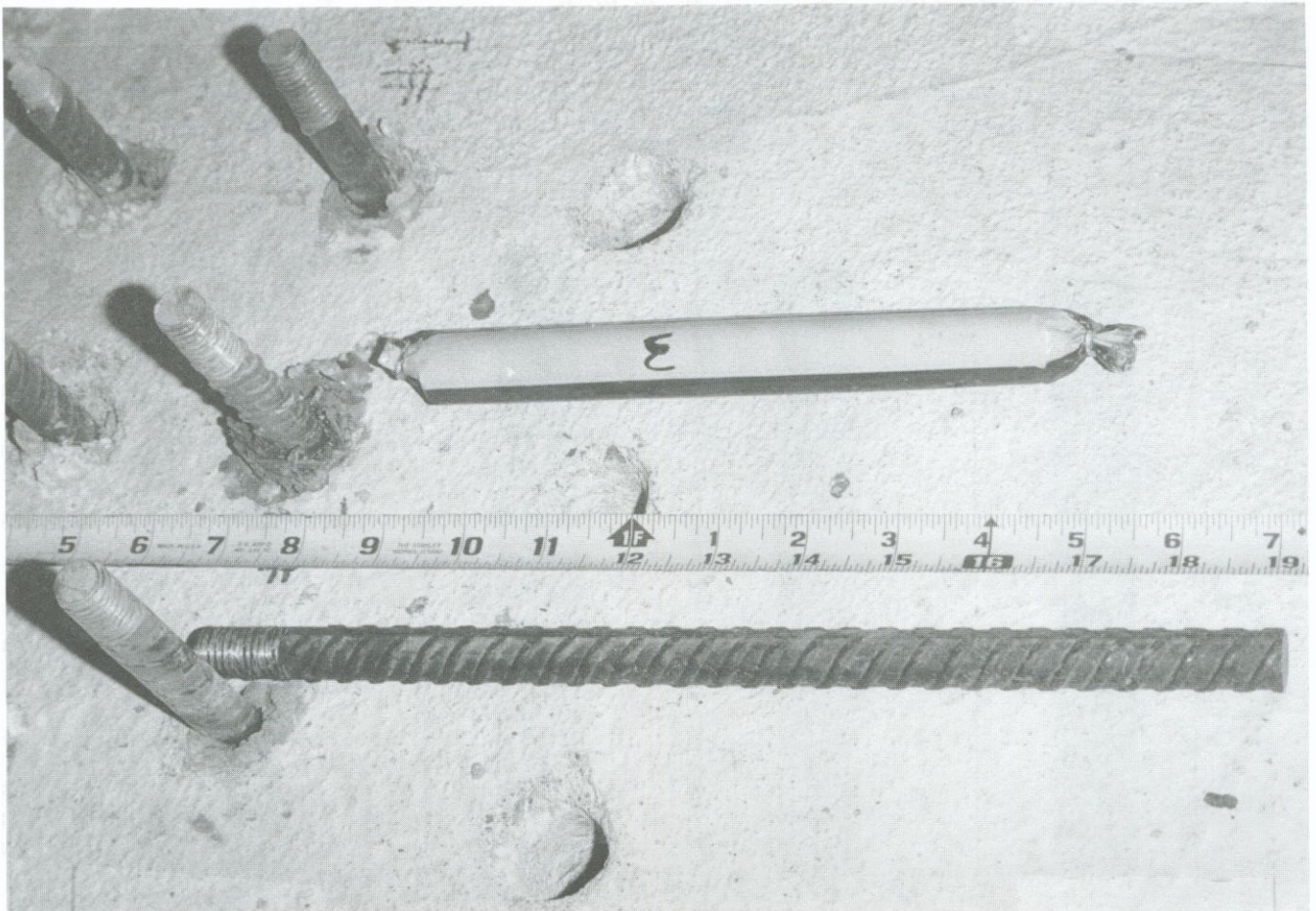


Figure 1. Block testing sample insertion.

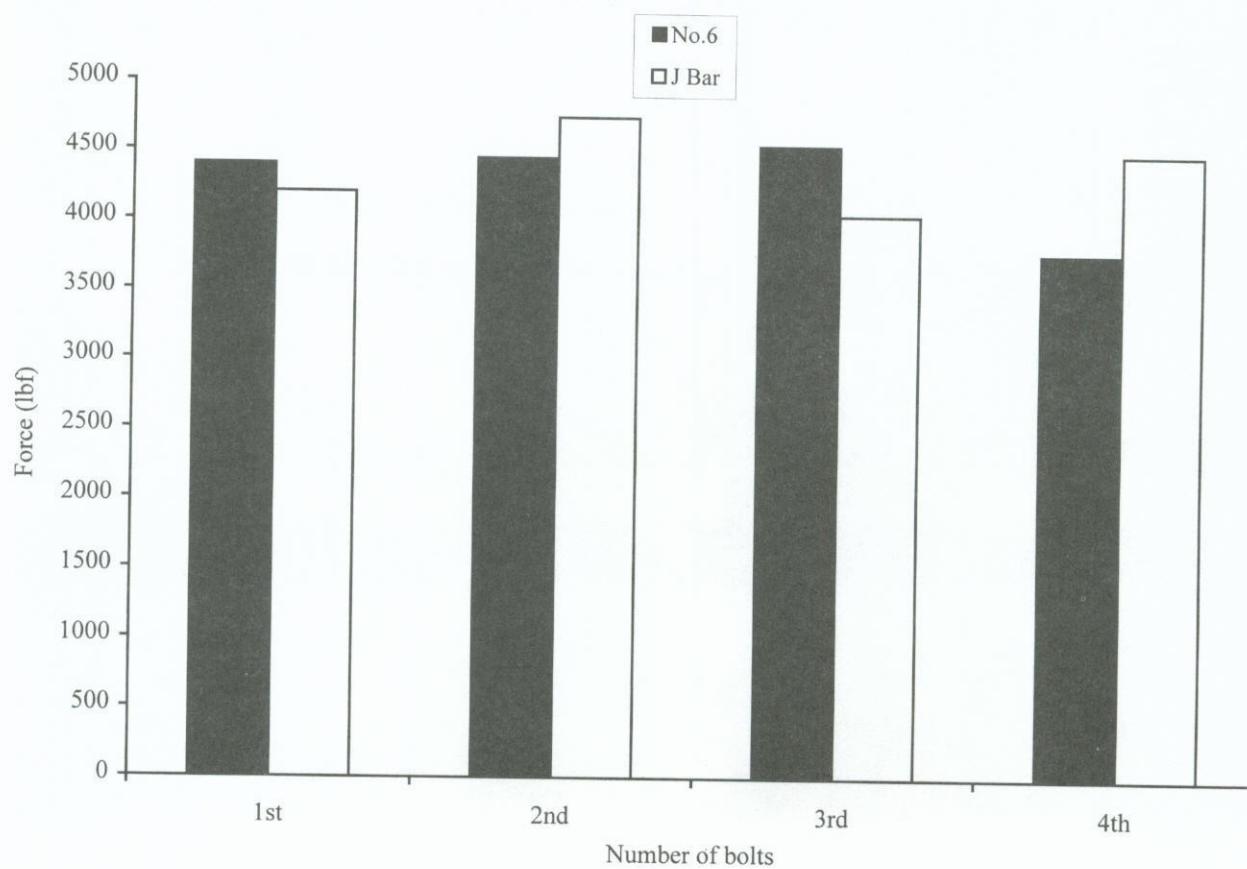


Figure 2. Resin pullout force per grouted inch for No. 6 standard rebar and No. 7 J-Bar, grouted in one inch diameter hole with standard viscosity resin.

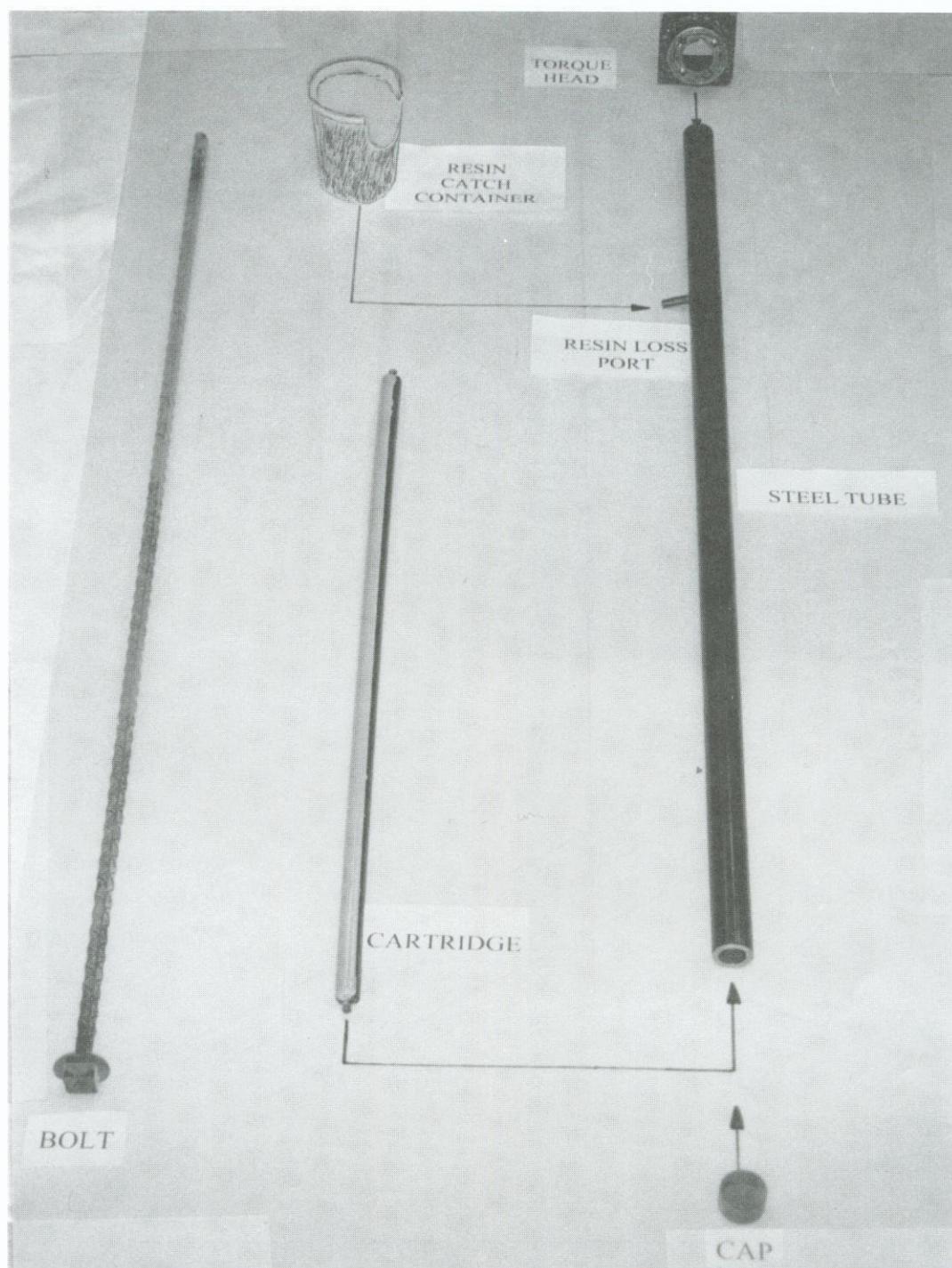


Figure 3. Sample bolt and tube assembly.

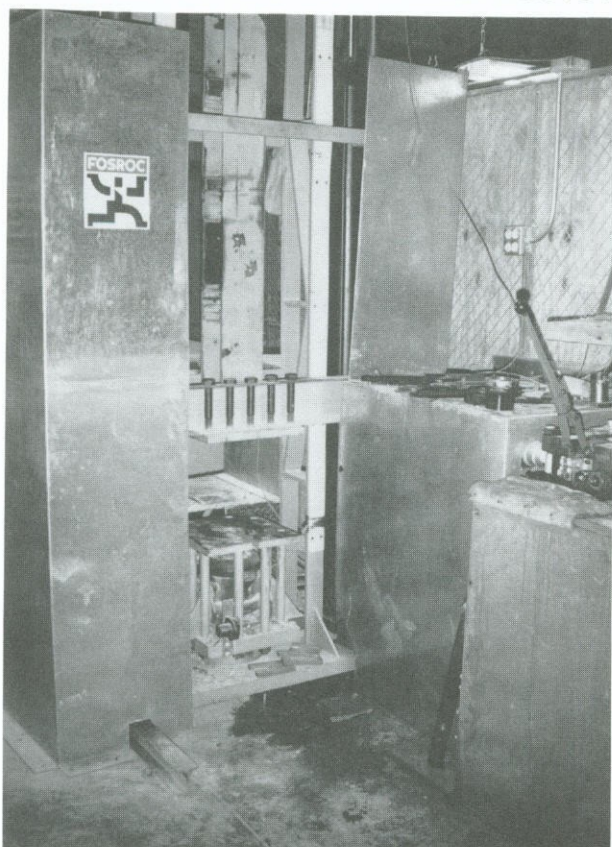


Figure 4. Fosroc's Bolt Test Machine.

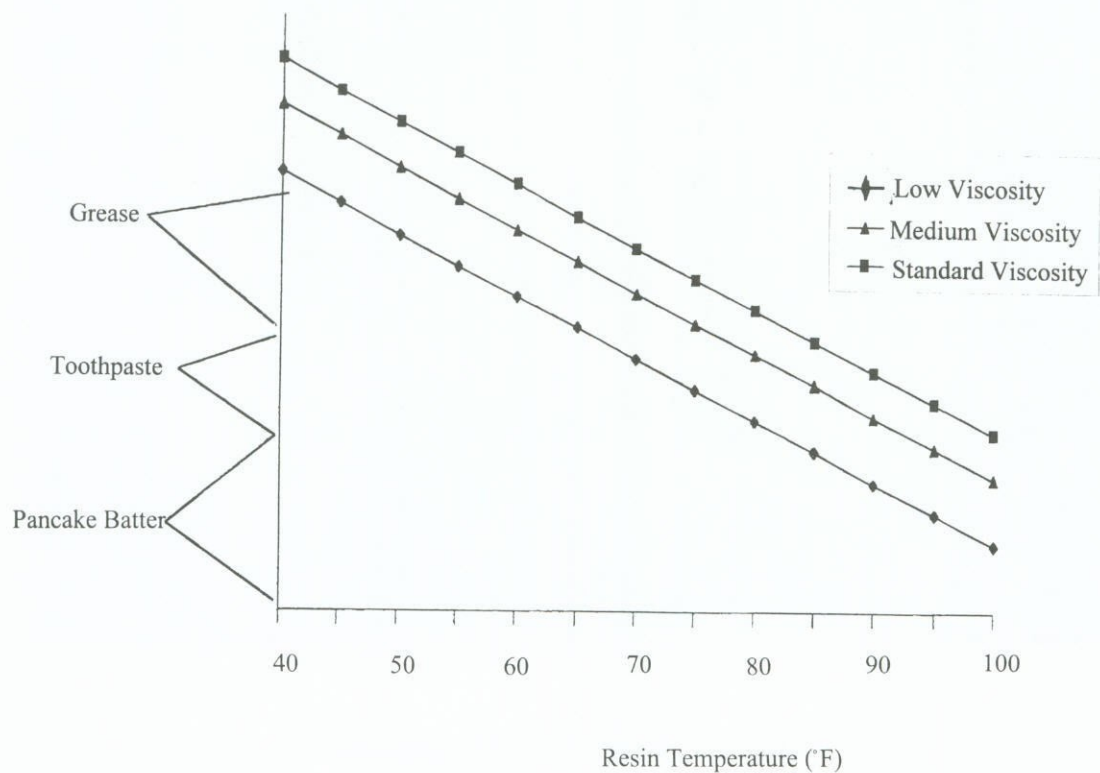


Figure 5. Effect of temperature on the viscosity of Fosroc's three resin viscosities.

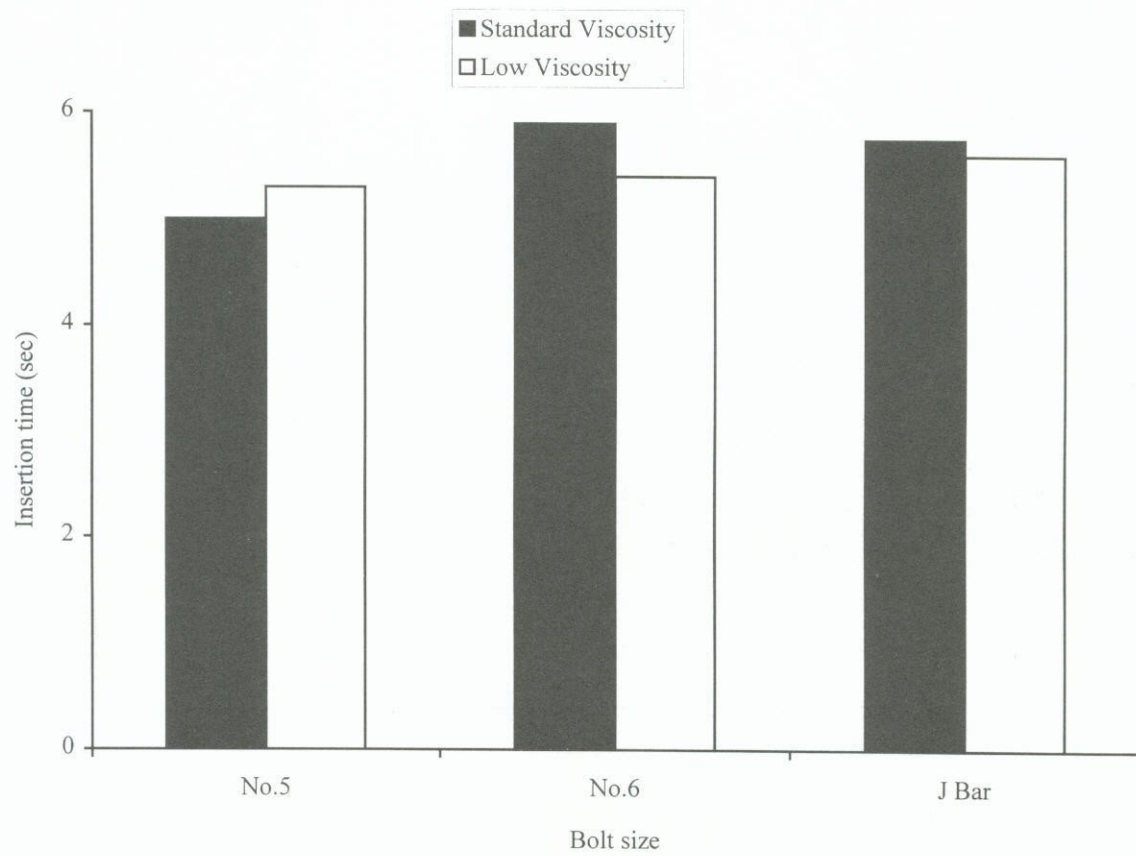


Figure 6. Average insertion time for the three bolt types.

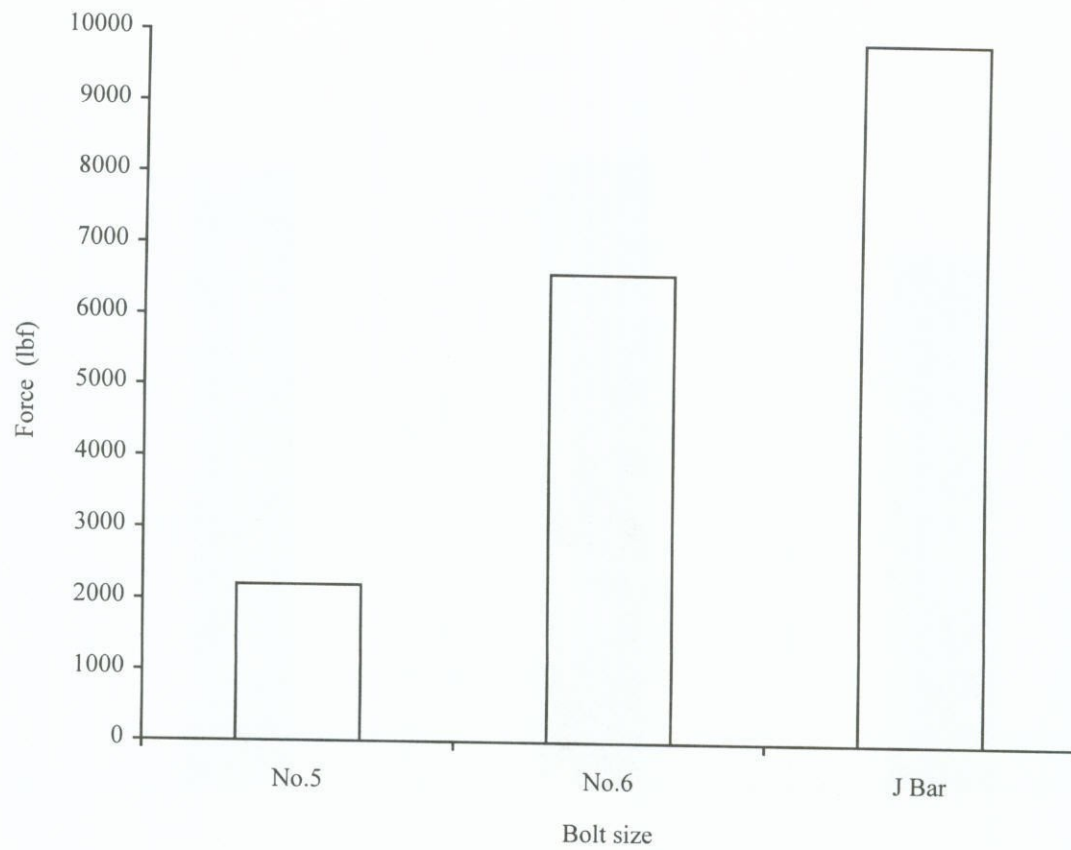


Figure 7. Closed pipe insertion force for the three bolt types with standard viscosity resin.

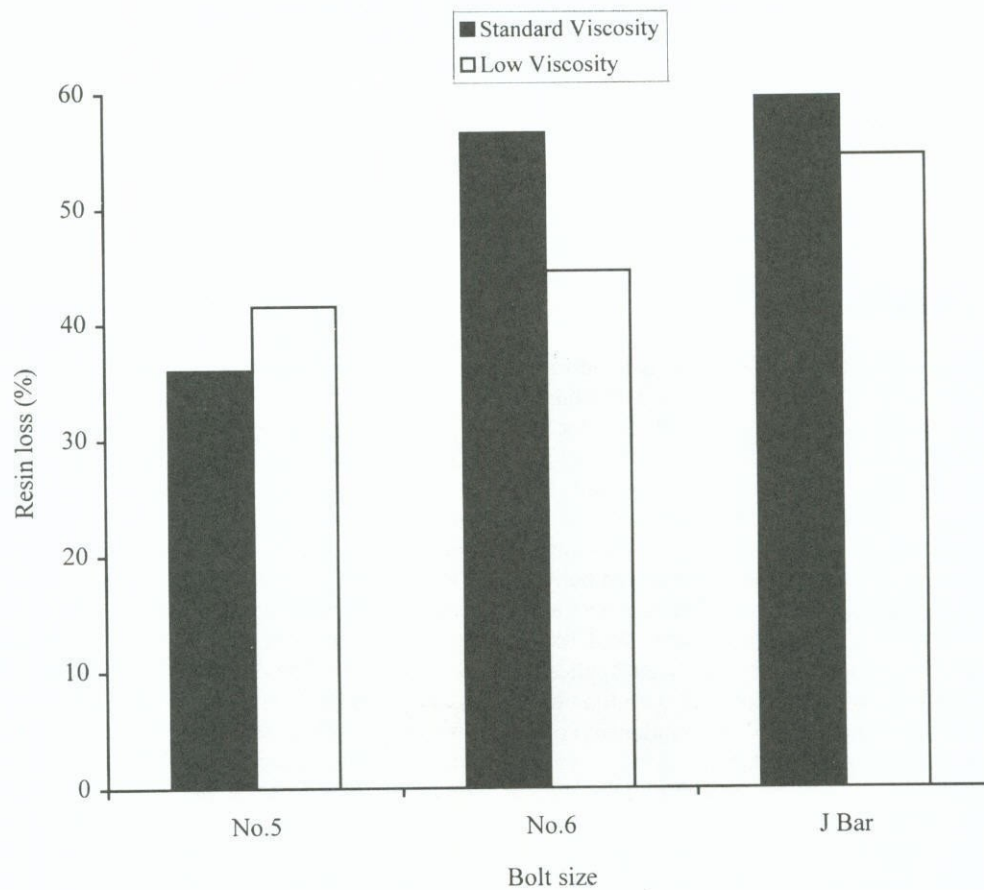


Figure 8. Resin loss for the three bolt types with standard and low viscosity resins.



Roof Bolt Response to Shear Stress: Laboratory Analysis

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ABSTRACT

Recent studies by researchers from the National Institute for Occupational Safety and Health indicate that shear loading contributes significantly to failure of bolts used for rock reinforcement in coal mine roofs. Laboratory tests on 17 bolts were conducted to study the behavior of roof bolts subjected to shear loading over a range of axial bolt loads. Fourteen strain gauges were attached to each bolt to measure axial and bending loads at seven locations along its length. The instrumented bolts were grouted through two high-strength concrete blocks, and axial tension was applied to as much as 75% of yield strength. With the block interface acting as a failure plane, shear loads were applied to the blocks at a constant rate of displacement to the ultimate strength of the bolts. The tests characterized the relationship of axial bolt loads to shear forces across a rock bedding plane and measured the distribution of axial and bending strain along the length of the bolts. These results will improve the selection of roof reinforcement in mine areas where high shear stresses are present, thus improving the safety of miners working in these areas.

INTRODUCTION

Resin-grouted roof bolts are standard elements in ground control systems at most underground coal mines in North America. In general, sagging of the mine roof and yielding of pillars result in vertical and horizontal stresses in the mine roof, imparting both axial and shear forces on roof bolts. Combined tensile and shear forces are at times sufficient to cause failure of the bolts. Mine Safety and Health Administration (MSHA) statistics show that more than 400 miners are injured in roof and rib falls annually; in the 5 years from 1994 through 1998, 53 miners died in such accidents. Twenty-two fatalities occurred in underground coal mines during 1998; of that number, 14 deaths (64%) resulted from falls of ground.

The Spokane Research Laboratory (SRL), a division of the National Institute for Occupational Safety and Health, is

conducting research on roof bolt loading to reduce support failure and subsequent ground falls. The intent of this study was to examine the effects of shear forces on rock bolts.

Early laboratory shear tests were conducted by Haas et al., (1976) using various bolt types and anchors in blocks of limestone and shale; they found that resistance to shear stress was increased about 3.7 times when fully grouted bolts were used to secure a natural fracture. Radcliffe and Stateham (1980) installed instrumented bolts in a Colorado coal mine. Each bolt had three pairs of strain gauges, one pair at each end of the bolt and one in the center. Most of the bolts exhibited bending at the center gauges, and about a third of the bolts were subjected to strain beyond the yield point of the steel. Haas (1981) applied laboratory shear test data to estimate safety factors for reinforcing a natural shear plane with grouted bolts. Turner (1987) found that appreciable opening of fractures prior to shear movements resulted in double bends and failure of rock bolts by tension. Bolts across unopened fractures had been actually cut (guillotined) during rock bursts. Pellet et al., (1996) related theoretical and experimental analyses of rock bolt shear strength and found that bolts installed perpendicular to a joint plane allowed the greatest displacement along the joint before failure, but that displacement at failure decreased rapidly as the angle between the bolt and joint plane decreased. They also found that harder rock led to bolt failure at smaller displacements. Goris et al., (1996) used essentially the same procedures as followed in this study to conduct direct-shear tests on conventional steel cable bolts. Signer and Lewis (1998) used axial loads on instrumented bolts in an underground coal mine as evidence of shear stresses that contributed to roof bolt failure.

Methods

The bolts tested were 58.4-cm-long (23-in), standard grade 60, Dwydgid No. 7 Threadbars.¹ Continuous, rolled-in threadlike de-

¹The mention of specific products or manufacturers does not imply endorsement by the National Institute for Occupational Safety and Health.



formation along the length of a bolt allows it to be tensioned to its full yield strength through the use of nuts and couplers. A 6.4-mm-wide by 3.2-mm-deep (0.25 by 0.125 in) slot was milled along the smooth sides of each bolt. After milling, the bolt's cross-sectional area was 3.47 cm² (0.5375 in²), and section modulus was 0.816 cm⁴ (0.196 in⁴). Tests have shown that the slotting process reduces axial bolt strength by approximately 10% (Signer et al., 1997, p. 184). Seven strain gauges were installed along each side of the bolt as described by Johnston and Cox (1993). The strain gauges were arrayed so that one pair of gauges was at the center and additional pairs of gauges were at 5, 10, 20 cm (2, 4, and 8 in) from the center in each direction (figure 1). The instrumented bolts were calibrated in a uniaxial test machine to correlate voltage change to load change at bolt load levels below the yield point of the steel. Voltage readings were also converted to strain according to the equation—

$$\epsilon = 4\Delta V / GF \cdot EV \quad (1)$$

where ΔV = change in voltage,
 GF = gauge factor (2.075),
 and EV = excitation voltage (5 V).

The instrumented bolts were installed in pairs of 29-cm (11.5-in) square concrete blocks made from a fine sand-cement mix having an average 28-day compressive strength of 85.5 MPa (12,400 psi). The first block of the pair was cast by pouring the mix in a steel mold with a flat aluminum bottom to create a smooth shearing surface of 780 cm² (121 in²). A 2.86-cm (1.125-in) in diameter rod was centered in the mold before casting. The rod was removed after about 24 hours to provide a hole through the block for placement of the bolt to be tested. A recess about 8.9 cm (3.5 in) square and 8.9 cm deep was also cast into the top of each block to allow room for a plate and a nut, and to permit access to the end of the bolt when it was to be tensioned. The second block of the pair was cast using the bottom surface of the previously cast block to create a matched shearing surface. This block was also cast with a centered rod and a bolt-end recess.

The instrumented bolt to be tested was positioned through the aligned holes so that the strain gauges faced one side of the block assembly. A polyester resin anchoring grout (Fosroc Anchortite) was used to assure direct contact between the bolt and the concrete block. The grout has an unconfined compressive strength of at least 69 MPa (10,000 psi) and a setting time of about 60 min. The bolt was coupled through an A-frame extension to an 18-mt (20-ton) hydraulic jack and tensioned against a plate and a nut on one end of the bolt. A through-socket and wrench were then used to tighten a nut at the other end to a specified torque to retain the desired load.

After the grout had set, the block assembly was placed in steel shear boxes and oriented so that the gauges faced in the direction of shear displacement. The shear boxes allow a 0.63-cm (0.25-in) gap between the walls of each box and the surfaces of the blocks. This annular space around the blocks was filled with quick-curing gypsum cement, which has a compressive strength of about 28 MPa (4,000 psi), to ensure complete contact with the shear boxes.

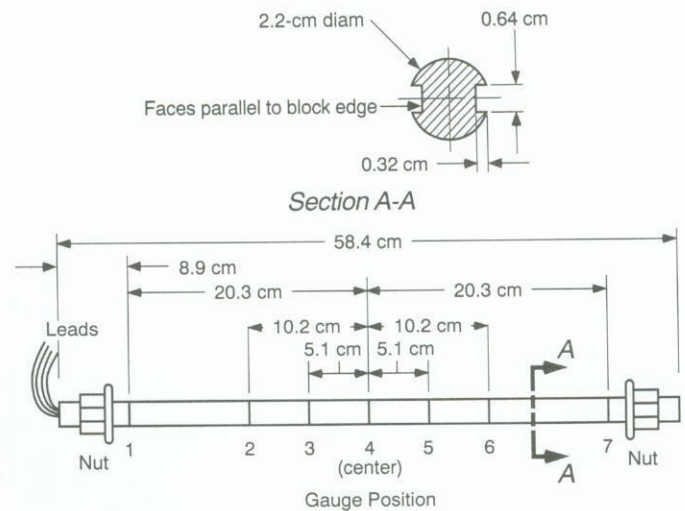


Figure 1.—Instrumented roof bolt

Testing

Tests were conducted in a Gilmore direct-shear, materials-testing machine. This high-pressure hydraulic test frame has a capacity of 1,334-kN (300,000 lbf) along both the shear and normal axes. The shear box assembly with the concrete blocks and a tensioned, instrumented bolt was placed on rollers in the test frame with the bolt aligned to the normal (vertical) axis. Figure 2 shows the shear box assembly ready for testing. An MTS TestStar II control system allowed for computer management of test parameters through templates that determined loads, displacement, and rates in both the shear and normal directions. For this series of tests, a constant load of 8.90 kN (2,000 lbf) was applied along the normal axis of the block assembly, principally to help maintain the position of the box assembly during shearing.

During the test, the upper box was held immobile against a backstop in the frame while shear loads were applied at a constant displacement rate of 0.55 mm/min (0.0216 in/min) on the lower box. With the steel shear boxes in the test frame, maximum shear displacement for the machine was 4.6 cm (1.8 in). Applied load and displacement data were collected at 3-sec intervals within the TestStar system and at 15-sec intervals with an external datalogger (Campbell Scientific Model 21X-QM). The datalogger also collected information from the array of strain gauges on the instrumented bolt.

SHEAR TEST DATA

The program was designed to test 15 instrumented bolts pretensioned at five different axial bolt loads. An initial test on an uninstrumented bolt was conducted to establish reasonable shear displacement rates, load ranges, and applied-load sampling intervals. Two tests on instrumented bolts were added to establish strain gauge sampling methods and intervals. Strain gauge data

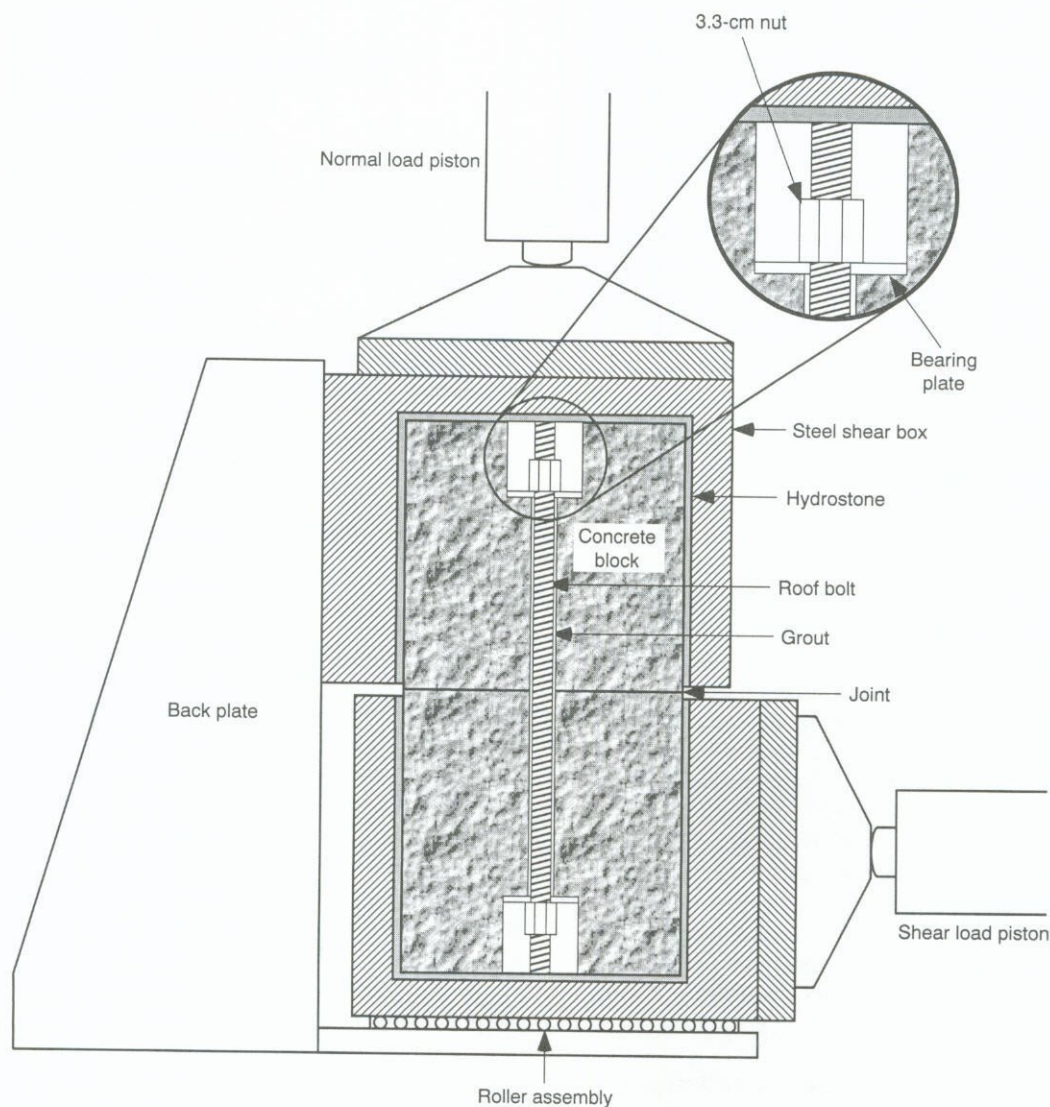


Figure 2.—Shear blocks and test assembly

from all 17 tests on instrumented bolts were considered valid and so are included here.

Calculated loads from all functioning gauges (signals from 3 of the 238 gauges failed prior to tensioning) were used to determine average pretensioned load on the bolt. Special care was required to minimize bending at the ends of the bolts where the plates and nuts secured them to the blocks. The bolts were pretensioned to axial loads ranging from 9.2 to 107.4 kN (2,065 to 24,155 lbf), or as much as 75% (approximately 143.4 kN (32,250 lbf) of the estimated yield strength of the bolts. Tensile tests of No. 7 slotted rebar bolts showed a mean yield point of 163 kN (36,600 lbf) and an ultimate strength of 233 kN (52,300 lbf). Since a vertical load of 8.90 kN (2,000 lbf) had been applied to the block assembly prior

to shearing, effective load on the tensioned bolt was reduced by that amount. Pretensioned bolt loads, applied shear loads, and shear block displacement for each of the 17 bolts are shown in table 1.

Applied Shear Loads and Displacements

The concrete blocks, the joint surface between the blocks, and the pretensioned bolt together comprise a system that responds to shear loading applied during the test. The general relationship between applied shear load and displacement is shown in a profile from test 14 (figure 3). The point at which the curve departs from the initial steep slope to a lesser slope is here designated the yield shear strength of the joint. Mean yield shear strength for these tests

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Table 1. Instrumented roof bolt pretensioned loads, shear strengths, and displacement.

Test No.	Pretensioned axial bolt load, kN (lbf)	Joint yield strength		Ultimate joint strength	
		Applied shear load, kN (lbf)	Block displacement, cm (in)	Applied shear load, kN (lbf)	Block displacement, cm (in)
1	9.19 (2,065)	125.90 (28,305)	0.528 (0.208)	242.73 (54,570)	3.091 (1.217)
2	22.17 (4,985)	99.21 (22,305)	0.635 (0.250)	222.78 (50,085)	3.475 (1.368)
3*	24.25 (5,451)	103.02 (23,160)	0.610 (0.240)	216.91 (48,765)	3.226 (1.270)
4	28.68 (6,448)	128.04 (28,785)	0.411 (0.162)	259.14 (58,260)	3.162 (1.245)
5	35.37 (7,951)	93.43 (21,006)	0.767 (0.302)	199.63 (44,880)	3.744 (1.474)
6	39.08 (8,786)	116.36 (26,160)	0.406 (0.160)	220.44 (49,560)	2.827 (1.113)
7	53.28 (11,979)	111.49 (25,065)	0.300 (0.118)	217.51 (48,900)	3.185 (1.254)
8	56.93 (12,799)	120.39 (27,066)	0.721 (0.284)	223.91 (50,340)	3.307 (1.302)
9	62.68 (14,091)	92.95 (20,898)	0.470 (0.185)	213.77 (48,060)	3.261 (1.284)
10	66.35 (14,917)	114.62 (25,770)	0.472 (0.186)	227.98 (51,255)	3.012 (1.186)
11*	70.88 (15,935)	92.61 (20,820)	0.330 (0.130)	235.12 (52,860)	3.294 (1.297)
12	75.37 (16,944)	112.14 (25,212)	0.335 (0.132)	220.44 (49,560)	3.495 (1.376)
13*	83.87 (18,855)	103.26 (23,214)	1.293 (0.509)	240.99 (54,180)	4.277 (1.684)
14	92.41 (20,776)	102.56 (23,058)	0.441 (0.162)	200.16 (45,000)	2.865 (1.128)
15	98.49 (22,142)	111.76 (25,125)	0.330 (0.130)	230.25 (51,765)	3.228 (1.271)
16	100.87 (22,678)	129.10 (29,025)	0.262 (0.103)	233.79 (52,560)	3.175 (1.250)
17	107.44 (24,155)	102.95 (23,145)	0.549 (0.216)	227.58 (51,165)	3.518 (1.385)

* Bolt did not break at maximum shear displacement.

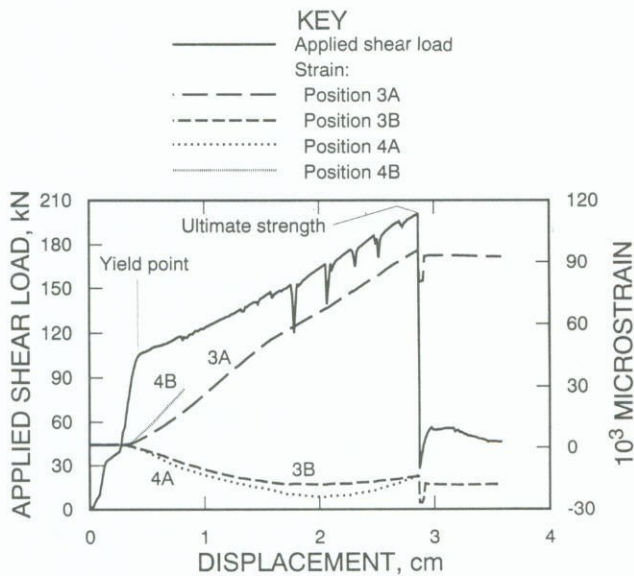


Figure 3.—Load and displacement profile, test 14

was 109.4 kN (24,595 lbf). The point at which the curve beyond the yield shear strength again departs from an approximate straight line because of either bolt failure or rapid loading of the system, is designated as the ultimate shear strength of the joint. Mean ultimate shear strength was 225.5 kN (50,692 lbf). No discernable correlation was found between pretensioned axial loads on the bolt and either yield shear strength or ultimate shear strength of the joint (figure 4). As shear loading progressed, each bolt was deformed into an S bend, with the center part of the bend at an angle to the joint surface between the blocks. In seven cases, applied shear load exceeded the ultimate strength of the bolt. This was attributed to

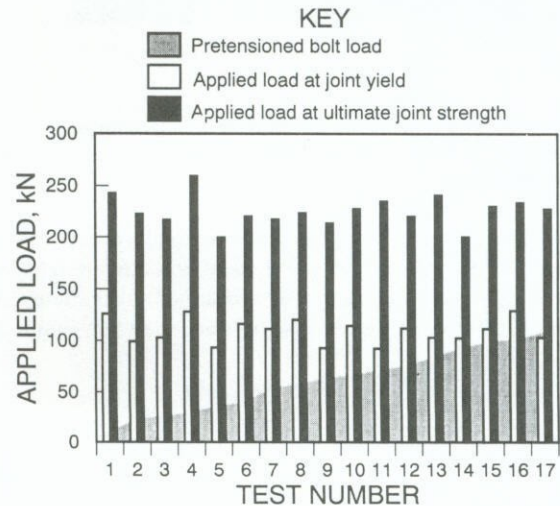


Figure 4.—Pretensioned bolt loads and shear yield/ultimate strength of joint

the bolt being forced through the confined block, crushing the concrete surrounding the hole, which was enclosed on five sides by the steel shear box. Applied shear loads in these cases reached as much as 658 kN (148,000 lbf). In all but three cases (tests 3, 11, and 13), the bolt ultimately broke before the limit of shear displacement was reached.

Displacements at the yield strength of the joint ranged from 0.33 to 1.29 cm (0.13 to 0.509 in) and averaged 0.52 cm (0.204 in). The data are not definitive, but a slight correlation is shown between higher initial axial tension and lower displacement at shear

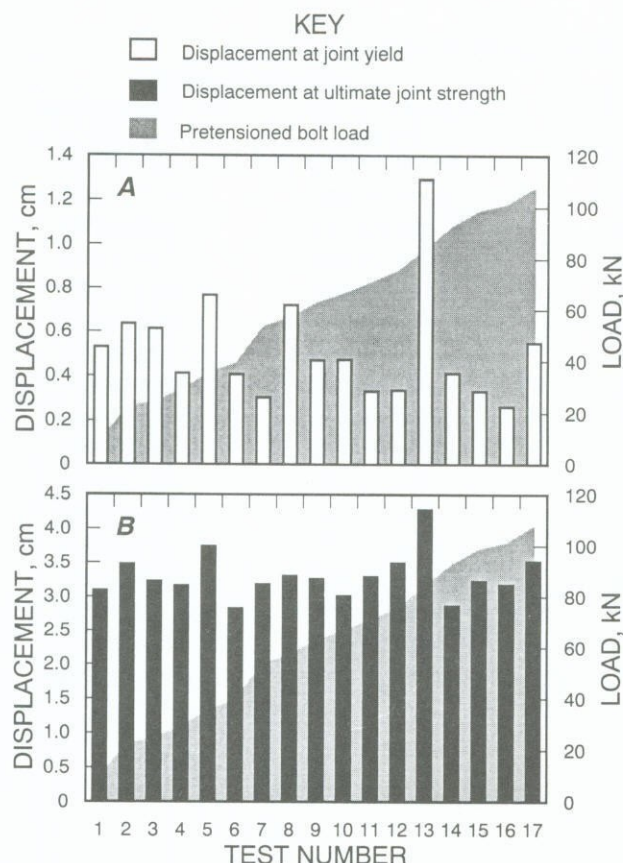


Figure 5.—Relation of initial tension to shear displacement, **A**, Displacement at joint yield strength; **B**, displacement at ultimate joint strength.

yield strength. Displacement at ultimate strength ranged from 2.83 to 4.28 cm (1.113 to 1.684 in) and showed no correlation to initial axial tension; average displacement at ultimate shear strength was 3.30 cm (1.300 in). These relationships are shown in figure 5.

Strain-Gauge Data

Axial and bending strains were calculated for the seven gauge pair positions along the bolt for the duration of each test according to the equations—

$$\epsilon_{\text{axial}} = (\epsilon_{\text{side 1}} + \epsilon_{\text{side 2}})/2 \quad (2)$$

$$\text{and } \epsilon_{\text{bend}} = (\epsilon_{\text{side 1}} - \epsilon_{\text{axial}}) \cdot d/y \quad (3)$$

where d = bolt radius
and y = d - depth of the slot.

Because of the bolt-and-block configuration, the maximum effects of applied shear loads consistently occurred between positions 3 and 4 on the instrumented bolts (figure 1). The gauges at position 4 were at the center of the bolt and therefore in the plane of shearing; electrical connection to these gauges was commonly lost before ultimate strain levels were reached. Signals were

normally not lost for gauges at position 3, and these gauges showed the most consistent maximum strains.

Axial strains at position 3 for all bolts were used to relate pretensioned load to applied shear load. Pretensioned axial strains on the bolts at position 3 ranged from 146 to 1617 microstrain. At shear yield strength of the joints, axial strains ranged from 777 to 2438 microstrain; yield strength of the steel (about 2000 microstrain) was exceeded in only two bolts (tests 7 and 13). Axial strains at ultimate shear strength of the joints ranged from 11,888 to 40,846 microstrain, all well above yield strength of the steel. Variability among tests obscures a generalized correlation between pretensioned bolt loads and axial strain at both yield and ultimate shear strengths of the joints. Mean values were calculated for bolts grouped into those with low (six bolts), moderate (six bolts), and high (five bolts) pretensioned loads (figure 6A).

Bending strains at shear yield strength of the joints were quite uniform and apparently not influenced by initial pretensioned bolt load; yield strength of the steel was exceeded at position 3 for 12 of the bolts. For the grouped data, figure 6B shows a correlation in bending strain between pretensioned bolt loads and ultimate joint shear strength. Bending strains reached as high as 64,029 microstrain at ultimate joint shear strengths.

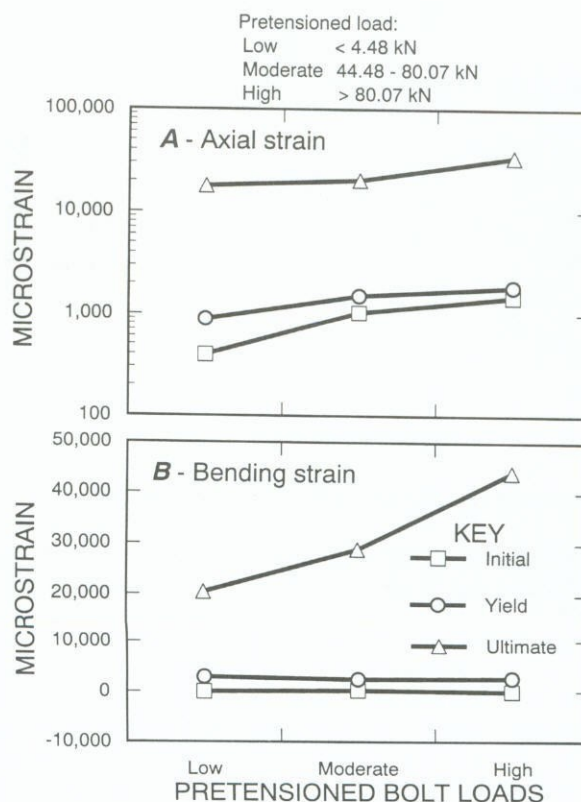


Figure 6.—Axial and bending strains for pretensioned load groups

Strain and load data for gauges at positions 3 and 4 from a representative test (table 1, No. 14) demonstrate how the bolts responded to shear loading (figure 3). Axial strain data for these and other gauges are shown in figure 7. Bolt 14 was pretensioned to 92.41 kN (20,776 lbf) before it was tested. Initial strains on individual gauges ranged from 917 to 1665 microstrain. The effects of applied shear were greatest at gauge sites nearest the center of the bolt. However, signals were lost at positions 4 and 5 before ultimate loads were reached. At the shear yield strength of the joint, axial strain exceeded yield strength of the steel only at position 4. At the ultimate strength of the joint, the yield strength of the steel was exceeded even at position 1, which was 20.3 cm (8 in) from the joint surface.

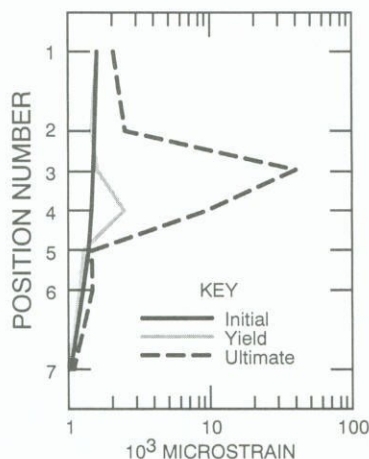


Figure 7.—Axial strain, test 14

Examination of the broken bolt showed that it had been offset 2.3 cm (0.92 in) across the S bend during shearing. About 0.2 cm (0.1 in) of extension was measured in the top 14.1 cm (9.5 in) of the bolt from the bolt end to gauge position 3 (about 10,500 microstrain). The longitudinal centerline of the bolt through the S bend and the rupture extended at least 0.58 cm (0.23 in) over the 10.2 cm (4 in) between positions 3 and 5 (>580,000 microstrain) before the bolt broke. No measurable extension was noted from position 5 to the other end of the bolt.

The gauges at position 3 (figure 8) showed pronounced convex bending on the side of the bolt toward the shear ram (the direction from which shear stress was applied), and strain at gauge 3A increased quickly beyond the yield strength of the steel. On the concave side of the bend (away from the shear ram), strain was compressional, or negative. As shearing proceeded, negative strain decreased to a minimum (-18,000 microstrain for gauge 3B), then reversed and began to increase. Bending at gauge position 4 was reversed from that at position 3, or concave (compressional) on the shear ram side of the bolt. Negative strain decreased to a minimum of -24,000 microstrain at gauge 4A, then began to rise. Strain on the convex side of the bolt (gauge 4B) had already increased to 29,000 microstrain when electrical connection to the gauge was lost. Gauges at position 5 showed bending much reduced, but in the same direction as at position 4.

As was common in all tests, gauges near the ends of the bolt at positions 1 and 7 (20.3 cm [8 in] from the joint surface) responded to shear loading with a slight initial decrease in strain, then an increase at all gauges. No bending was apparent. Gauges at positions 2 and 6, which lay 10.2 cm (4 in) from the center in each direction, also showed eventual increases in axial strain, but also small amounts of bending in the same direction as at positions 3 and 5. Negative strains mirror positive values for gauges on the opposite side of the bolt, but at lesser magnitudes.

CONCLUSIONS

Shear loading of the bolts and the bending that results created tensional (positive) and compressional (negative) strain at the bending site. The effect of applied shear stress was observed throughout the length of the bolt, but was translated to tensional stress at gauges farthest from the joint surface (positions 1 and 7). Bending was observed at gauges 10.2 cm (4 in) from the joint surface (positions 2 and 6), but continuing shear displacement resulted in tensional strain on both sides of the bolt. In two tests, bending strain at positions 2 and 6 exceeded the yield strength of the steel. For all bolts, the magnitude of negative strain was consistently less than the opposing positive strain on the other side of the bolt and was reversed as shearing continued. With increased displacement, strain on both sides of the bolt increased. Eventually applied shear force exceeded ultimate joint strength, and the bolt essentially failed in tension.

The tests suggest that axial loading has little effect on a joint's resistance to shear loading. Yield joint shear strengths may have a minor correlation to axial loading on the bolt, but ultimate joint shear strength is apparently unaffected. Joint shear strength at yield averaged about 76% of expected axial strength of the bolt alone. Field tests in underground coal mines have shown that roof bolts commonly have axial loads beyond their rated yield strength, a condition not duplicated by pretensioned loads in these tests. However, the tests suggest that bolts subjected to these high loads can still provide resistance to shear loading.

Although concrete blocks are similar in compressive strength to the type of host rocks common in coal mines, the hardness of host rocks plays a role in how a bolt responds to shear loading. Concrete on one side of the bolt crushes as it bends, and a void is created on the other side. Very hard host rock would likely tend to slice a bolt off at a joint face rather than allow the bolt to bend. In such cases, bolts may provide less resistance to shear loading. Laboratory tests have been designed to test the effects of shear stresses in harder materials.

Pretensioning of the bolts effectively minimized separation of the joint faces during shear displacement. Unlike fully grouted bolts in mine installations, the nut and plate anchors at the ends of the test bolts ensured that grout failure would not be a factor in load and displacement profiles. Both separation of the shearing faces and plastic failure of grout would tend to propagate bending forces for greater distances along the bolt.

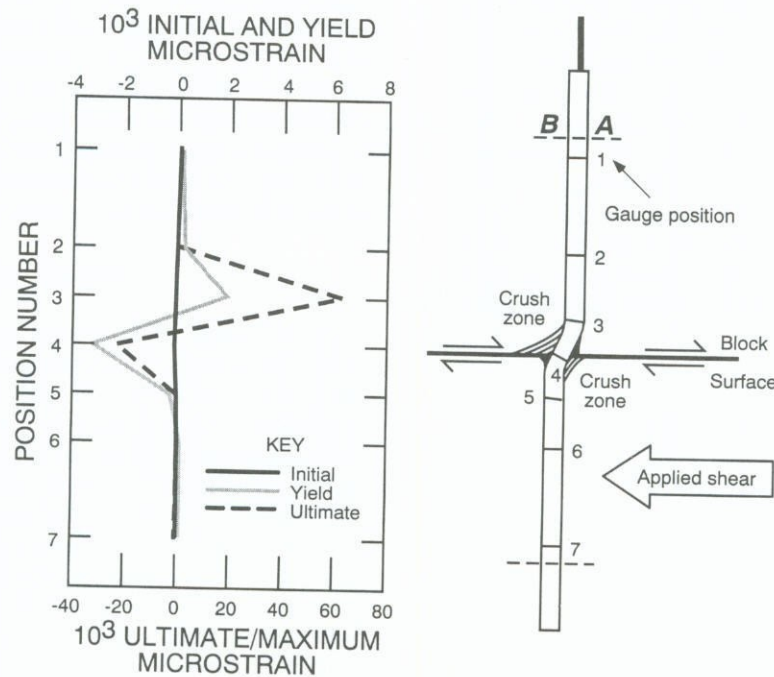


Figure 8.—Bending strain and bolt deformation, test 14.

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Laboratory Study of Shear Loading and Bolt Load Transfer Mechanisms Under Constant Normal Stiffness Conditions

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ABSTRACT

One of the most important parameters in rock/resin/bolt load transfer mechanism is the surface property of the bolt. Although there are various bolt surface configurations available in the market, very little work is reported on the bolt/resin interface failure mechanism. This paper examines the behaviour of bolt surface roughness under constant normal stiffness conditions which is considered as being a realistic way of evaluating bolt surface roughness. A series of shear tests have been conducted on the flattened bolt surface to study the shear behaviour of bolt/resin interface. Based on the experimental results, a mathematical model has been developed to predict the shear resistance of the bolt/resin interface at various normal stress levels for the bolt types used in the study.

INTRODUCTION

A large number of roof bolts are being used every year to support the gate roadways in the mines. The load transfer characteristics of the bolt plays a very important role in the design of effective support system for stabilising rock mass in various types of excavation. The load transfer characteristics of bolt in the field largely depends on the behaviour of its surface among other parameters. The study of load transfer mechanism would remain incomplete without the study of bolt surface roughness and the shear behaviour of bolt-resin interface. In the recent past, the shear stress developed at bolt-resin interface have been calculated by the use of strain gauged bolts by many authors [1, 2, 3, 4]. Accordingly, the shear stress developed at any point could be calculated by the following formula:

$$\Delta \tau = \frac{F_1 - F_2}{\pi d l} \quad (1)$$

Where,

$\Delta \tau$ = Shear stress at bolt-resin interface,

F_1 = Axial force acting on the bolt at strain gauge position 1, calculated from strain gauge reading,

F_2 = Axial force acting on the bolt at strain gauge position 2, calculated from strain gauge reading,

d = Bolt diameter, and

l = Distance between strain gauge position 1 and strain gauge position 2.

One of the major shortcomings of the above method is that, it does not consider the effect of horizontal stress or the confining pressure on the shear stress at bolt/resin interface. As the confining pressures or the horizontal stresses around the opening play a very important role in the failure mechanism of grouted rock, incorporation of the confining pressure in the above formula would result in a better approximation of the in-situ condition. Following the installation of bolt in the field the relative movement, though very small, between the rock and the bolt causes the load to be transferred on the bolt. In this process the normal stress is applied on the resin/rock interface through the ribs of the bolt. This is dependent on the relative displacement, shape of the rib and the composite stiffness of the bolt/resin/rock interface. The stiffness is a material property and cannot vary without any change in material and it is the normal stress which varies during the load transfer process. The Constant Normal Stiffness condition thus represents a better approximation of the deformation behaviour in the field as compared to much used Constant Normal Load condition. The above hypothesis has been indicated by many researchers [5, 6, 7, 8, 9]. A novel approach was, therefore, adopted to study the shear behaviour of bolt/resin interface under Constant Normal Stiffness (CNS) condition. In this method, the confining pressures were modelled by applying predefined normal stresses on the bolt/resin interface and the CNS condition was achieved by an assembly of four springs on top of the upper shear box. The CNS testing machine developed at the University of Wollongong and reported



earlier [8] was modified to incorporate the surface feature of the bolt for direct shear testing of cast resin blocks.

BOLT SURFACE PREPARATION

A 100 mm length of a bolt was selected for the surface preparation for CNS shear testing. Each specified length of bolt was cut and then drilled through as shown in Figure 1. The hollow bolt segment was then cut along the bolt axis from one side and preheated to open up into a flat surface as shown in Figure 2. The surface feature of the bolt were carefully protected while opening up the bolt surface. The flattened surface of the bolt was then welded on the bottom plate of the top section of the shear box of the CNS testing machine as shown in Figure 3. Table 1 shows the detail specification of two types of bolt used for the study.

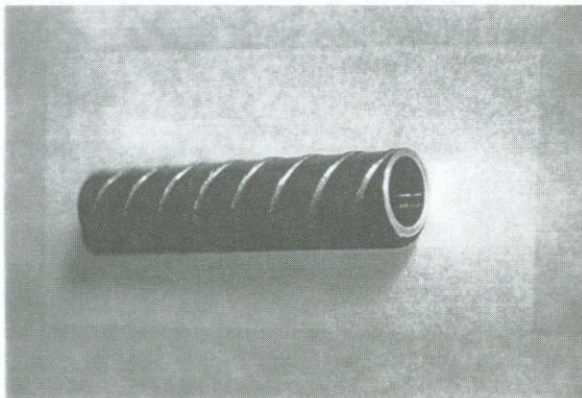


Figure 1: Photograph showing hollow bolt segment

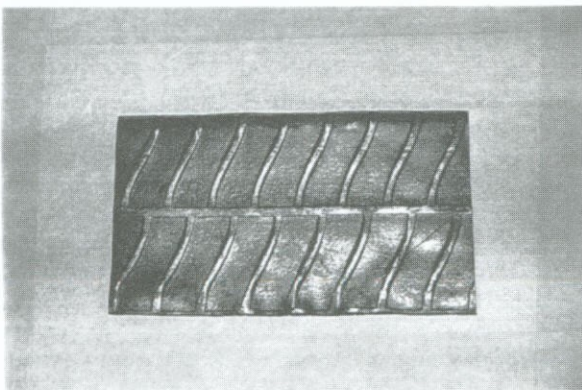


Figure 2: Photograph showing flattened bolt surface

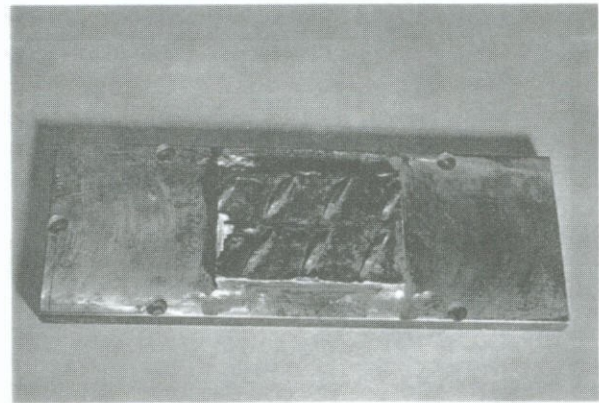


Figure 3: Photograph showing bolt surface welded on the top shear box plate

Table 1: Specification of bolt types

Bolt	Core Dia (mm)	Finished Dia (mm)	Rib Spacing (mm)	Rib Height (mm)
Type I	21.7	24.4	28.5	1.35
Type II	21.7	23.2	12.5	0.5

SAMPLE CASTING

The welded bolt surface on the bottom plate of the top shear box was used to cast samples on it. The arrangement for casting the sample is shown in Figure 4. For obvious economic reason the samples were cast in two parts. Nearly two-third of the mould was cast with high strength casting plaster and the remaining one-third was topped up with Fosroc Chemfix *mix and pour grout resin PB 1*. Figure 5 shows a typical cast sample used for the shearing tests and Figure 6 shows a cast sample in the shear box of the CNS testing machine. The samples prepared in this way fitted exactly into the bolt surface during testing and allowed a close representation of the bolt-resin interface in practice.

CNS TESTING MACHINE

Figure 7 shows the general view of the CNS testing machine. The detail of the machine has been reported earlier [8]. The cast resin sample was placed in the bottom shear box and the welded bolt surface was mounted at the base of the top shear box. The shear force was applied through the bottom shear box causing it to move in horizontal direction while the movement of upper shear box was restricted to



vertical direction. The mould resin surface was maintained in contact with bolt surface during shear testing. Load cells were used to measure both shear and normal loads and dial gauges were used to measure the shear displacement and dilation during testing.

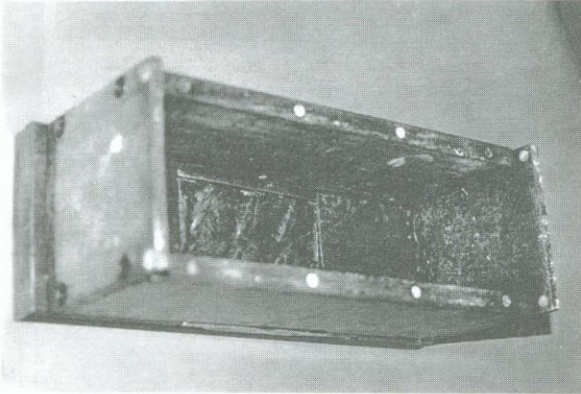


Figure 4: Photograph showing arrangement of casting resin samples

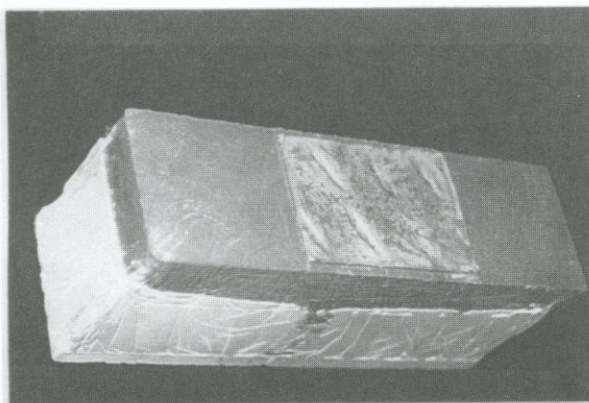


Figure 5: Photograph of cast resin block

EXPERIMENT AND DATA ANALYSIS

A total of twelve samples were tested for two different types of bolt surface at initial normal stress (σ_{no}) levels ranging from 0.1 to 7.5 MPa. Each sample for bolt type I was subjected to five cycles of loading in order to observe the effect of repeated loading on bolt/resin interface. Samples for bolt type II was subjected to only three cycles of loading as it was found that the stress profile did not vary significantly after third cycle of loading. The σ_{no} applied to the samples represented typical confining pressures which might be expected in the field. Figures 8 through 13 show the stress profile and dilation behaviour of bolt/resin interface for



Figure 6: Photograph showing cast resin samples inside the bottom shear box

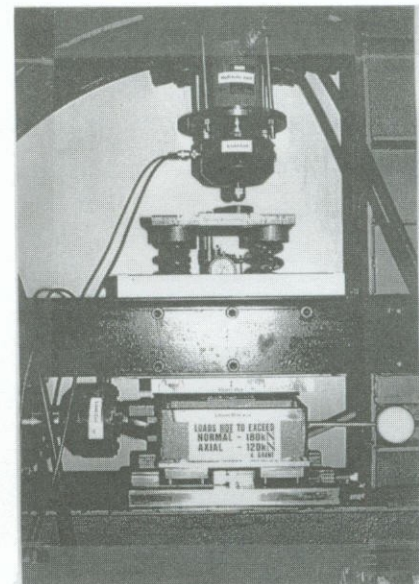


Figure 7: Photograph showing CNS testing arrangement

various normal stress conditions for bolt type I. The difference in stress profiles for various loading cycles were negligible at low values of σ_{no} . This was gradually increased with increasing value of σ_{no} reaching maximum at 3 and 4.5 MPa. The increased gap between peak shear stress level becomes more prominent between loading cycles I and II as shown in Figures 10 and 11. Beyond 4.5 MPa confining pressure the stress profile gap between I and II cycles decreased again. Similar trend was also observed for bolt type II.

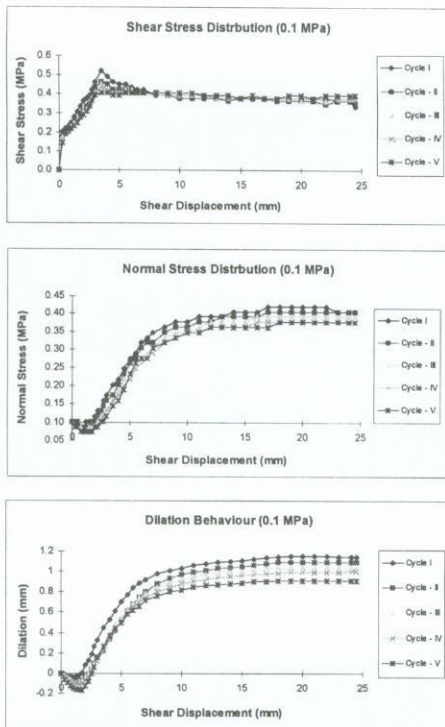


Figure 8: Stress profile and dilation of type I bolt at an initial normal stress of 0.1 MPa.

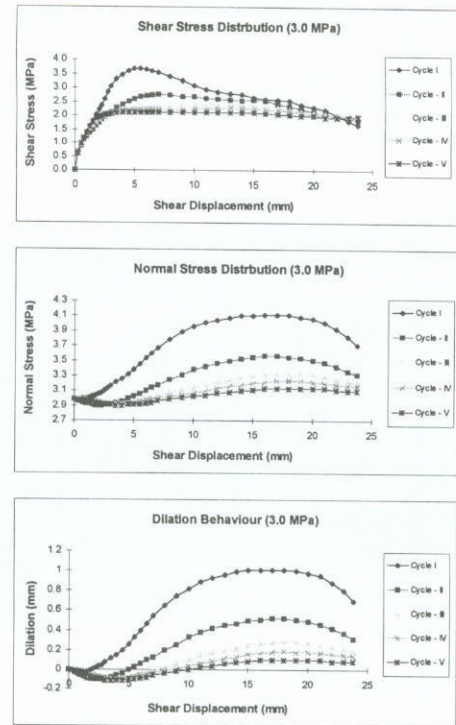


Figure 10: Stress profile and dilation of type I bolt at an initial normal stress of 3.0 MPa.

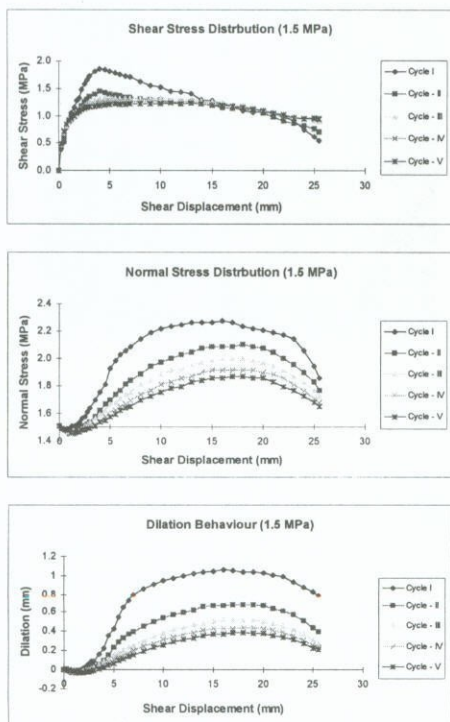


Figure 9: Stress profile and dilation of type I bolt at an initial normal stress of 1.5 MPa.

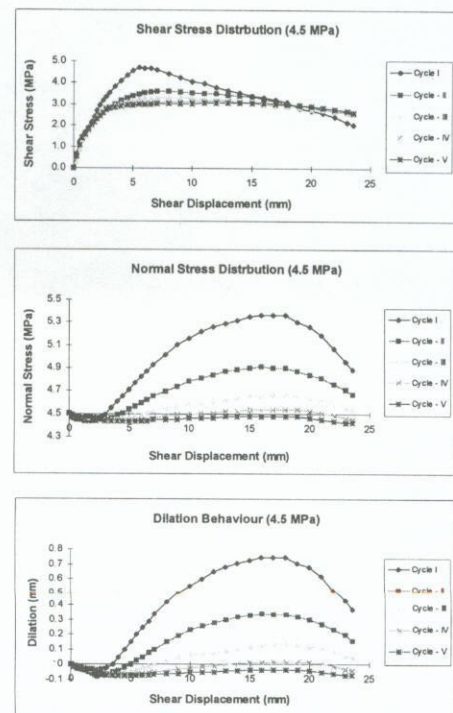


Figure 11: Stress profile and dilation of type I bolt at an initial normal stress of 4.5 MPa.

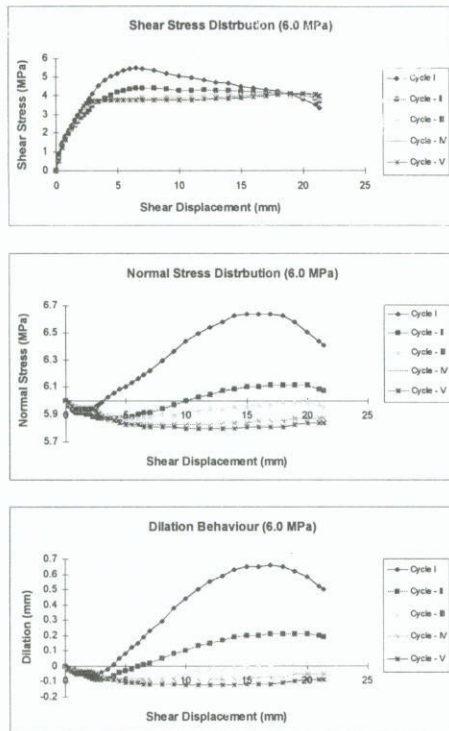


Figure 12: Stress profile and dilation of type I bolt at an initial normal stress of 6.0 MPa.

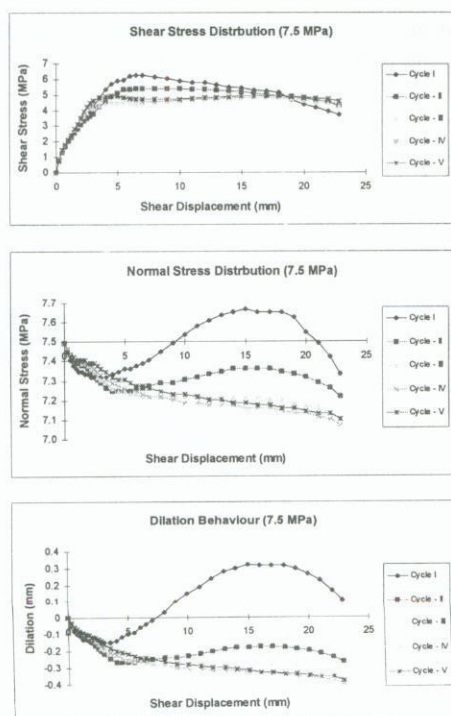


Figure 13: Stress profile and dilation of type I bolt at an initial normal stress of 7.5 MPa.

At low σ_{n0} values, the relative movement of the bolt/resin surface caused an insignificant shearing and slickensiding of the resin surface, and for each additional cycle of loading the shear behaviour remained almost constant. However, as the value of σ_{n0} was increased, the shearing of the resin surface was also increased and the difference in stress profiles for various cycles of loading became significant. At the medium level of normal stress application, the shearing and slickensiding of the contact surface was not sufficient to smoothen the resin surface excessively in the first cycle, and further cyclic loading caused a significant difference in the stress profile. As the initial normal stress was increased beyond this level, the difference in stress profile for various cycles of loading was decreased due to excessive shearing and smoothening of resin surface after the first cycle of loading. The difference in stress profile after third cycle was negligible at all normal stress levels thus indicating no change in surface properties after third cycle of loading as shown in Figure 8 through 13.

The dilation of bolt/resin surface for both the bolt types were characterised by the initial contraction and then subsequent increase in dilation for all the loading cycles. This initial contraction may be due to the early settlement of very fine irregularities of the resin surface. The dilation was reduced with increasing value of σ_{n0} , correspondingly the increase in normal stress was also reduced. This gives an indication that the behaviour of rock/resin interface at CNS condition may approach to CNL condition at very high σ_{n0} values. The characteristics of normal stress profiles were similar to dilation behaviour as shown in Figures 8 through 13 for type I bolt.

Figure 14 and 15 shows the variation of shear stress, normal stress and dilation with shear displacement at various normal stresses for first cycle of loading for type I and type II bolts respectively. The maximum dilation occurred at a shear displacement between 17 to 18 mm and 7 to 8 mm for type I and type II bolts respectively at all values of σ_{n0} for almost all loading cycles, indicating the minimum shearing of resin surface at a distance of 17 to 18 mm for type I bolt and 7 to 8 mm for type II bolt from the initial position. The distance between the ribs for both the bolt types are shown in Table 1. Therefore, it may be concluded that the maximum dilation occurred at a shear travel of nearly 60% distance of bolt rib spacing. This need to be confirmed with the other surface patterns which is currently under investigation.

The shear displacement for peak shear stresses increased with increasing value of σ_{n0} for both the bolt types. This was due to increased amount of shearing of resin surfaces with the increasing value of σ_{n0} . However, there was a gradual reduction in the gap between the peak shear stresses with increasing value of σ_{n0} . As discussed later in the mathematical model in detail, the shear displacement required to reach the peak shear strength is a function of normal stress being applied and the surface properties of the

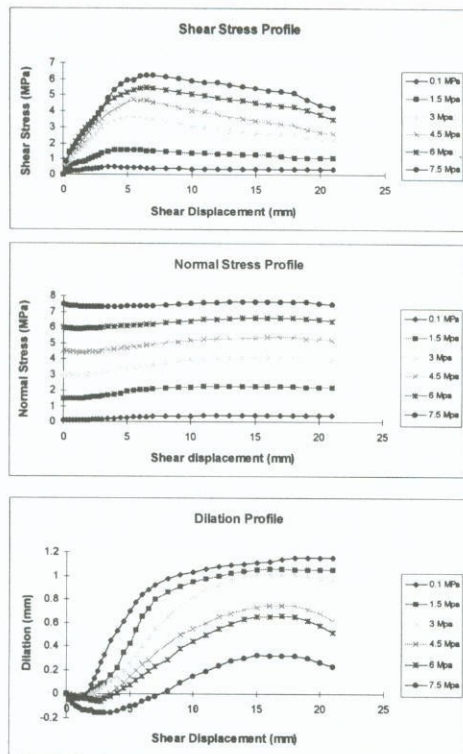


Figure 14: Stress profile and dilation of type I bolt for first cycle of loading.

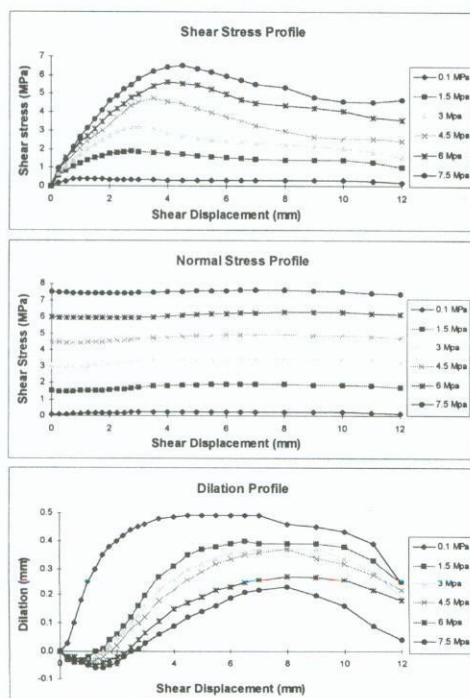


Figure 15: Stress profile and dilation of type II bolt for first cycle of loading.

resin assuming that the property of the bolt surface remains constant for a particular type of bolt. This is clearly evident from Figure 14 and 15.

Figure 16 and 17 shows the variation of peak shear stresses with normal stress applied for various loading cycles for type I and type II bolts respectively. For type I bolt the graphs of cycle I through cycle III show a dual trend whereas the graphs representing both cycle IV and V show only a single trend. For bolt type II, only cycle I shows a dual trend and cycle II and III show a single trend. At the low initial normal stress, the shearing of resin surface is negligible and hence the rate of increase of peak shear stress with respect to normal stress is high. At higher normal stress, the degree of shearing of resin surface is greater and some of the energy is thus utilised to shear off the resin surfaces. As a result, there is a lower increasing rate of peak shear stress with respect to the normal stress. As the samples are loaded repeatedly, the resin surface become smoothened reducing the surface roughness and as a result the rate of increase of peak shear stress is likely to remain constant with respect to normal stress.

Figure 18 shows the stress profile of both type I and type II bolts plotted together for the first cycle of loading. The following were observed from Figure 18.

- Peak shear stress values are nearly similar for both bolt types.
- Post peak shear stress values are higher in bolt type I indicating better performance in post peak region.

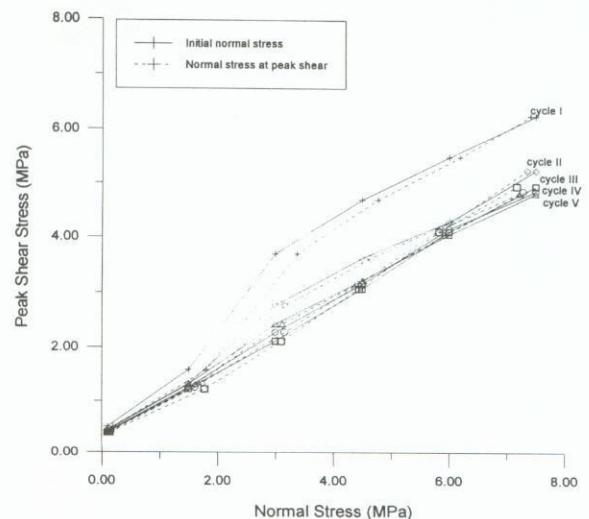


Figure 16: Variation of peak shear stress with normal stress for type I bolt

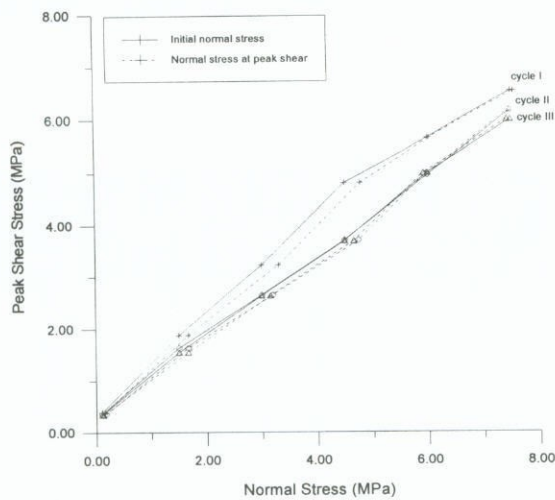


Figure 17: Variation of peak shear stress with normal stress for type II bolt

- Shear displacements at peak shear are higher in bolt type I indicating the safe allowance of more roof convergence before instability stage is reached.
- Dilation is more in bolt type I

The implications of the above observation need detail explanation. The laboratory experiments were carried out with spring assembly with an effective stiffness of 8.5 KN/mm. In practice the stiffness of resin/rock system will be much higher than the laboratory simulated stiffness. As the stiffness increases the effective normal stress on the bolt/resin interface at any point of time will also increase as per following equation:

$$\sigma_n = \sigma_{no} + \frac{k \cdot \delta_v}{A} \quad (2)$$

where,

σ_n = Effective normal stress,

σ_{no} = Initial normal stress,

k = System stiffness,

δ_v = Dilation, and

A = Area of bolt surface.

Therefore, much higher values of effective normal stresses should be observed in type I bolt as compared to type II bolt. Higher values of effective normal stress will have a direct positive impact on the peak shear stress values. In field situation type I bolt should outperform the type II bolt as the ground stiffness is much higher than laboratory simulated stiffness.

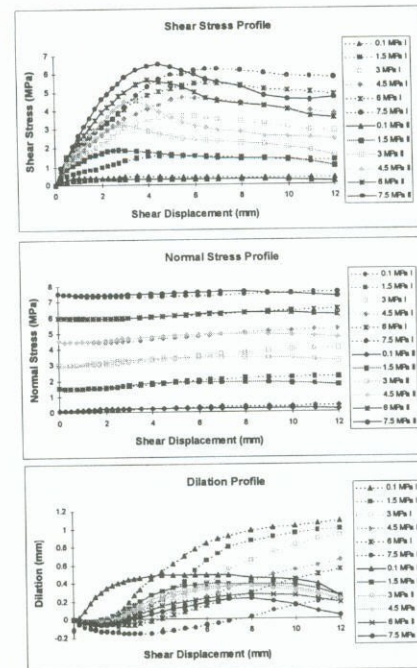


Figure 18: Comparison of stress profile and dilation of type I and type II bolt for first cycle of loading.

MATHEMATICAL MODEL

Based on the above study a mathematical model was developed to predict the shear resistance of bolt-resin interface at any normal stress within the range of experimental study. The shear stress at any particular point of time could be expressed as a function of shear displacement, normal stress and the surface properties of bolt. Examination of the shapes of shear stress profiles at various normal stresses suggests the following empirical relationship[10].

$$\tau = a \cdot x^b \cdot e^{c \cdot x} \quad (3)$$

where,

τ = Predicted shear stress,

x = Shear displacement,

e = Natural log base,

a = $f(\text{normal stress})$,

b = $f(\text{normal stress})$, which takes into account the effect of shearing of asperities at high normal stress values, and

c = $f(\text{surface properties})$ i.e. constant for a particular set of bolt-resin interface and is equal to -0.092 and -0.29 for type I and type II bolt respectively.

Assuming that the parameters a and b are constant for any particular normal stress, the peak shear stress can be obtained by differentiating equation (3) with respect to the shear



displacement and equalling it to zero. Therefore, for τ to be maximum,

$$\frac{d\tau}{dx} = 0 \quad \text{or, } x = -\frac{b}{c} \quad (4)$$

By incorporating the value of x in equation (3) we have,

$$\tau_{\max} = a \cdot \left(-\frac{b}{c \cdot e} \right)^b \quad (5)$$

For the set of data generated from the laboratory experiments the following relationship was obtained based on the regression analysis.

If N = initial normal stress:

for type I bolt,

$$a = 0.425 + 2.79 e^{\frac{-1.9}{N}} \quad (6)$$

$$b = 0.405 + 0.08 N \quad \text{for } N \leq 3 \text{ MPa} \quad (7a)$$

$$= 0.580 + 0.0265 N \quad \text{for } N > 3 \text{ MPa} \quad (7b)$$

for type II bolt,

$$a = 0.45 + 3.48 e^{\frac{-1.29}{N}} \quad (8)$$

$$b = 0.525 + 0.142 N \quad \text{for } N \leq 4.5 \text{ MPa} \quad (9a)$$

$$= 0.95 + 0.0467 N \quad \text{for } N > 4.5 \text{ MPa} \quad (9b)$$

The two different functional values of b could be explained from Figure 16 and 17 where the trend of the lines are divided into two segments. The first and steeper trend is continued up to 3 MPa for type I bolt and 4.5 MPa for type II bolt, after which the second and flatter trend starts indicating the effect of shearing of asperities at high normal stress levels. The above anomaly in trends necessitates the use of different functional values of b for different normal stress levels. Table 2 shows the calculated values of a , b and c using above equations for various normal stress levels.

Figure 19 and 20 shows the predicted values of shear stresses for any shear displacement at any particular normal stress and the corresponding values obtained from the laboratory experiments for type I and type II bolts respectively. As can

be seen from the Figure 19 and 20 the predicted values are very close to the experimental values throughout the loading period.

Table 2: Calculated values of a , b and c

Normal stress (MPa)	Type I bolt			Type II bolt		
	a	b	c	a	b	c
0.1	0.43	0.41	-0.092	0.45	0.54	-0.29
1.5	1.2	0.52	-0.092	1.9	0.74	-0.29
3.0	1.9	0.65	-0.092	2.7	0.95	-0.29
4.5	2.25	0.699	-0.092	3.06	1.16	-0.29
6.0	2.46	0.739	-0.092	3.25	1.23	-0.29
7.5	2.59	0.78	-0.092	3.38	1.30	-0.29

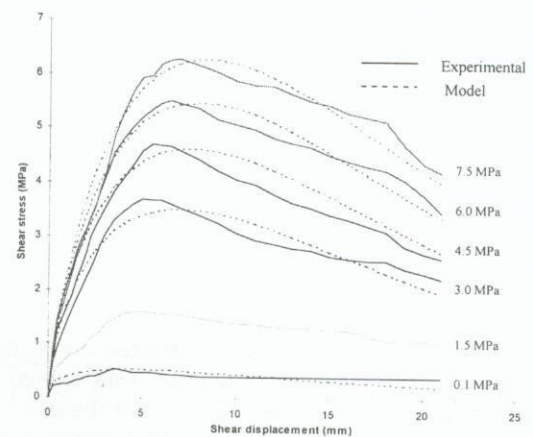


Figure 19: Comparison of predicted values of shear stresses and the values obtained from experiments for type I bolt.

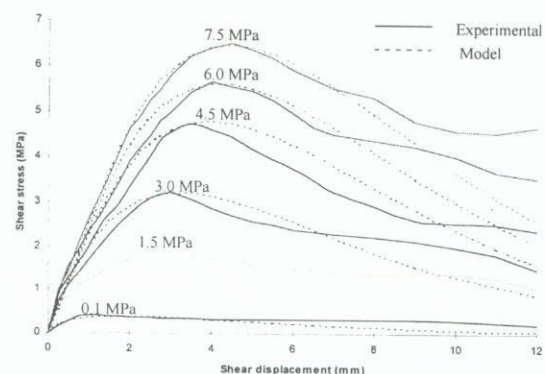


Figure 20: Comparison of predicted values of shear stresses and the values obtained from experiments for type II bolt.



CONCLUSION

The study of bolt surface roughness is directly related with the shear behaviour of bolt/resin interface in the field. The shear behaviour of bolt surface at various confining pressures (which is being modelled through the various normal stresses during laboratory testing) will directly affect the load transfer mechanism from the rock to the bolt and will be a practical guiding tool for designing suitable bolt types for a particular geo-mining condition. The impact of repeated loading on the effective shear resistance of bolt/resin interface was examined and found to vary with the level of applied normal stress, the number of loading cycles and the surface profile of the bolt. The displacement at peak shear was found to vary with applied initial normal stress. Two different types of bolt studied revealed dissimilarities in shear behaviour. Bolt type I showed better performance than bolt type II in many respect. The proposed mathematical model could predict the complete shear stress profile at various normal stress conditions with very close approximation to the laboratory results. The future study in this line would reflect the relative performance of various type of bolt surfaces under different normal stress condition and a comprehensive guideline for the selections of bolt type for a particular geo-mining condition would be established.

ACKNOWLEDGMENT

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Analysis of Cable Bolt Performance Using Numerical Modeling

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ABSTRACT

A finite element modelling approach is developed for the evaluation of the mechanical performance of cable bolts supports in underground mine structures. The modelling approach permits the simulation of any arbitrary number or pattern of cable bolts, that can be either partially or fully grouted, with or without end plates. The modelling technique used treats each cable bolt as a set of bar elements representing the steel cable. The bond stiffness characteristics of the grout bolt interface are represented by shear springs uniformly distributed along the cable length. Coupled with a simple linear elastic analysis, this technique can serve as a dedicated numerical model for cable bolted excavations. The application of the model is illustrated with two case studies: one for a hard rock mine and another for a coal mine.

INTRODUCTION

Cable bolts have been used extensively in underground mines in recent years. They initially appeared in hard rock mines some three decades ago, and more recently they were introduced in underground coal mines. Intensive research efforts on cable bolts have successfully led to leap advancements in their technology. For example, innovative cable bolt geometries, grout mixes, grout pumps and installation techniques have been introduced, tested and shown to give promising results. The most common type of cable bolts used in Canadian mines is the 7-wire, 5/8" single strand. The wires are made of high strength steel, and give a total ultimate tensile strength of 26 tonnes. Modified geometry cable bolts were subsequently introduced; they include the bulbed cable, steel birdcage and cable strand with buttons (Figure 1). In highly fractured ground, and ground with potential relaxation after installation, birdcaged, nutcaged or bulbed strand offer better performance due to their stiffer characteristics. In corrosive mine environments, epoxy coated cable bolts are recommended. The need to monitor the performance of cable bolts in-situ have led to the development of different devices based on different measurement techniques.

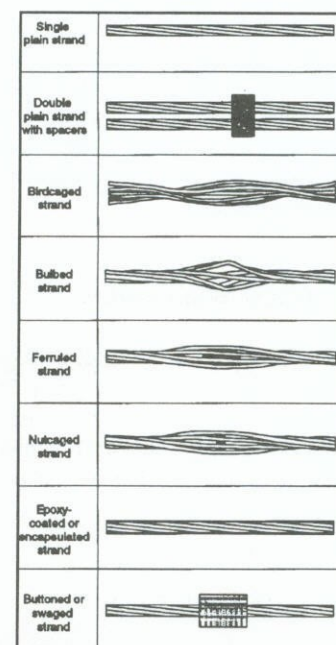


Figure 1. Modified geometries of cable bolts [1]

INSTALLATION TECHNIQUES

In situ installation methods are numerous. The selection of the best method depends on the type of grout used. Generally speaking, two groups of cable grouts are currently in use: cement-based grout and polyester, resin-based grout. Cement grout consists of water and Portland cement with occasional use of additives such as sand, silica fume and plasticizers [2]. Resin grout became more popular in coal mines because of its mechanical characteristics and ease of installation in the form of resin cartridges. Also, it is often preferred when effective reinforcement is required from cables in a short period of time after installation. Resin grouts usually set in two hours and achieve appreciable bond strength in 12 hours, whereas cement based grouts may require 24 to 48 hours to achieve equivalent bond strength [3].

The cable bolt installation methods most commonly used for cement grout are : The breather tube method and the grout tube method [4]. The breather tube method (Figure 2-a) is used for upholes only with grout of 0.375-0.45 water:cement (w:c) ratio. The optimum grout for this method is one with w:c ratio of 0.4. This method should be used with caution in areas with open fractures which may cause grout loss with thinner grouts and may prevent complete filling of the hole. In loose, thinly laminated ground, caution is required to avoid pressuring fractures causing the laminations to separate and rupture.

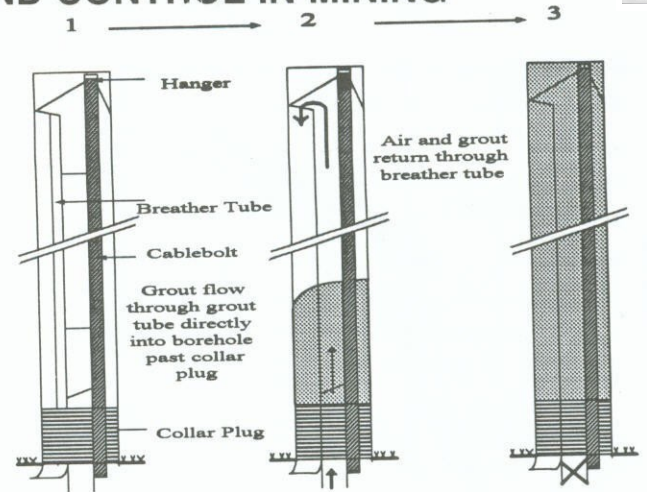
The grout tube method (Figure 2-b) can be used for any hole orientation and with grout of 0.3 to 0.375 w:c ratio. The optimum grout for this method should have a w:c ratio of 0.35. These thicker grouts may cause pumping difficulties when less powerful pumps in long holes are used. Cable bolts are installed in lengths varying from a few meters to some 25 meters.

For resin cable bolt, the installation is different (Figure 2-c). after drilling the desired hole based on the correct depth, correct cartridges are inserted into the drilled hole. The cable is inserted under rotation and after the resin cable bolt has been spun for the specified time, plates and nuts are mounted .

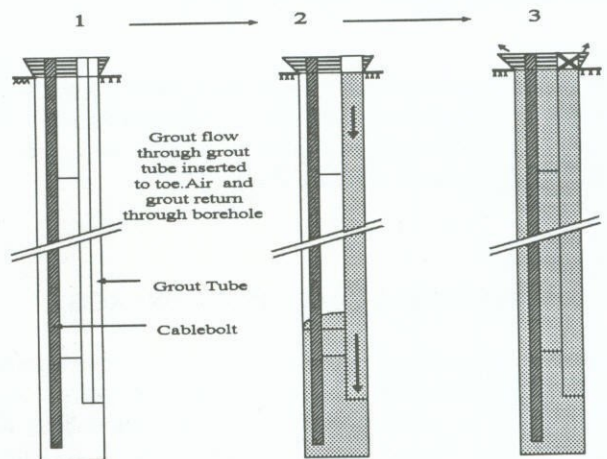
EMPIRICAL DESIGN METHODS

The design of cable bolts requires the understanding of the mechanical behaviour of cables, grout and the rockmass. At present, the most common method of design of cable bolts in Canadian mines is the so-called Modified Stability Graph Method [5].

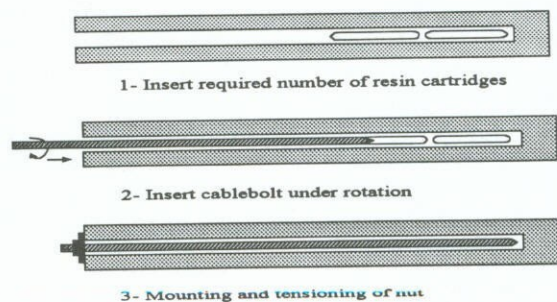
Empirical methods are based on observations in the field and the engineer's experience to estimate the stability of mine stopes. The first methods were developed in the seventies: the RMR classification system [6] and the Q-system [7]. In 1980, Mathews et al. [8] developed the stability graph method for the mining industry. The method uses an adjusted "Q" rating to determine stable excavations dimensions. This Q' rating is then adjusted for induced stresses, orientation of structure and orientation of the surface being examined. The method was later modified by Potvin [5] on the basis of a large database from operating mines. It is called the modified stability graph method which is presently the most popular method in Canadian mines. The modified stability graph method relates the modified stability number (N') to the hydraulic radius of a stope wall. The hydraulic radius (S) is given by the surface area of the stope wall or back divided by its perimeter. The design graph, as shown in Figure 3, has essentially two main zones: "stable and caved" separated by three narrow zones: unsupported transition zone, stable with support, and supported transition zone.



a) Breather tube method



b) Grout tube method



c) Resin grout

Figure 2. Installation of cable bolts

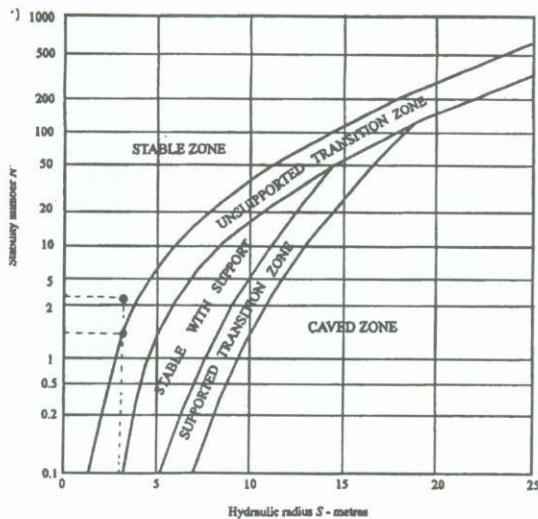


Figure 3. Modified Stability Graph Method [5]

This method has some limitations for example it doesn't take into account some important parameters like mining induced stress distribution over the stope wall (only the point at the center is considered for the evaluation of stability).

NUMERICAL MODELLING APPROACH

The use of numerical models for mining geomechanics applications has been increasingly popular in recent years. One area that saw much numerical modelling work is the stability of underground mine structures and the assessment of rock failure phenomena such as rockburst. A finite element package called e-z tools [9] was developed at McGill University to handle, 2-dimensional, linear elastic, stress/failure analysis of underground excavations. The package has a capability for modelling internal supports like rockbolts and cable bolts. The complete details of the finite element formulation are reported elsewhere [10]. The modelling technique permits the simulation of each cable bolt as a set of bar elements that are connected to the rock mass by shear springs; see Figure 4. The geometric and mechanical properties of the bar element represent those of the steel cable, whereas the shear springs represent the bond stiffness of the grout and its interfaces with the cable surface and the rock wall. Thus, the input modelling parameters required for the simulation of cable bolts are:

- A_b = cross sectional area of the cable bolt
- E_b = modulus of elasticity of the cable material
- K_b = shear bond stiffness between the cable surface and the host rock (thus depending on the type of grout)

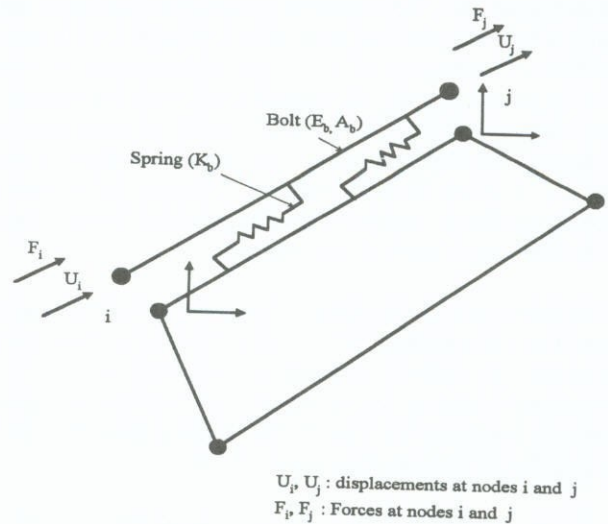


Figure 4. Cable bolt modelling technique in e-z tools software [9]

Modelling procedure

The first step in the analysis procedure is to carry out a preliminary stress analysis of the mine opening. This requires knowledge of the following input parameters: a) Geometry of the opening(s), b) In situ stresses, and c) Geomechanical data of the rock mass. With this information, a finite element model can be constructed and the mining-induced stresses and deformations are calculated. The second step is to determine the need, if any, for ground support, i.e. the delineation of any potentially unstable zones, thus requiring support. In this context, unstable zones may be defined as those areas where failure is induced by high stress concentration, or high stress relief thus creating a zone of ground relaxation and possible sloughing/caving. If the results of the preliminary analysis are indicative of unstable zones around the excavation, the method proceeds to the next step. This involves the selection of a cable bolting system, i.e., a proposed bolting pattern, with specified cable geometry, grout mix, etc. A second stress analysis can then be carried out to examine the effectiveness of the selected cable bolting system. In the second analysis, both the cable bolts and unstable zones are simulated. The latter is done by reducing the elastic properties of the rock mass in the unstable zones. When the purpose of the work is to design a support system, different cable systems may be tried out, in an iterative process, until a satisfactory system is judged appropriate. Otherwise, the above mentioned procedure can be used for back analysis of existing cable support systems. This process is illustrated by two practical case studies, one for a hard rock mine and another for a coal mine which are explained below.

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CASE STUDIES

Two case studies will be presented using cable bolts as ground support: one for a hard rock mine and another for a coal mine.

Stope hanging-wall support by cable bolts

Figure 5 shows a cross section of an open stope in a Canadian underground mine using sublevel, longhole mining method. The stope, located 1170m below surface, is 30m high and 7m wide. The orebody is a stiff, massive sulphide, sandwiched between soft, schistose host walls. As a result, ore extraction causes significant stress relief in the walls, which results in ore dilution when the walls are not supported. To remedy the situation, cable bolts are installed, prior to stope mining, from a hanging wall drift in a fanning pattern as shown in Figure 5. This pattern is repeated every meter in the strike direction of the orebody. The geomechanical data are listed in Table 1 below. The in situ stresses at the mine site are given by:

$\sigma_y = \gamma H$, (vertical stress) MPa
 $\sigma_x = K_x \gamma H$ (horizontal stress in the plane of the section) MPa
 $\sigma_z = K_z \gamma H$ (horizontal stress in parallel to the strike) MPa,
 where, H is the depth below surface (1170m), γ is the average unit weight of the rock mass (0.027 MN/m³), $K_x = 2.26$, and $K_z = 1.52$.

The deformation modulus of the rock mass reported in Table 1 was calculated in terms of its Young's modulus of elasticity (of intact rock material) and rock mass rating RMR as follows [11]:

$$E_{RM} = \frac{E}{2} \left[1 - \cos \frac{\pi(RMR)}{100} \right]$$

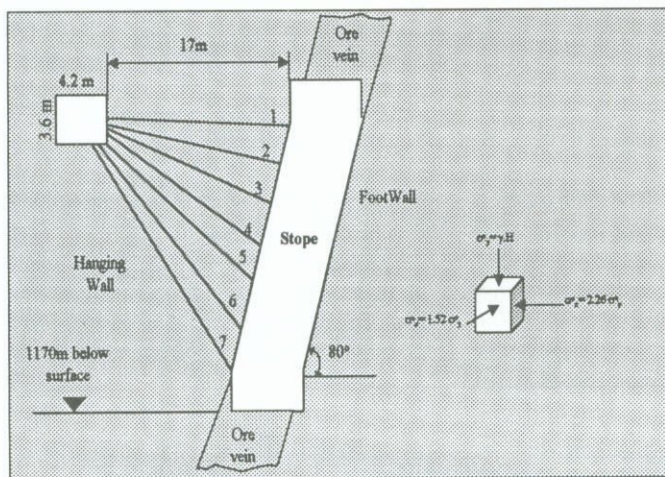


Figure 5. Sublevel open stope supported by 7 cable bolts

A finite element model was constructed using the above mentioned data. A reasonably dense mesh was designed around the mine stope to accommodate high stress gradients; refer to Figure 6. The results of this preliminary analysis show high stress concentration reaching up to 241 MPa in the stope top and bottom

corners. It also shows significant stress relief zone both in the hanging-wall and footwall with relief or tensile stresses reaching nearly 19 MPa.

Property	Hanging wall (HW)	Foot wall (FW)	Sulphide Orebody	Weak rock (HW)	Weak rock (FW)
γ (MN/m ³)	0.027	0.027	0.027	0.025	0.025
E (MPa)	54000	75000	125000		
RMR	50	55	75		
E_R (MPa)	31000	49000	115000	3100	4900
ν	0.21	0.15	0.1	0.3	0.3

Table 1. Rock mass properties used for preliminary modelling

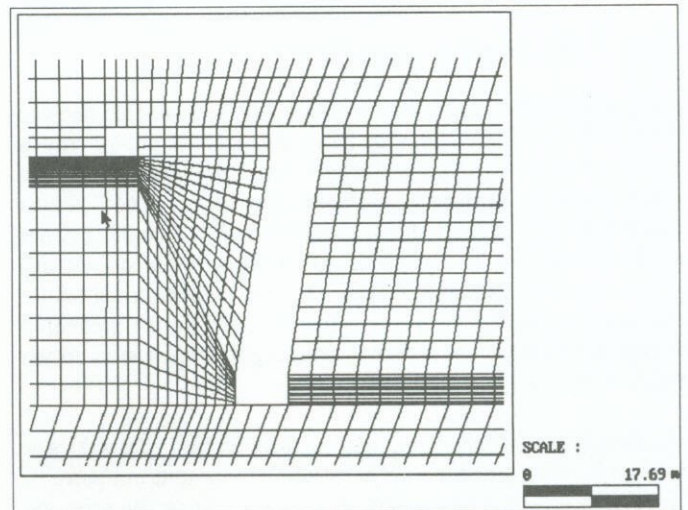


Figure 6. Zoom-in view of the finite element model

Analysis of results

The analysis with cable bolts is conducted with the cable bolting pattern shown in Figure 5. Standard 7-wire, 5/8" diameter, stranded cable bolts are used. These cables have a load bearing capacity of 26 tonnes. The following input parameters were used in modelling of cable bolts: $A_b = 139 \text{ mm}^2$ (cross sectional area), $E_b = 200 \text{ GPa}$ (modulus of elasticity) and bond stiffness per unit length $K_b = 20 \text{ MN/m/m}$ (for standard cables with grout of w:c ratio of 0.35). Additionally, the caving zones identified by the tension zones (Figure 7) were modeled as weak rock masses with their deformation modulus equal to 10% of the original values given in Table 1. Thus, the hanging-wall caving zone was given a modulus of 3100 MPa, and that of the footwall 4900 MPa [12].

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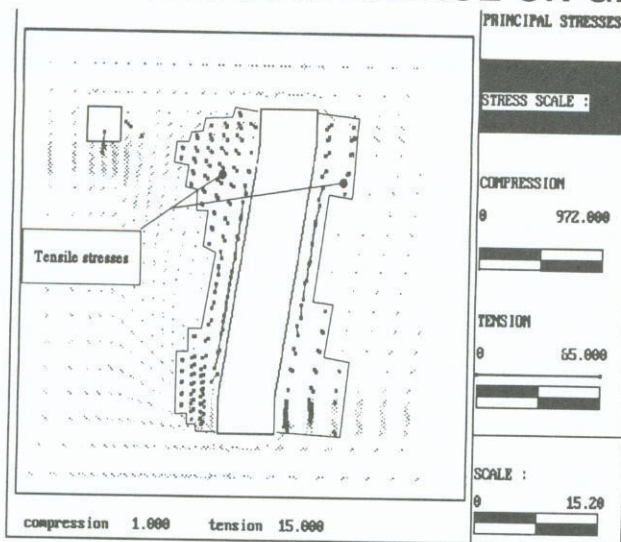


Figure 7. Delineation of potentially unstable zones

It is noteworthy that previous studies by the authors [12] showed that the Mohr-Coulomb and the Hoek-Brown rock failure criteria do not adequately predict the failure zones in the stope hanging wall. Comparison of stope cavity monitoring survey (CMS) results with numerical modelling showed that the caving/sloughing zones are best estimated by the tensile stress zones in the stope walls [12]. It is not surprising, however, that conventional rock failure criteria did not work here, as they are developed for the prediction of rock failure under compressive, confining stresses, e.g. in crown and sill pillar problems.

The software, e-z tools, computes the axial forces in the cables at each node along the cable. The results are plotted in Figure 8, whereby each cable is represented by 11 nodes. As can be seen, the axial force is zero at both ends of the cable. This is to be expected as the cable in this application (non-entry stope) is not anchored with an end plate. The collar (drift) end of the cable is also free, as the need for anchorage at this end is not needed. It can also be seen from Figure 8 that cables 1,2 and 3 appear to carry considerably more load than the lower stope cables no. 5, 6 and 7. The maximum computed loads are 20.7, 23.5 and 21.0 tonnes for cables 1,2 and 3 respectively.

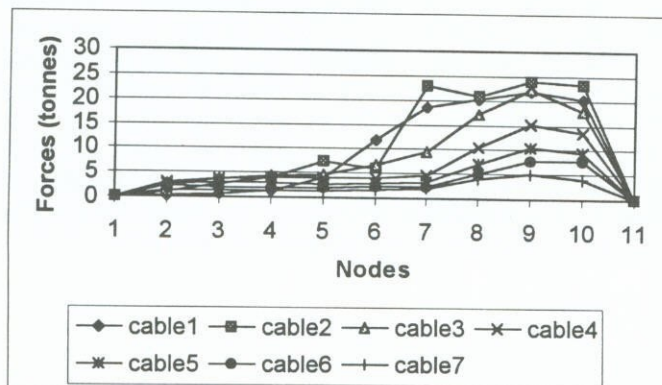


Figure 8. Axial force distribution in the cable bolts as computed by e-z tools

Tailgate secondary supports

Cable bolting is becoming widely used as secondary support in coal mines. The use of cablebolting in coal mines was initially developed in Australian coal mines and now it is used in United Kingdom, U.S.A and Canada. The cables are generally placed in critical areas (gate roads, belt roads and travelling roads) to assure safety in the mine. Resin-grouted cable bolts are now the most commonly used in coal mines, as resin grouts provide an excellent mechanical interlock thus creating a high degree of cable anchorage [3].

This case study describes the installation and evaluation of resin-grouted cables for secondary support in a longwall gate road in a Western U.S coal mine [13]. Cable support systems were designed to provide stability in a gate road that was utilized for two longwall panels, first as headgate and then a tailgate (Figure 9).

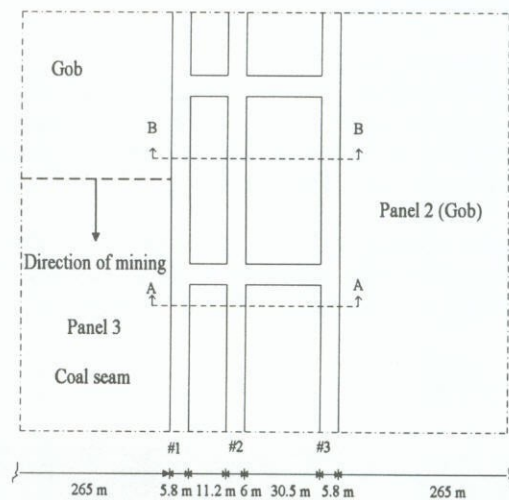


Figure 9. Plan view of the mining area under study

The mining depth is approximately 365 m and the initial mining height is 2.8 m. The abutment pillar designed to absorb the first panel abutment stresses and protect the integrity of the yield pillar measured 30.5 m wide by 40 m long. The yield pillar measures 11.2 m wide by 40 m long. The geology consisted of top coal, shale and sandstone. Rock mass properties of different materials are presented in Table 2.

Property	Coal	Shale	Sandstone	Gob
γ (MN/m) ³	0.012	0.0224	0.0256	0.016
E (MPa)	3378	14410	33784	1441
ν	0.4	0.37	0.3	0.40

Table 2. Material properties of the coal mine case study

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To model rock bolts and cablebolts in the mine two modelling scenarios, namely model A and B, were adopted. For model A, panel 2 is mined out and panel 3 is not (Figure. 10), which correspond to mining of panel 2 and only rock bolts were used. For model B, both panel 2 and panel 3 are mined out. Here, the cable bolts are used to support the tailgate (Figure.11).

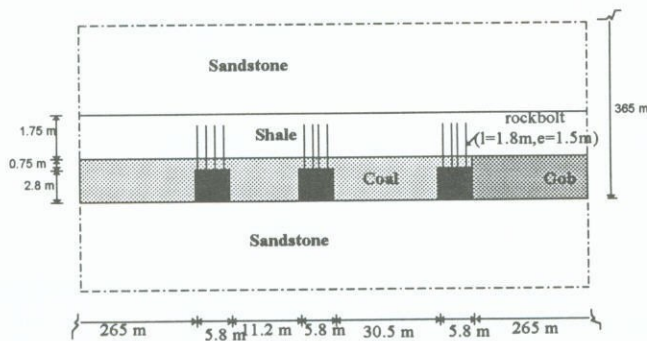


Figure 10. Coal mine case study - Model A

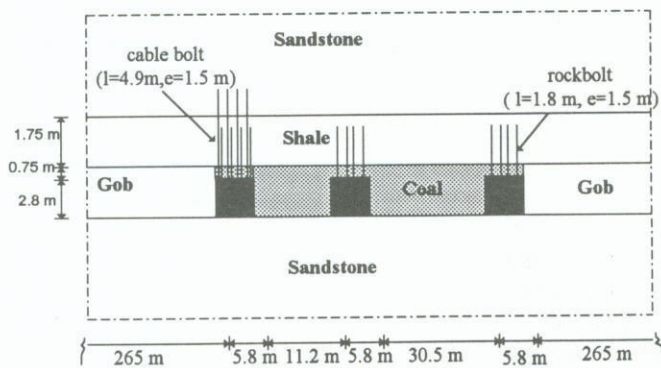


Figure 11. Coal mine case study - Model B

A finite element mesh was constructed with 2598, 4-node isoparametric elements (Figure 12). A preliminary analysis without bolts was done to examine the stress distribution in the mine and delineate the tensile stress zones in the roof. For the model A, both panel 2 (gob) and the tensile stress zone were modeled with a weak material ($E=1.44$ GPa, $\nu=0.40$). For the model B, panels 2 and 3 as well as the tensile stress zone in the roof were modeled with the same weak material (Figure 13).

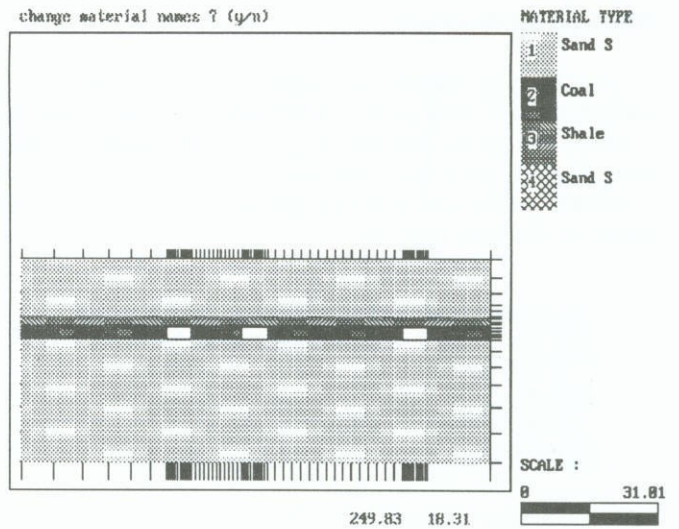


Figure 12. Finite Element mesh and geological materials modeled

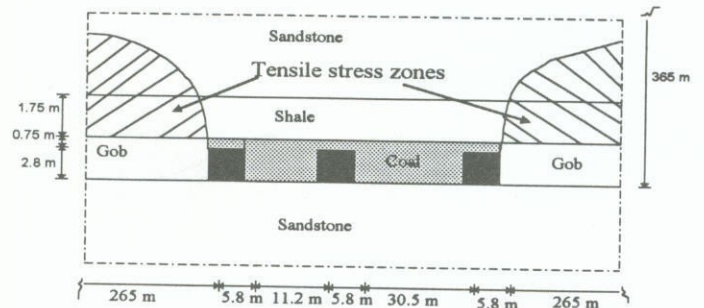


Figure 13. Delineation of tensile stress zones in the roofs of panels 2 and 3

The instrumented test area was initially supported with 1.9 m full column resin-grouted bolts as primary support on a 1.5 m row spacing as shown in Figure 10. Resin grouted cable bolts supports were installed after mining panel 2 as a secondary support with end bearing plates to support the tailgate (Figure 14) with a length of 4.9 m and 1.5 m spacing. Standard 7-wire, 1.52 cm diameter cables were used. These cables have a load bearing capacity of 26 tonnes, a modulus of elasticity of 200 GPa and are partially grouted (1.7 m resin). For modelling purposes, a shear bond stiffness (k) of 300 MN/m/m for the cable bolts in the grouted length was taken and for rock bolts $k=105$ MN/m/m [14].

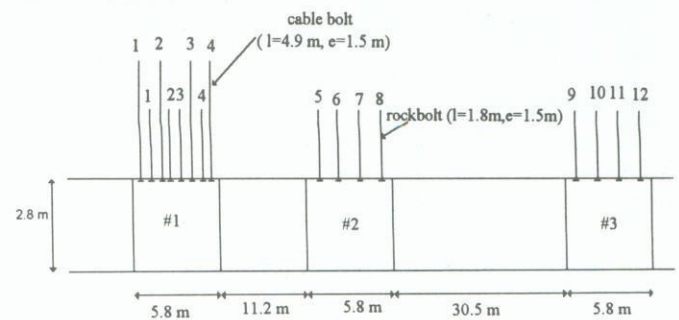


Figure 14. Location of cable bolts and rock bolts

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Discussion of the results

For the first analysis, the maximum forces in the bolts as computed by e-z tools are shown in Table 3. The forces decrease to reach zero at the free end (Figure 15). The maximum force computed is 5.3 tonnes. We see that the maximum force is mobilized by the rock bolt 12 which is near the panel 2, that is mined out and filled with gob.

Bolts	1	2	3	4
Forces (tonnes) Model A	5.0	4.1	4.1	5.0
Forces (tonnes) Model B	4.3	4.1	4.1	4.3

Table 3 . Maximum forces in the rockbolts

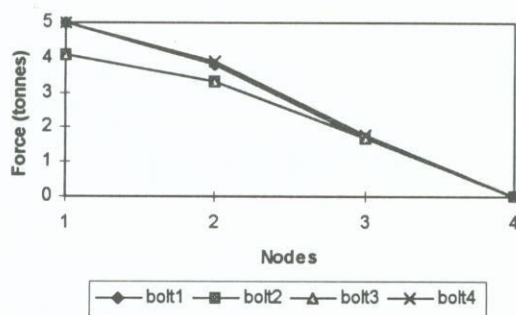


Figure 15 . Axial force distribution in the resin- grouted rock bolts (Model A)

For the second analysis, rock bolts and cable bolts were used. The maximum forces in the rockbolts are shown in Table 3 and in Table 4 for cablebolts. The load distribution in the rockbolts which support gates 2 and 3 (bolts 5 to 12) is the same as for model A and the maximum force is 5.3 tonnes (rock bolt 12) but for the gate 1, the load distribution is different and for the rock bolts the maximum force is 4.3 tonnes (bolts 1 and 4). For the cablebolts, the maximum force mobilized is 13.6 tonnes (cable 1) at the plated face and the minimum force is zero at the toe of the cables (Figure16). The monitoring results from measured field instrumentation indicated that the maximum cable bolt force was 14.8 tonnes [13].

Cables	1	2	3	4
Forces (tonnes)	13.6	13.5	13.5	13.5

Table 4. Maximum forces in the cables (Model B)

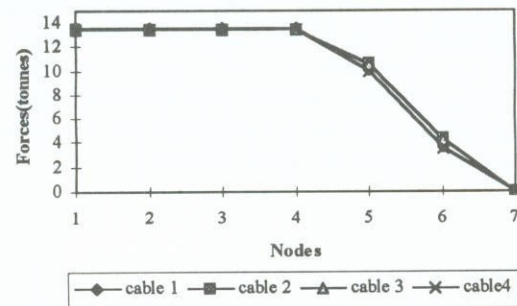


Figure 16. Axial force distribution in the cablebolts

CONCLUSION

A finite element modelling approach with cable bolt modelling capability is developed for numerical modelling of cable bolts. Two case studies were presented to illustrate the use of the modelling technique as a back analysis tool. The case studies are: stope hanging-wall support by cable bolts in hardrock mine and tailgate secondary supports in a coal mine. It appears from the first case study (hardrock mine) that hanging-wall cable bolts are quite effective, particularly those which are nearly perpendicular to the stope face (cables No. 1, 2, and 3) and that the cablebolting pattern could be optimized by moving more of the bolts towards the upper end of the stope. That would result in more evenly shared loads on the bolts and would reduce overall costs. From the second case study (coal mine), it is shown that the maximum cable bolt force is at the anchored end with forces reaching 5 tonnes in the resin-grouted rock bolts and 13.4 tonnes in the resin grouted cable bolts. The latter figure compares favourably with field measurement of 14.8 tonnes for the maximum axial force in the cable bolts. The proposed modelling technique, demonstrated here as a back analysis tool can be equally applied in the design of support.

ACKNOWLEDGEMENTS

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Roof Bolting Application in Longwall Mining in Indonesia and Japan

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ABSTRACT

The application of roof bolting technology to control the stability of roadways in coal mining within weak strata or under high ground pressure has improved progressively. One of several factors that affect the design of rock bolting is the immediate roof characteristics of the roadway. On the competent rocks, the mining conditions may indicate a high level of stability and a low density of bolting patterns may be adequate to control the ground stability. However, at a weak strata, the roadway will be affected by both horizontal and vertical stresses that may generate a high level of deformation in the immediate roof. This sensitivity of the roadway will require a mine plan design which consider the pillar sizes and sequences of extraction as well as a conservative reinforcement to maintain the integrity of the deforming weak roof strata.

Rock bolting technology has been used to maintain the stability of roadways in Indonesia and Japan for a few years. In this paper the overview of the application of rock bolting reinforcement in Ombilin Coal Mine, Indonesia and Miike Coal Mine, Japan is given.

INTRODUCTION

Rock bolting has become an established means of support world-wide over the past 80 years. This method has satisfied the need for an economic, safe and effective form of rock support, particularly in coal mines. Bolting became an economically efficient ground control method due to the high degree of mechanization, including the development of heavy duty machine, which are used for mining and tunneling.

The function of bolting is to bind together a discontinuous rock mass, such as a sedimentary rock made up a series of laminae, or rock containing natural joints or fractures, the blast-fractured rock around an opening excavated with explosives.

In longwall mining systems, the design of the gateroad support system plays an important role in the stability of the ground work, economics and efficiency. The support system in the gateroad is provided to control the excessive deformation of the roof, so that the requirement of the costly repair work and intensive man power may be minimized.

Standing support such as steel arch or square wood support have been traditionally employed in Indonesian and Japan Coal Mines. A few years ago, the roof bolting technology was introduced in longwall mining in both countries, to achieve high efficiency of the roadway drive work.

This paper presents an assessment of the ground stability of a gateroad in the Ombilin Coal Mine Indonesia and at the previous Miike Coal Mine in Japan, in which the bolting method as a support system is applied.

BACKGROUND

Ombilin Coal Mine

Ombilin Coal Mine is located in West Sumatra Province, Indonesia, as shown in Figure 1, and is owned by PT Bukit Asam. The mine consist of several open pits and an underground mine. The mine was started in 1992 by underground method. The total production in 1995 was about 1.1 million tons. Of the total, about 120,000 tons per annum is produced from the underground mine. Currently the underground mining activities are located at Sawahluwung area namely Ombilin II, in which this study was conducted.

Retreating longwall method is applied for winning the coal from underground, and the coal is excavated by means of fully mechanized system. The width of the panels varies from 70 m to 150 m, and the length can go up to 400 m. In this mining system the heavy duty roadheader machines are used in the drive works of the roadway.

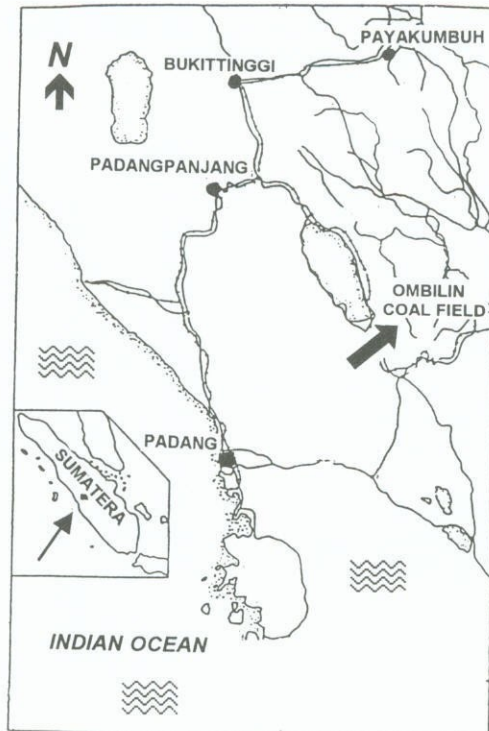


Figure 1. Location of Ombilin Coal Mine, Indonesia.

According to Northwest Resources Consultant Ltd the total mineable coal reserve in 1996, including seams A and C, are 15,509 million and 39,968 million tons, respectively, or for a total of 56,477 million tons. This coal reserve is calculated based on the minimum mineable seam thickness of 1.0 m.

In order to increase coal production, as it is in accordance with the National Policy, expansion of the Ombilin Coal Mine takes priority. Consequently, an underground coal mine project so-called Ombilin III is going to take place before 2002. The new Ombilin III Coal Mine will be at the area of Sigalut, and at present this mine site is under development stage.

Rock types

Rock types at Ombilin II, consists of Waringin and West Sugar, located in the Sawahlunto formation of the Ombilin basin. The rock type in this formation includes; coal, calcareous shale, claystone, mudstone, siltstone, shale and sometimes sandstone, and yet the coal bearing strata is dominated by the clay bearing rock.

The Sawahlunto formation where the Ombilin II is covered, has three coal seams, A,B and C. The average thickness of A coal seam is 1.9 m and varies from 1.0 m to 2.5 m. The thickness of the B coal seam, about 25 m below

the A seam, is about 1.4 m, and ranges from 1.0 m to 2.3 m. The deepest coal seam, C, is separated from the B seam by about 25 m and the average thickness is 5.47 m and varies from 1.0 to 18.0 m.

The rock floor of the coal seam varies from calcareous claystone to siltstone, and its thickness is about 1.5 m. Figure 2 shows the roof and floor lithology of A seam at the Panel IV, Ombilin II Coal Mine.

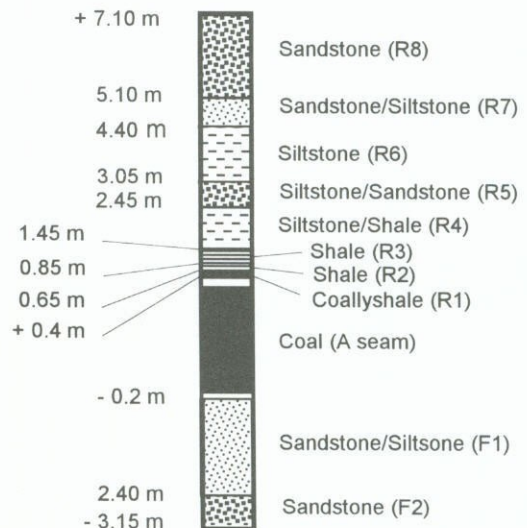


Figure 2. Roof and floor lithology of Panel IV Ombilin II Coal Mine

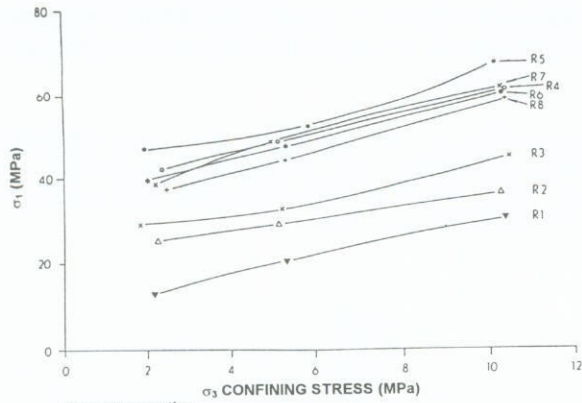
Characteristic of coal bearing strata

Lithologically, the coal bearing strata consists usually of siltstone, claystone, mudstone, shale and sometime sandstone and coalclay but predominantly siltstone. These types of rock can be categorized as weak rocks and they are characterized as follows :

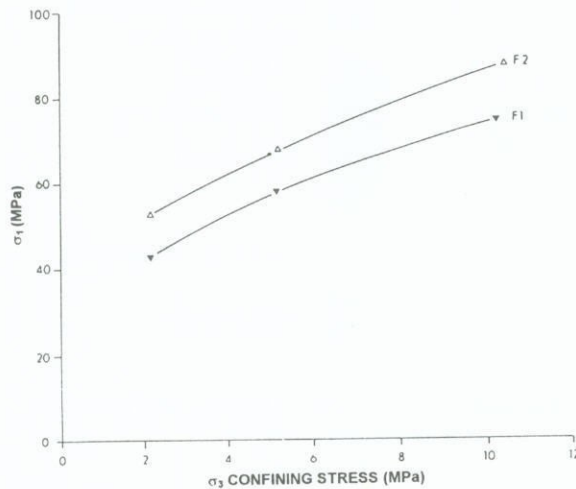
- they have high permeability
- sometimes consist of clay that may swell and has a slaking behaviour
- some parts are highly damaged due to their structural damage.

According to the RMR (1), the UCS of these types of weak rocks can be categorized as Class IV and V rock, which means that support is required immediately after excavation. These results of the triaxial tests of the roof at Panel IV indicate that the material in the immediate roof is structurally competent but in low strength as presented in Figure 3. The strength, however, does increase with the distance into the roof. The strength of shale may be deteriorated when contact with water. The slake durability tests showed that clay bearing rock such as shale exhibits a medium to high slaking behaviour.

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a) Roof properties



b) Floor properties

Figure 3. The results of the triaxial tests for the roof and floor of the maingate of Panel IV.

The UCS of these rock type varies from 20 to 50 MPa, as exhibited in Table 1.

Miike Coal Mine

Miike Coal Mine, the largest undersea coal mine in Japan was closed down in March 1997 due to adverse market conditions. During its operation, strata bolting method was applied in longwall gateroad (2).

Miike Coal Mine was located near Ohmuta City in the island of Kyushu as shown in Figure 4. The Colliery was a large undersea Coal Mine by domestic standards. The coal reserves covered a wide area under Fukuoka and Kumamoto Counties on Kyushu Island, and extended far out under Ariake Sea. The mining area still contains vast reserves, quoted at the end of September 1992 as 199.7 Mt (219.7 million short tons). The coal is mined by highly mechanized retreat longwall faces.

Table 1. Physical and mechanical properties of rock at Ombilin mine (after PTBA).

Parameter	Clay	Silt	Sand stone
- Density (nat), ton/m ³	2.64	2.51	2.26
- Density (sat), ton/m ³	2.69	2.58	2.4
- Dry density, ton/m ³	2.53	2.45	2.22
- Absorption, %	6.52	5.22	7.94
- Porosity, %	16.13	12.87	17.12
- Specific gravity	3.03	2.82	2.7
- Compressive strength, MPa	23.01	36.34	43.18
- Modulus elasticity, GPa	4.25	5.25	4.59
- Poisson ratio	0.36	0.3	0.35
- RQD, %	65	70	-
- Cohesion, Mpa	6.6	2.22	-
- Internal friction angle, °	44.43	55.08	-
- Spacing discontinuity, mm	60-200	60-100	-
- Condition of discontinuity	sliken sided	sliken sided	-



Figure 4. Location of the former Miike Colliery, Japan.



Geology

The formations in the Miike coal fields are of Pre-Tertiary, Tertiary and Quaternary age. The pre-Tertiary formations consist of granite and crystalline schists which are overlain unconformably by the Tertiary formations. The latter have a thickness of 700 m (2296 ft) and comprise of beds of sandstone intercalated with shales, conglomerates and coal seams. These seams are exposed in the northeastern district of Ohmuta City. The Quaternary formations, which have a maximum thickness of 400 m over the Tertiary formations under the Ariake Sea.

Within the Tertiary formations there are total of nine coal seams, three of which are workable. These are known respectively as the Second Upper Seam, Upper Seam and Main Seam in descending order. The principal dip of the seams is between 2° and 5° to the southwest - they go deeper as they go further off-shore.

Mining activities

In 1980 Miike Coal Mine produced 5.3 Mt (5.83 million short tons). But in 1991 production decreased to 2.3 Mt (2.53 million short tons). The reason for this reduction was primarily due to the widening price gap between domestic coal production and the imported coal and the national plan for domestic coal production (2). Under this difficult situation, Miike Colliery introduced a new longwall system to maximize coal face productivity. In this mining system, rapid drivage and effective maintenance of the gateroad are of critical importance for achieving safety, economy and efficiency of longwall mining. Standing support has been conventionally employed in the longwall gateroads. Along with standing supports, the strata bolting system was employed in order to increase the drivage performance and reduce the cost of drivage and the maintenance of gateroads.

GROUND CONTROL

Strata bolting technology is introduced as a roof supporting system in the Ombilin Coal Mine in order to sustain the rapid drivage system of the gateroad by the use of a heavy duty roadheader.

Prior to roof bolt installation, an initial investigation into the applicability of the roof bolting was conducted at the gateroad of Panel IV, of the existing Ombilin II Coal Mine. Extensometer was installed to determine the characteristics of the strata behaviour around the maingate called the Alpine Heading (AM-50) and tailgate called Dosco 3 Heading (MK 2A) of Panel IV. The roof of Alpine Heading shows a high level of stability. Since the Heading is narrow, 3.3 m wide, no significant deformation was noted within 0.5 m into the roof and the total displacement of 7 mm can be considered very low for the weak immediate roof strata. The roof of Dosco 3 Heading shows a very high level of

deformation. The displacements shown are relative to the normal mining roof and allow more than 800 mm of material to fall from the roof at the face prior to the installation of the instrumentation. The result shows that the deformation extends to 2.2 m above the roof line and exceeds 120 mm in magnitude. The majority of deformation occurs during the first 10 meters of face advance but with continued low levels of additional movement occurring with time. Majority of the deformation is occurring in the clay rich immediate roof strata. The deformational characteristics at various distances of face advance in the Alpine and Dosco 3 Headings from the extensometer site are shown in Figure 5. Figure 6 summarizes the total deformation occurring in the Alpine and Dosco3 Headings. A high level of stability is achieved in the Alpine Heading with only 7 mm of movement, with very low additional deformation with time. In the Dosco 3 Heading general stability was achieved after 10 days with 100 mm movement. However, low levels of movement continued at a rate of 2 mm per day.

The investigation also showed the influence of the depth of cover to roof stability. Under shallow depth cover, less than 160 m, the ground stability seem to be excellent in longwall workings, such as shown by the tailgate of the Panel III longwall which was in good condition even though it was separated by a small pillar from the previous longwall. Under greater cover, a higher level of deformation occurred such as that in Dosco 3 Heading and in Panel IX. As the depth of cover increased in the Dosco 3 Heading, conditions were seen to deteriorate, initially with central cracks developing and at greater depths, with shearing of the roof material and sag of the steel cross beams.

Historically, a trapezoidal three-piece-set with 3.0 m wood or 4.2 m steel beams at 1 m interval was employed to maintain the stability of the gateroad. However, with the increase of the drivage rate, the strata bolting system is now utilized to support the roadway. So far the strata bolting method in the Ombilin Coal Mine has been used to support the gateroad development of the Panel VI at Ombilin II Mine. The bolts configuration that are used consisted of five fully grouted bolts with 2.4 m long on one w-strap and 0.8 m interval are employed in the maingate and tailgate as shown in Figure 7. The bolts are installed in 32 mm (1.25 in) diameter holes wet-drilled by hand-held rotary type drilling machine called "Wombat". Tell-tales monitoring systems are used to control the roof deformation. The results showed that there are no significant displacement occurred at the roof of the adjacent drivage face. In the future, the bolting system is expected to be employed as well in the roof supporting method in the new Ombilin III Mine.

In 1993, Matsui (3) conducted the study in Miike Coal Mine, in order to investigate the bolting performance. The study site was located in the Second Upper seam. The depth cover in this area about 430 m below sea level and the thickness varies between 4.0 m and 4.3 m. The seam is overlain and underlain by sandstone, but locally underlain by shale. Mechanical properties of these rocks and coal

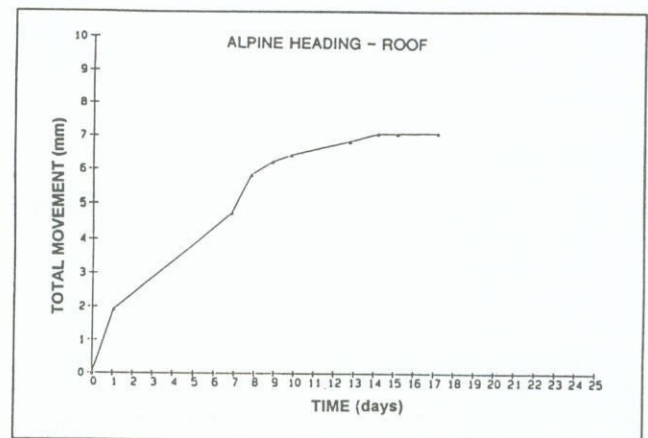
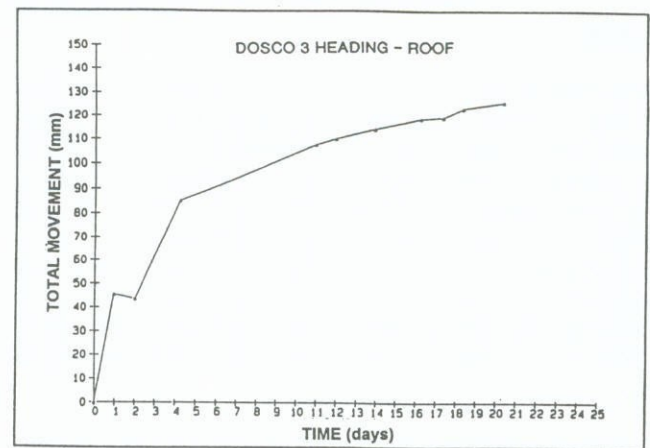
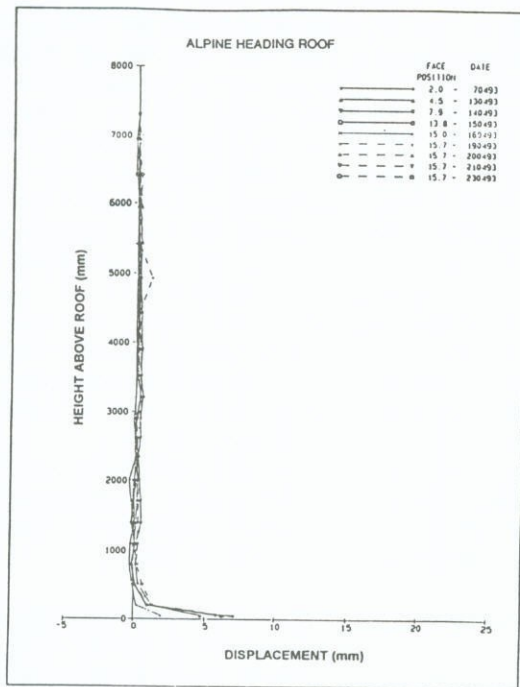


Figure 6. Summary of the total deformation occurring at the Alpine and Dosco3 Headings.

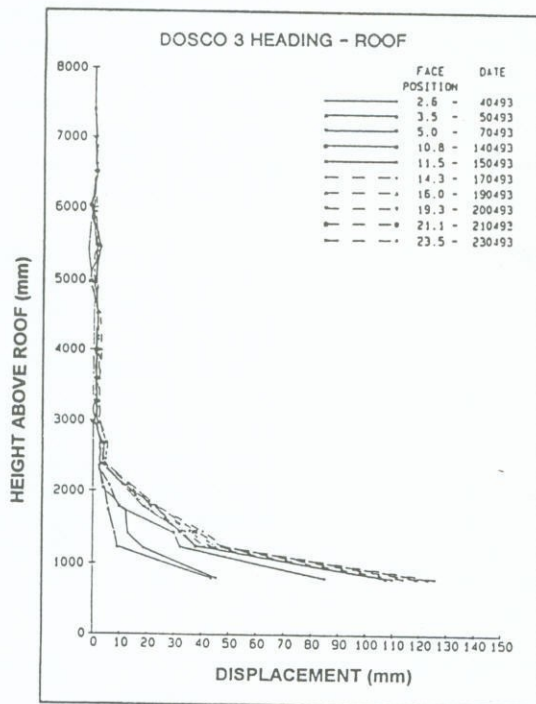


Figure 5. The results of extensometer measurement at Alpine and Dosco3 Headings.

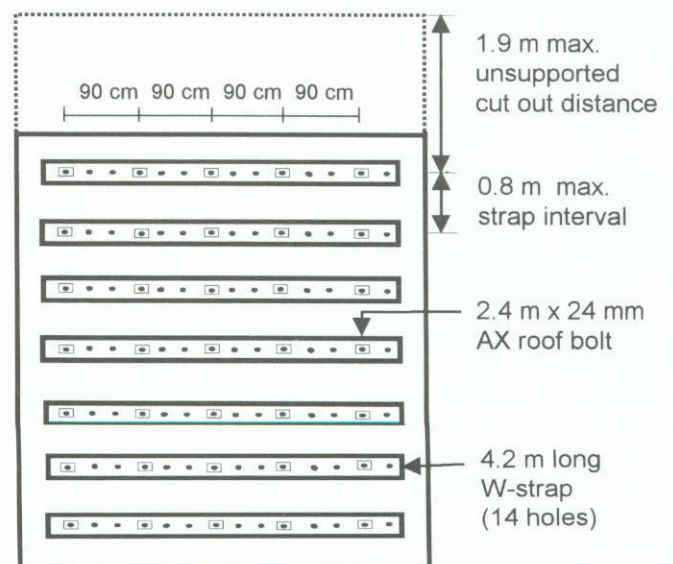


Figure 7. Roof bolt configuration at Ombilin Coal Mine

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deteriorate greatly in the presence of water.

The investigation was conducted in the tailgates of the longwall Panels. The faces were 150 m wide and the mining height was about 2.8 m, leaving the un-worked floor coal 1.5 m thick. The rate of the advance was approximately 10 m/day. The mining system applied to this face was the powerful, more efficient and mechanized mining equipment (4). The gateroads were driven by a roadheader of the MRH S-65 (Mitsui Miike Machinery Ltd.), and supported by a trapezoid shaped three-piece set with a 4.2 m steel beam and two 3.0 m wooden props at 1.0 m centre, as shown in Figure 8. The supports were lagged and tied with wood. As an additional support in the tailgates, strata bolting by means of resin grout steel bolt and wooden dowels was employed. Strata bolting was not installed immediately in front of the faces, but several tens of meters ahead of the active longwall face. Under poor strata conditions, supplemental wooden props along the pillar were also erected ahead of the active face. These gateroads were developed utilizing 25 m wide diamond-shaped pillars.

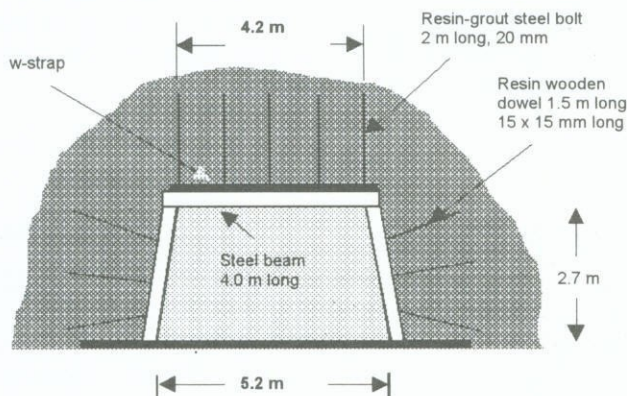


Figure 8. Dual support system at Miike Coal Mine.

The second investigation site was located in the tailgates at the Upper seam at a depth of about 460 m below sea level and with the thickness of about 2.7 m. The seam was overlain and underlain by shale similar to the previous study site, shale will deteriorate with the presence of water. The mining operation in this site is influenced by the interaction effects from the previous longwall mining in the Second Upper seam. Roadway was driven by a drill and blast system and supported by a trapezoidal beam three-piece set steel with a 3.8 m steel beam and two 3.0 m steel props at 0.6 m intervals. The standing supports were tied and lagged with wood. Three 2.0 m long resin-grouted steel bolts were installed in the roof at 1.2 m intervals between the supports immediately after the excavation. Three 1.5-m resin-grouted wooden dowels were also installed in the sidewalls at 0.6m intervals.

The results of the studies are presented in Figures 9 and 10. Figure 9 presents the vertical closure of the tailgate

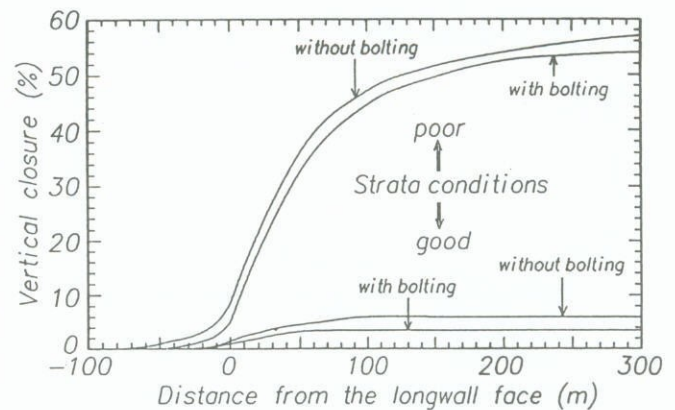


Figure 9. Vertical closure due to longwall mining.

during the adjacent longwall mining in the study site No. 1. In poor and good strata conditions, vertical closure reduced by 3 % with the application of strata bolting. In good strata conditions, the bolt maintained the stability of the roof although great deformation occurred on the roadway. Figure 10 illustrates the result of the extensometer measurement in which strata bolting clearly reduces the roof displacement within 2m above the roofline.

DISCUSSION AND CONCLUSION

Considerable advances have been achieved in rock bolting technology in recent years, due to of their benefit in ground control and in terms of economics. The roadway stability in longwall mining in many countries was controlled successfully by the application of strata bolting. Experience in Miike Coal Mine demonstrated that the bolting system has performed a prosperous ground control method of the weak strata under high ground pressure whereas, in Ombilin Coal Mine, the strata bolting has been considered to be employed as a primary ground control system of the roadway.

Although the depth of cover in Ombilin Coal Mine is much less than that in Miike Colliery, the recorded roof deformation was much larger, due to its weak coal bearing strata. In this instance, the roof bolting applicability that installed in the gateroad of Panel VI of Ombilin II Mine has shown good progress. In addition, the minimum time consumed in the bolting installation, about 25 minute per w-strap, has made roof bolting one of the viable alternatives of roof support along with the applicability of the heavy duty road header in Ombilin Coal Mine.

Despite these situations, it is quite important to determine the method of choosing the optimum parameters of bolting patterns for a given strata condition, from the point of view of safety and economics and efficiency. Several parameters that may be important to consider are the type of anchorage, bolt length, bolt diameter, bolt density and

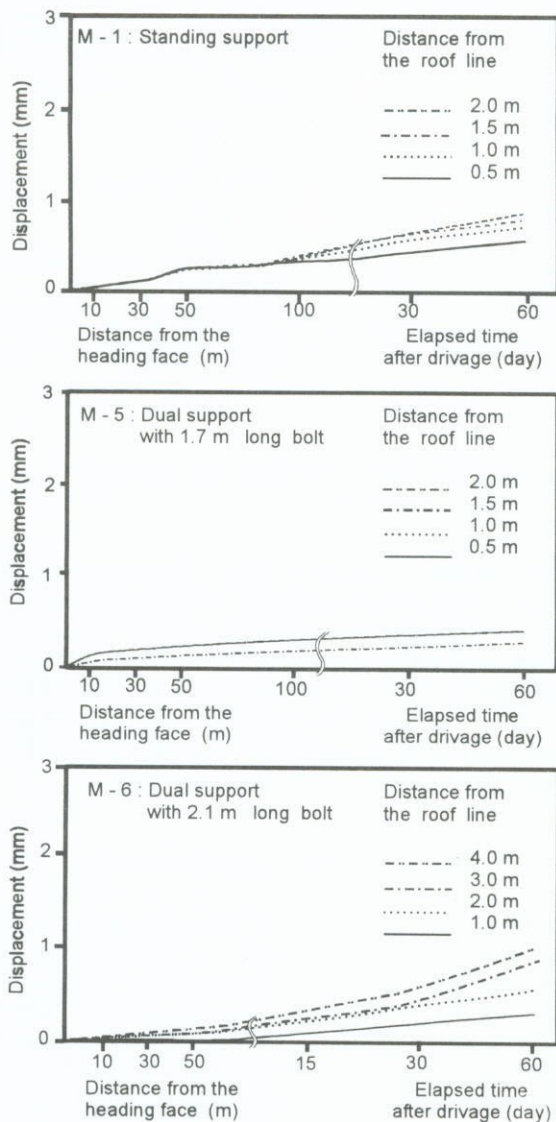


Figure 10. Extensometer measurement during gateroad development at Miike Coal Mine (Matsui, 1993b).

quality. These problems could be solved by accurate preliminary investigation. However, post installation monitoring system is of significance to determine the bolting performance.

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Ground Control Design for Multiple Seam Mining Using Finite Element Analysis

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ABSTRACT

Seam interaction in multiple seam mining has significant effect on entry stability. The remaining pillar in an old working usually creates a high stress concentration zone while the gob creates a stress release zone. A good panel layout should take advantage of the stress release effect created by old workings. This paper presents two case studies of multiple seam mining. The Jennmar Finite Element Analysis System (JFEA) is utilized to analyze the seam interaction effect and identify the stress concentration and relief zones. Panel layouts are designed based on the results of the finite element analysis. Appropriate roof support systems are also designed for different seam interaction areas.

INTRODUCTION

Ground control practices in U.S. coal mines have shown that seam interaction in multiple seam mining can cause serious ground control problems such as roof falls, pillar failure, and floor heave. To maintain ground stability, panel layout should be designed in such a manner that high stress concentrations are avoided. In those areas where seam interaction cannot be avoided, the pillar size and roof control method must be carefully designed. Jennmar Corporation has conducted a number of the ground control designs for multiple seam mining utilizing finite element analysis. Two of them will be presented in this paper. The first case is a room and pillar operation and the second is a longwall mine.

MINE A – ROOM AND PILLAR MINING

Geological and Mining Conditions

Mine A is currently mining the "F" seam. The Taggart seam, which is 150'-200' below the "F" seam, was

previously mined. The overburden depth is 400'-1300'. The section layout in the current mining area is shown in Fig. 1. The pillar is 35' x 60' in the sections and 60' x 80' in the Mains. The mining area overlaps the remaining pillars and gob of the Taggart seam.

Historically, pillar sloughage and roof problems were encountered in the area that overlapped the remaining pillar in the Taggart seam. To analyze the seam interaction effect, an underground examination was conducted in the current mining area shown in Fig. 1. The 1 Left and 2nd Left panels were developed successfully. However, during pillar extraction, the first panel was able to extract through Pillar 4, and the second panel had to stop over the Pillar 4 area due to pillar and roof failures. The third panel was abandoned during development due to significant pillar failure caused by seam interaction over the Pillar 5 area. In the Mains, although no serious roof problems were experienced, significant rib sloughage was experienced in the area overlapping the barrier pillar in the Taggart seam.

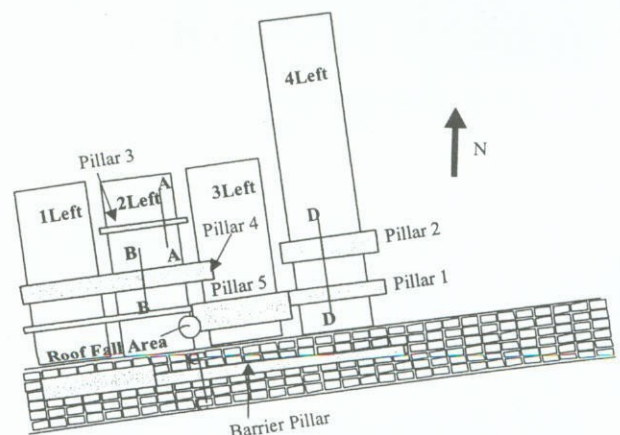


Fig. 1 Panel layout in current mining area and remaining pillars in lower seam

Seam Interaction Analysis

To analyze the seam interaction effect, finite element analysis was utilized to simulate the remaining pillar effect on the "F" seam mining.

Pillar 3:

Shown in Fig. 2 is the finite element model for Pillar 3 along cross section A-A.

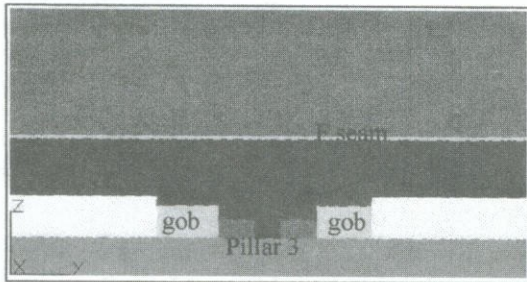


Fig. 2 Finite element model for Pillar 3

The vertical stress distribution shown in Fig. 3 clearly indicates that the remaining pillar in the Taggart seam creates a "stress bulb" and the "F" seam is in the influence zone of the bulb.

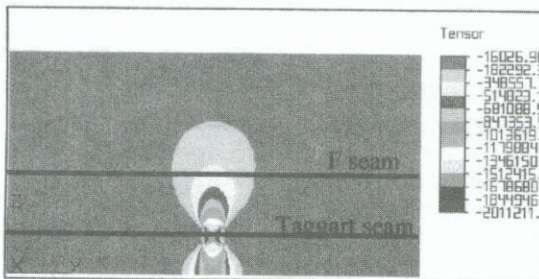


Fig. 3 Vertical stress distribution before Pillar 3 yielded

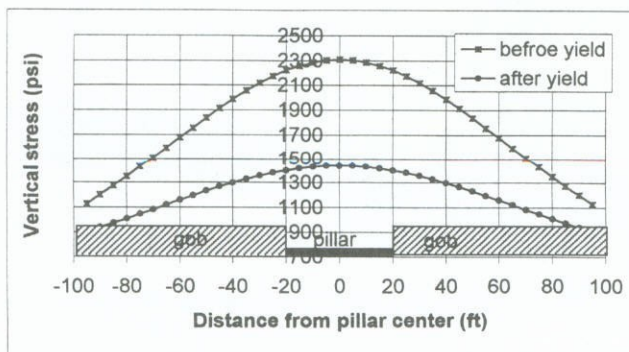


Fig. 4 Stress profile on F seam

The stress profile in the "F" seam is shown in Fig. 4. It can be seen that the maximum stress over the pillar center is 2300 psi. This value is three times the in-situ stress. However, this is only true if the remaining pillar in the Taggart seam is not yielded. From finite element analysis, the average stress on the remaining pillar is 11,057 psi and the pillar strength is 4932 psi, which equates to a safety factor of 0.45. Therefore, it can be concluded that this pillar has yielded. An additional model was run to simulate this yield condition. The stress distribution shown in Fig. 5 indicates that the "stress bulb" still exists, but is much lower in magnitude. Referring back to Fig. 4, it can be seen that the maximum stress over the pillar center in the "F" seam is 1450 psi. This value is much lower than before the remaining pillar in the Taggart seam yielded.

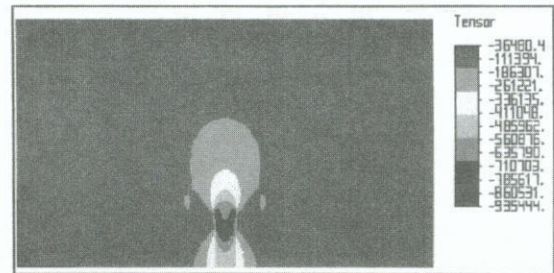


Fig. 5 Vertical stress distribution after Pillar 3 yielded

It can be concluded that Pillar 3 in the Target seam is yielded and has minimal effect on "F" seam mining. The 2 and 3 Left panels have been successfully developed and retreated over the Pillar 3 without any ground control problems.

Pillar 4:

Shown in Fig. 6 is the vertical stress distribution created by Pillar 4. The average stress on Pillar 4 is 9166 psi and strength is 9540 psi. The safety factor of the remaining pillar is 1.04. Therefore, the pillar has not yielded. The stress profile on the "F" seam (Fig. 7) indicates that the maximum stress over the center of Pillar 4 is 3200 psi. This value is much higher than the in-situ stress. Therefore, "F" seam mining will be affected significantly. Underground observations indicated extreme rib sloughage and roof problems in this area. The 2nd Left panel had to be abandoned due to pillar and roof failures caused by seam interaction.

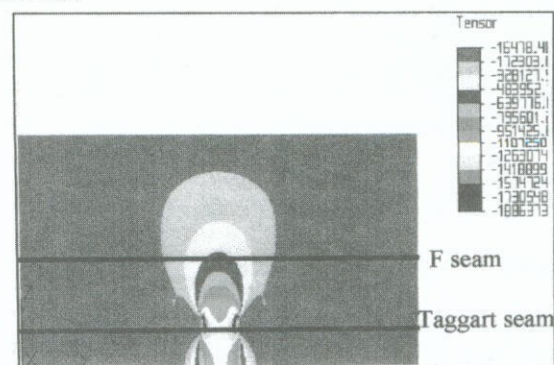


Fig. 6 Vertical stress distribution (Pillar 4)

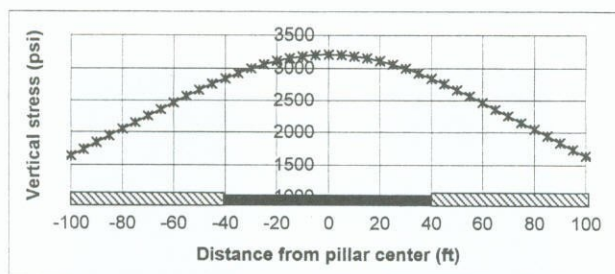


Fig. 7 Vertical stress profile on F seam (Pillar 4)

Barrier pillar:

Shown in Fig. 8 is the finite element model for the barrier pillar. The width of this pillar is 60', interburden is 150', and overburden is 1000'. The left side of the barrier pillar is gob and on the right side are a series of 40' remaining pillars.

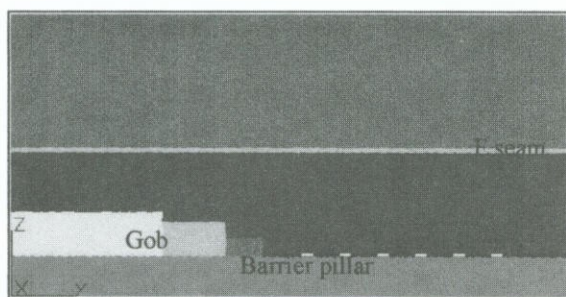


Fig. 8 Finite element model for barrier pillar

Shown in Fig. 9 is the vertical stress distribution.

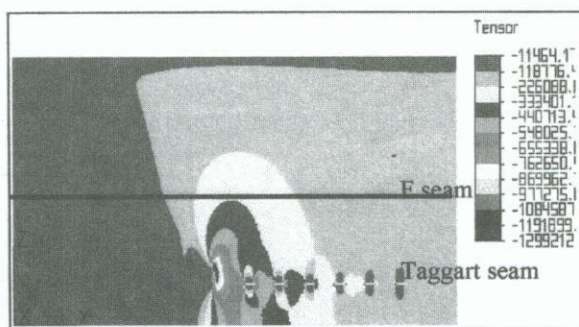


Fig. 9 Vertical stress distribution (barrier pillar)

The stress profile in the "F" seam shown in Fig. 10 indicates that the maximum stress is 2100 psi, which is almost two times the in-situ stress. Therefore, the "F" seam will be affected. However, due to the larger pillar size in this area (60 x 80'), no roof problems were incurred, but significant rib sloughage was observed.

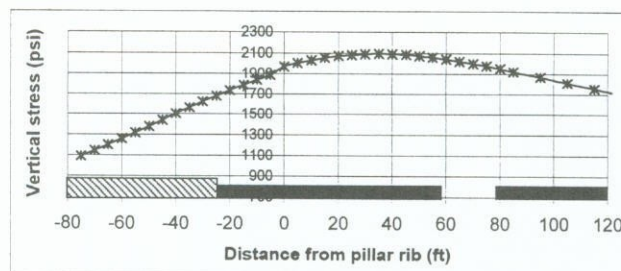


Fig. 10 Vertical stress profile on F seam (barrier pillar)

Analysis of the Future Panel in Current Mining Area

The 3 Panel has been abandoned due to roof problems related to seam interaction. Underlying the 4 Panel are two major pillars (Pillar 1 and Pillar 2) which may cause roof problems.

Seam Interaction

Shown in Fig. 11 is the finite element model for 4 Left panel along cross section D-D as shown in Fig. 1.

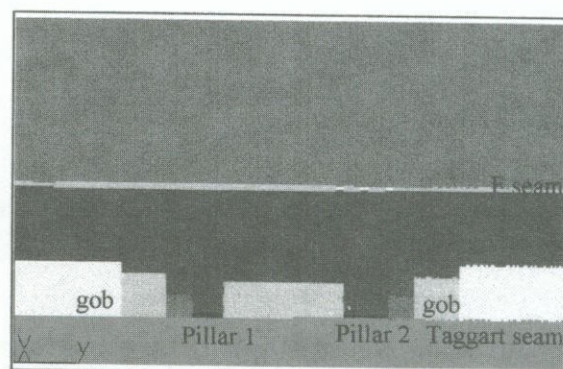


Fig. 11 Finite element model for 4 Left panel

The vertical stress distribution shown in Fig. 12 indicates that the "stress bulbs" created by the two remaining pillars in the Taggart seam reach the "F" seam level. Fig. 13 shows the stress profile on the "F" seam with a maximum value of 2600 psi. Based on the experiences in 3 Left panel, this stress concentration could cause a pillar failure if the pillar is not properly sized. Therefore, the pillar design in "F" seam must be based on the seam interaction.

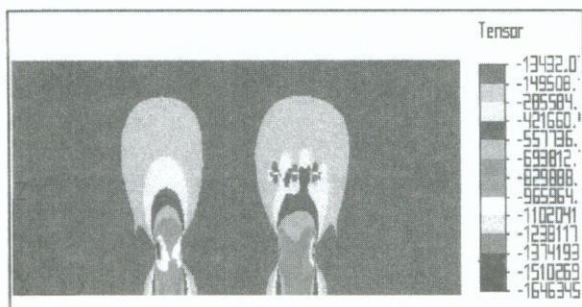


Fig. 12 Vertical stress distribution for 4 Left panel

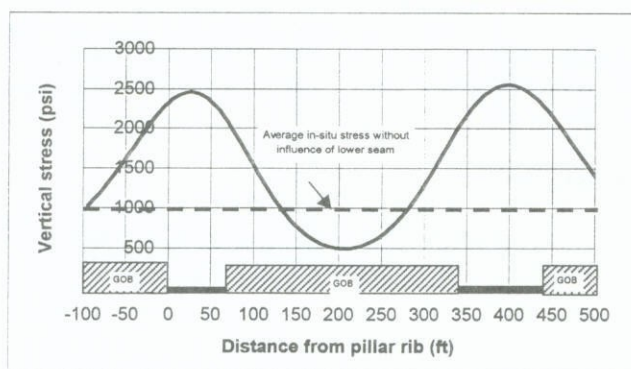


Fig. 13 Vertical stress profile on F seam (4 Left panel)

Pillar Design

Starting from the current pillar size (35'), a series of finite element models were run with different sizes. The results are listed in Table 1.

Table 1 Safety factors the different sizes of pillars

PILLAR SIZE (FT)	STRENGTH (PSI)	STRESS (PSI)	SAFETY FACTOR
35	4356	5197	0.84
40	4932	4976	0.99
45	5508	4804	1.15
50	6084	4667	1.30
55	6660	4555	1.46
60	7236	4461	1.62

In consideration of pillar extraction, the minimum safety factor should be 1.3. Therefore, in the 4 Left areas that overlap the pillar in the Taggart seam, the minimum pillar width should be 50'

Roof Support Design

Finite element analysis indicates that a high shear stress develops on the roof corner. Also, significant tensile stress occurs at mid-span of the entry due to the seam interaction effect. To control the shear and tensile stress, supplemental support is designed as shown in Fig. 14. A cable truss system combined with a cable bolt is recommended in the "stress bulb" influence area.

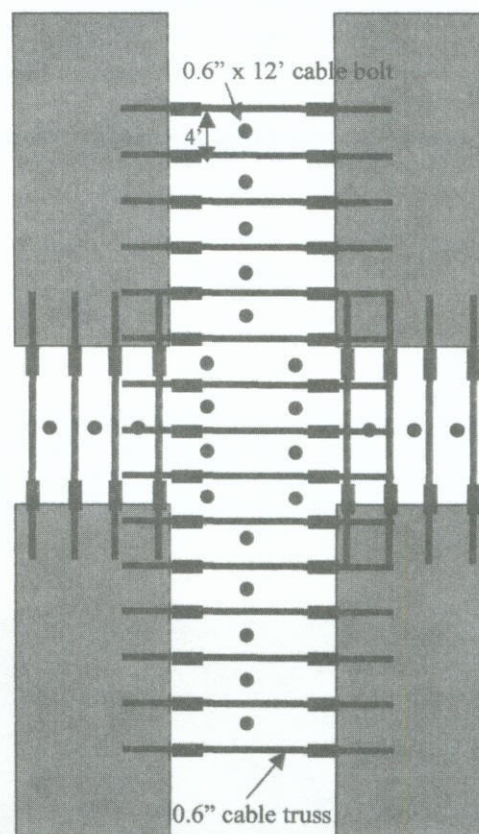


Fig. 14 Recommended supplemental support plan

MINE B – LONGWALL MINING

Geological and Mining Conditions

Mine B operates a longwall in the Lower O'Connor seam under 1000' of overburden. The Upper O'Connor, which is 40' above the Lower O'Connor seam, has been mined out. As shown in Fig. 15, part of the 5 Left headgate was developed under the un-mined dike zone of the Upper O'Connor seam. Due to the seam interaction effect, extreme rib sloughage occurred during headgate development. Although the pillar was enlarged to prevent sloughage, the condition did not improve. The roof strata and rock mechanic properties for Mine B are listed in Table 2.

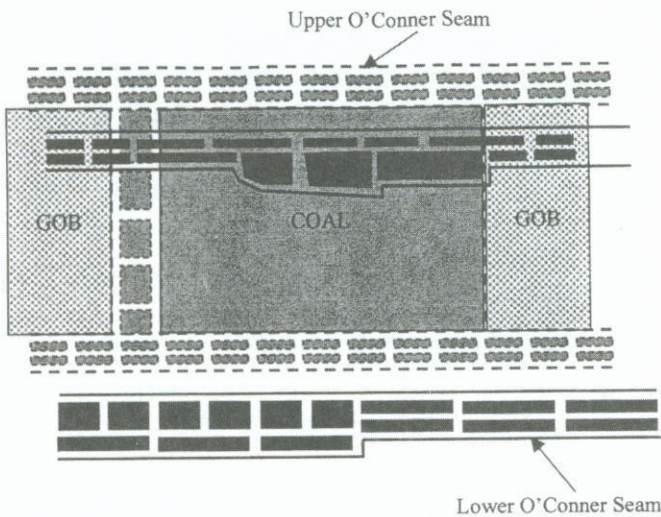


Fig. 15 5 Left panel layout

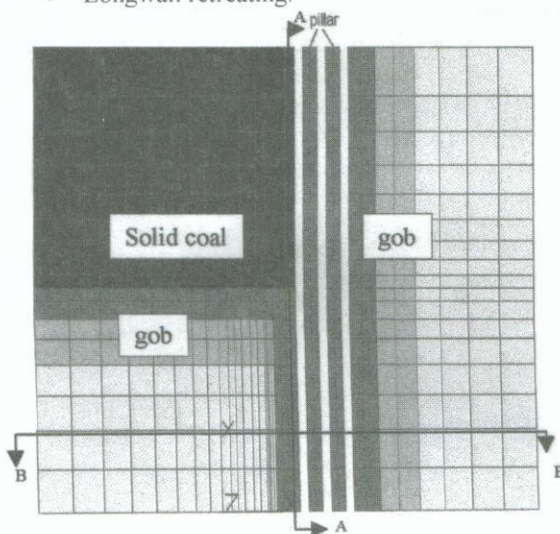
Table 2 Rock Mechanic Properties

STRATA	YOUNG'S MODULUS (X 10 ⁶ PSI)	UNIAXIAL COMPRESSIVE STRENGTH (PSI)	TENSILE STRENGTH (PSI)
Sandstone (upper seam roof)	200	8910	510
Upper seam	0.22	1710	60
Sandstone (upper seam floor)	1.68	9120	560
Shale (lower seam roof)	1.68	6460	330
Lower seam	0.35	2160	140
Shale (lower seam floor)	1.17	6640	580

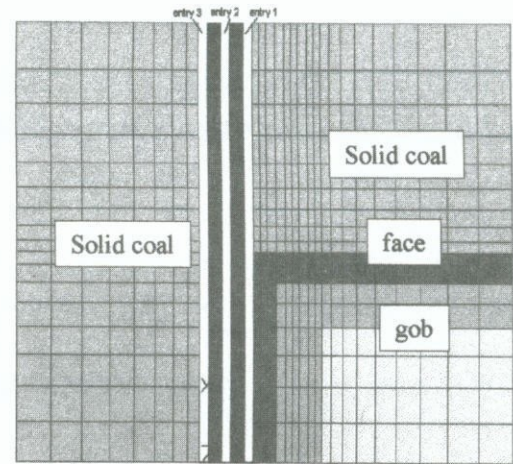
Finite Element Analysis

To analyze the seam interaction effect, a three-dimensional finite element model was constructed as shown in Fig. 16. The analysis consisted of a three phase process:

- Pre-mining stress analysis.
- Entry development.
- Longwall retreating.



Plan view at Upper O'Conner seam level



Plan view at Lower O'Conner seam level

Fig. 16 3-Dimensional finite element model

Pre-mining Stress Analysis

In this phase, the Upper O'Conner seam was mined out with coal left in the dike zone. The 5 Left headgate has not yet been developed.

The vertical stress distribution on the Lower O'Conner seam is shown in Fig. 17. It clearly shows a high stress concentration zone on the Lower O'Conner seam around the edge of the unmined Upper O'Conner seam. Shown in Fig. 18 is the stress profile along cross sections A-A and B-B.

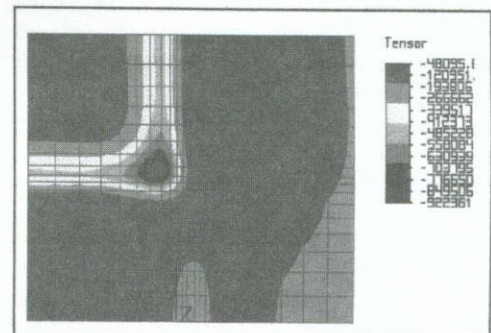


Fig. 17 Vertical stress distribution on the Lower O'Conner seam

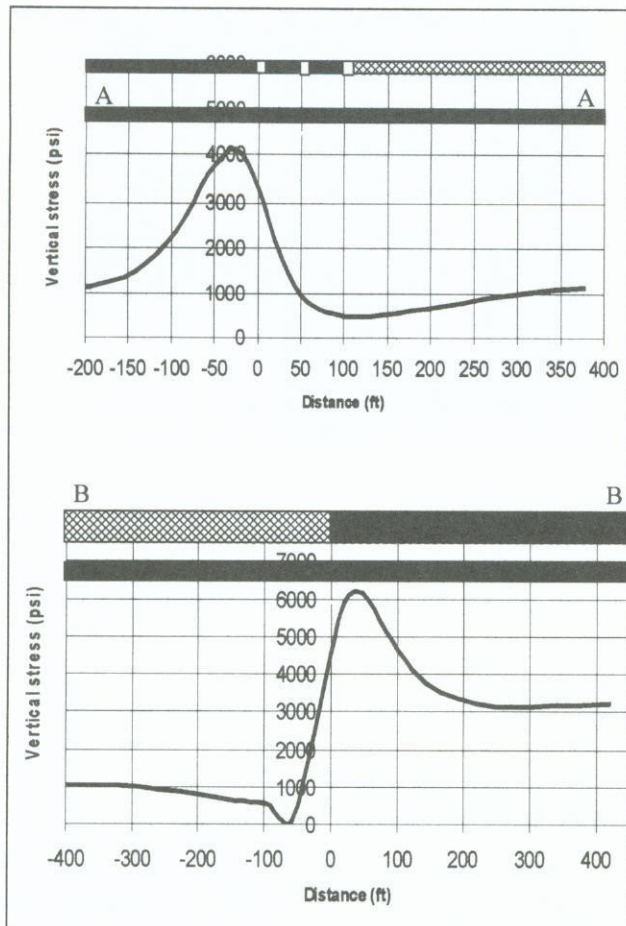


Fig. 18 Stress profile along cross sections A-A and B-B

The influence zone of the unmined Upper O'Conner seam can be identified from Fig. 18. The maximum stress of 6000 psi occurs underneath the corner of the unmined upper seam. The gate entry in the lower seam should be placed in the stress relief zone. However, unfortunately the 5 Left gate entry was placed in the stress concentration zone.

Stress Analysis During Headgate Development

In this phase, the 5 Left headgate has been developed under the coal block left in the Upper O'Conner seam.

The vertical stress distribution on the lower seam is shown in Fig. 19. The headgate is in a high stress concentration zone. Due to excessive yielding of the pillars, stress on the pillar is released and transferred to both sides of the solid coal. This can be clearly seen from the vertical stress profile shown in Fig. 20.

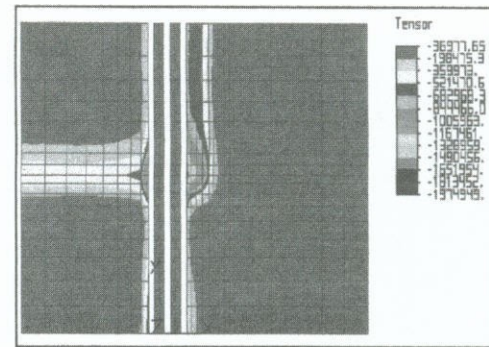


Fig. 19 Vertical stress distribution on the Lower O'Conner seam

This analysis indicates that due to the pre-mining high stress concentration and the transferred stress resulting from pillar yielding, rib sloughage on the solid coal side can be expected.

Stress Analysis During Longwall Retreat

In this phase, the longwall is approaching the edge of the dike zone and the headgate is exposed to the frontal abutment pressure. The results of the finite element analysis are presented as follows:

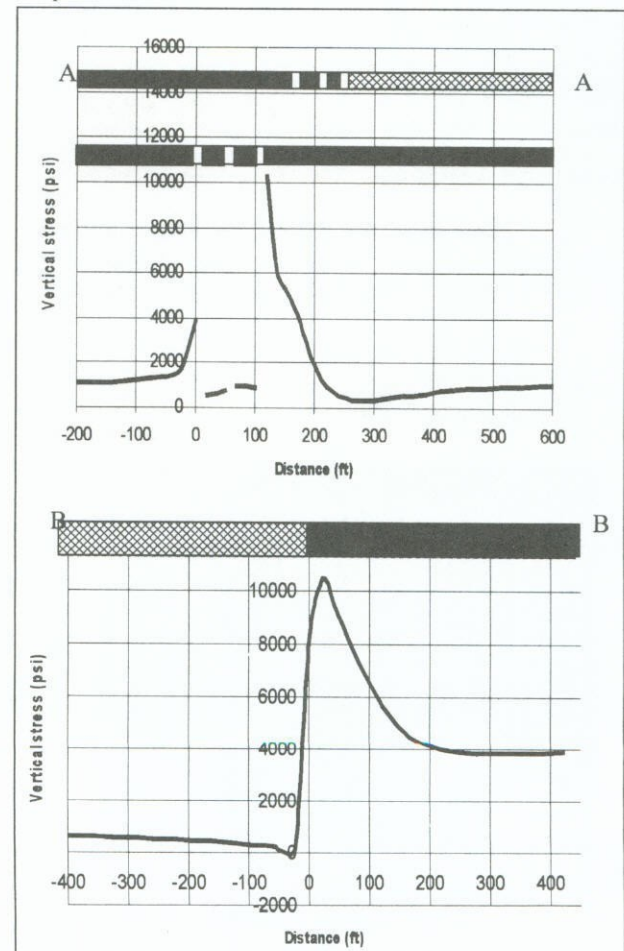


Fig. 20 Stress profile along cross sections A-A and B-B

Vertical Stress

The vertical stress distribution on the lower seam is shown in Fig. 21. Vertical stress profiles for the lower seam level along cross section B-B and A-A are shown in Fig. 22. On the panel rib, the maximum stress is 10,500 psi. This value is 3500 psi higher than the development stage. More rib sloughage would be expected.

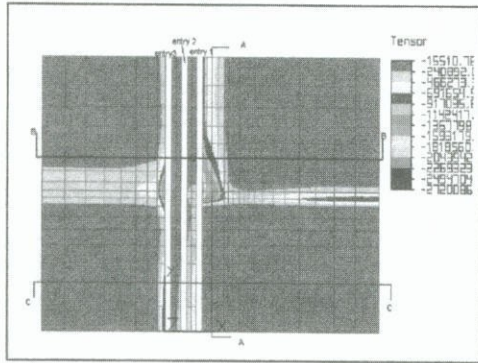


Fig. 21 Vertical stress distribution on the Lower O'Conner seam

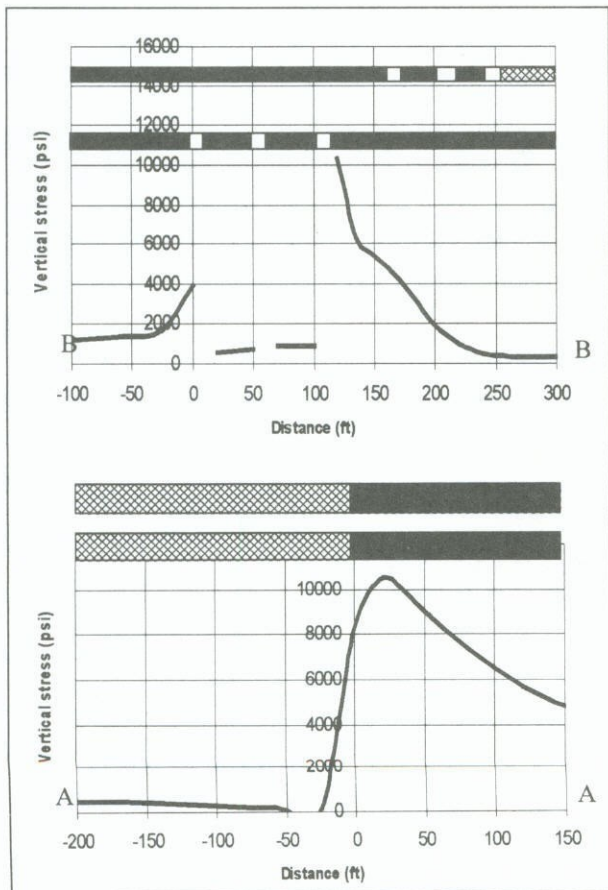


Fig. 22 Stress profile along cross sections A-A and B-B

Shear Stress in the Roof

The 3-D shear stress (Von Mises Stress) distribution in the entry roof along cross section B-B (outby the face) is shown in Fig. 23. Entry 1 (belt entry) has the highest shear stress of 6335 psi at the panel side. The shear stress in the roof of Entry 2 and 3 is 1609-2284 psi. Therefore, failure in the form of roof cutter may occur in the belt entry outby the face.

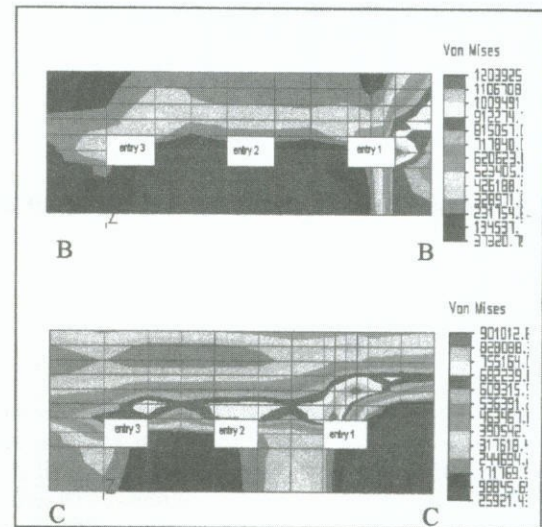


Fig. 23 Shear stress distribution along cross sections B-B and C-C

Fig. 23 also shows the 3-D shear stress distribution along cross section C-C (inby the face). The maximum shear stress is 5244 psi in Entry 2 and 4737 psi in Entry 3. Therefore, the entry roof will gradually deteriorate inby the face as it advances.

Supplemental Support Design

Based on the stress analysis, it can be predicted that further rib sloughage on the panel side and possible roof cutter will be encountered during longwall retreat. The cable truss and cable bolt support system was recommended as shown in Fig. 24. The cable truss has proven to be the most effective way to control roof cutter and transfer load over the pillar and panel rib. In consideration of excessive rib sloughage, the angle bolt was 12' in length with a 45° angle. The cable bolt was 16' or more in length.

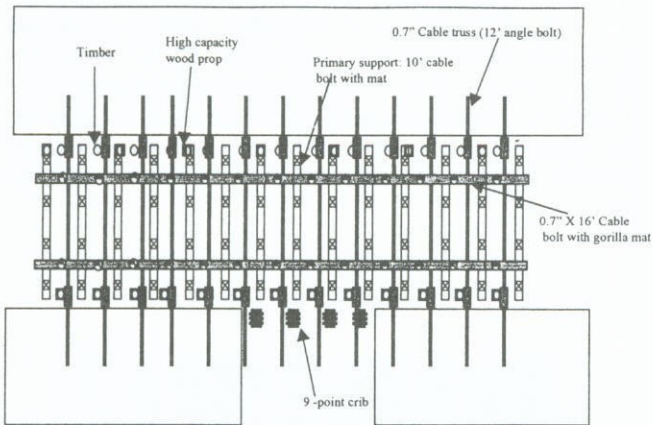


Fig. 24 Supplemental support plan for 5 Left headgate

CONCLUSIONS

The case studies have shown that seam interaction has significant effect on ground control in multiple seam mining. A good mining plan should take advantage of stress relief zones created by seam interaction. Otherwise, additional roof and rib support is required. Finite element analysis is an effective tool in analyzing multiple seam interaction and the design of appropriate support systems.



Pillar Design Issues for Underground Stone Mines

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ABSTRACT

Underground stone mining represents an emerging sector for the U.S. mining industry. As this expansion takes mines under deeper cover and as more efficient mining methods are utilized, adequate stone pillar design methods will become more important. It is the purpose of this paper to examine current design practices and to discuss issues for safe mine layouts so that a rational first approach towards balancing the demands for increased production can be weighed against increased risk to worker safety from rib instabilities and pillar failures. Seventy-two stone mine pillar designs were examined. Pillars with width-to-height ratios below 1.5 and subjected to excessive stress levels appear more likely to fail. When width-to-height ratios fall below 1.0, defects in the pillars, such as through-going discontinuities, can have a significant influence on stability. Discontinuity persistence, dip, material properties, and orientation are important factors controlling pillar strength. The influence of discontinuity dip, a characteristic easily identified in the field, was examined so that its presence could be accounted for in developing generalized guidelines for pillar design.

INTRODUCTION

During the last three years the number of active underground stone mines in the U.S. has ranged between 90 and 100. This number is expected to increase as the crushed stone industry responds to growing demands for their products (Anon, 1998). Additionally, production at underground stone mines is expected to increase above its current level of approximately 66 million metric tons (60 m.st) per year as more of the industry moves towards the unique advantages of underground mining. Parker (1996) identified three advantages for underground operations: 1) surface developments, zoning laws, and environmental concerns are often less of an issue, 2)

stripping and restoration requirements are eliminated, and 3) additional reserves are often available beneath the quarry floor, under pit slopes, or under adjoining property. There are also the added benefits of: 1) working in a constant underground climate rather than the variable surface climate, 2) minimizing community concerns by placing the crushing, sizing, and stowing operations underground, and 3) reducing surface vibration concerns with smaller blast sizes. Drawbacks of underground mining relate to the added health and safety hazards for the stone miners associated with; increased exposure to falls of ground, airborne contaminants, and fog in large underground openings. Also, injuries from falls of ground have occasionally been above incident rates for other mineral resources mined underground (Iannacchione, et al., 1995).

It is clear that existing underground stone operations mine more stone at a faster rate and with larger equipment. Because of high demand there is increasing pressure to yield more stone per production blast. A large majority of the industry uses the V-cut blasting pattern, that limits the magnitude of depth or pull for each shot to about 4 m (13 ft). Therefore, to produce more stone the miners must work more faces or existing faces must be enlarged.

Enlarging underground openings poses problems for maintaining strata stability. First, widths of the mine entries partially control the amount of sag or deflection any given roof beam can withstand before failure. This deflection is, in general, very small and can take place quickly after an opening is excavated or much later as weathering processes aid in forming new, thinner beams. Because increasing deflection increases the potential for roof beam failure, there is some limit to safely widening rooms and these limits depend on local geologic and stress conditions.

Room width limitations have placed more attention on expanding production through benching. Heights of

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rooms partially control the strength characteristics of adjacent pillars. As the room height increases for a given pillar width, the pillar width-to-height ratio (w/h) decreases. In general, stone pillars are extremely strong, however, when the width-to-height ratio falls, the potential for pillar rib instabilities and pillar failures increases (Figure 1).



Figure 1 - Photographs of failing pillars. Photo on left shows a crushed edge with a concaved shape and photo on right shows a failed pillar in the foreground [Note the crushed ribs and the slender appearance, $w/h < 1$ while the pillar in the background is stable with a $w/h > 1$].

In this environment the desire to utilize wider rooms, higher benches, and multilevel mining may result in the potential for more unstable ground conditions. While the number of failed pillars is currently very low, the potential for additional failures will grow if more slender pillars are developed: 1) especially under deeper cover, 2) with wider mining sections, and 3) in multilevel mining scenarios. It should also be noted that time acts to decrease the strength of many existing pillars. It is the purpose of this paper to examine current design practices and to discuss issues for safe mine layouts so that a rational first approach towards balancing the demands for increased production can be weighed against increased risk to worker safety from rib instabilities and pillar failures.

CURRENT STONE PILLAR DESIGN PRACTICES

The National Institute for Occupational Safety and Health (NIOSH) surveyed 70 underground stone mines to collect information on pillar design practices. Every mine (with one exception) used the room-and-pillar mining method; however, this method has many variations in practice:

- When used in relatively flat lying seams, the pillars are often arranged in regular, reoccurring patterns. Based on the 70 mines surveyed, 93% are regular.
- Occasionally random patterns have been used, although this practice is decreasing in response to improvements in surveying and mine planning. Currently only two mines use the random method.
- The majority of mines use square pillars where the

width of the pillars are equal to the length of the pillars. Nine mines use rectangular pillars (herein referred to as rib pillars) where the pillar length is greater than the pillar width. This design technique has been championed by two prominent consultants in the field, Jim Scott (Iannacchione, 1997) and Jack Parker (1973).

- The room-and-pillar method takes on some unique characteristics when mining steeply dipping beds with multi-levels. Generally, one to three entries are driven along the strike of the strata which may dip from 45 to 70 degrees. Crosscuts are developed horizontally, perpendicular to the entries. Raises or windows between the outer, up-dip crosscuts and the inner, down-dip crosscuts provide for ventilation passages between mining levels. Only 2 mines used this method.
- Finally, the stoping method which employs raises, crown pillars, etc. is used by only one mine.

Because 93% of the mines surveyed uses the regular room-and-pillar technique in a flat lying seam, this paper will focus on analyzing this design method.

In the room-and-pillar mining method, entries and crosscuts are generally driven perpendicular to each other. When these headings are developed horizontally into unmined strata they are called development entries which outline development pillars. Faces are advanced by drilling horizontal holes and blasting the rock back into the development headings. Seventy-two development mining scenarios (two of the 70 mines surveyed used multiple development designs) with 63

Table 1 - Mine layout characteristics for underground U.S. stone mines.

Characteristic		Mean	Standard deviation	Median	Minimum	Maximum
Development	Pillar height, m(ft)	7 (23)	1.7 (5.6)	7 (23)	3.7 (12)	12.2 (40)
	Opening width, m(ft)	13.1 (43)	2.6 (8.6)	12.8 (42)	6.1 (20)	18.3 (60)
	Pillar width, m (ft)	12.2 (40)	4.1 (13.3)	12.2(40)	4.6 (15)	27.4 (90)
	Extraction ratio	0.76	0.07	0.75	0.56	0.91
	Pillar w/h	1.73	0.48	1.72	0.54	3.13
	Overburden, m(ft)	80 (262)	98 (321)	46 (150)	7 (23)	610 (2000)
Bench	Pillar height, m(ft)	14.6 (48)	3.8 (12.5)	14.6 (48)	6.7 (22)	24.4 (80)
	Opening width, m(ft)	13.7 (45)	2.1 (6.9)	13.7 (45)	9.1 (30)	18.3 (60)
	Pillar width, m(ft)	13.1 (43)	4.1 (13.3)	13.7 (45)	6.1 (20)	27.4 (90)
	Pillar w/h	0.92	0.35	0.90	0.4	1.92



square pillar designs and 9 rib pillar designs were examined. Development pillar sizes averaged 12.2 m (40 ft) wide by 7 m (23 ft) high (Table 1). Adjacent rooms averaged 13.1 m (43 ft) in width.

If the stone deposit is thick enough, benching of the floor often occurs. Bench faces are advanced by drilling vertical floor holes from development entries and blasting the rock back into the benched headings. Bench depths average 7.6 m (25 ft). A few of the deeper benches are mined in multiple lifts. Benchng does not alter either the widths of the entries or the pillars. It does have an influence on the height of the rooms and pillars, which directly impacts the pillar width-to-height ratio (Table 1). Thirty-five mines bench with the room-and-pillar method.

The distribution of width-to-height ratios for all of the 70 mines surveyed in this study is shown in Figure 2. Two distinct distributions exist based upon the functional characteristics of the pillar. The average pillar width-to-height ratio for development pillars was 1.73 with a standard deviation of 0.48, while the average width-to-height ratio for benched pillars was 0.92 with a standard deviation of 0.35. The distribution of the 72 development pillar designs is somewhat normal while the distribution of the 35 benched pillars is somewhat skewed to the left (Figure 2).

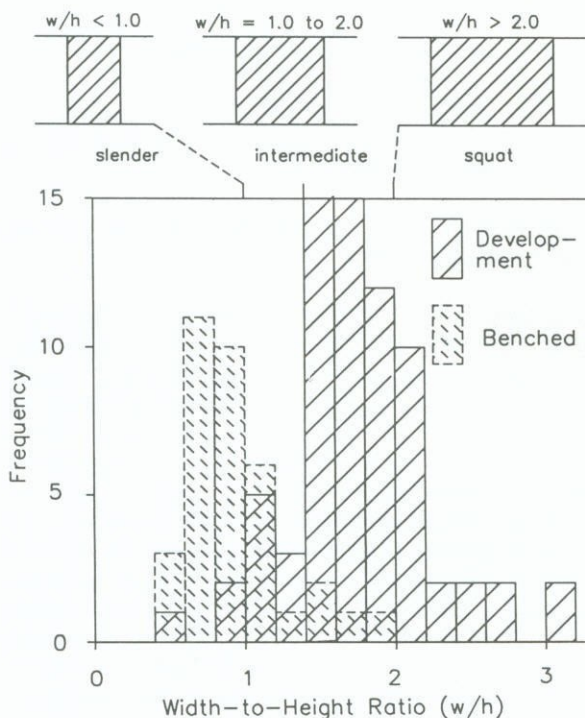


Figure 2 - Width-to-height ratios of pillars used in development and benching sections.

PILLAR PERFORMANCE ISSUES

The extraction ratio is another geometric characteristic which analyzes the relationship between the area of a pillar and the area of the adjacent opening along the horizontal plane. The extraction ratio for perpendicular intersections is determined by the following relationship:

$$e = \frac{(w + r) \times (l + r) - w \times l}{(w + r) \times (l + r)}$$

where

e = extraction ratio
w = pillar width
l = pillar length
r = opening width

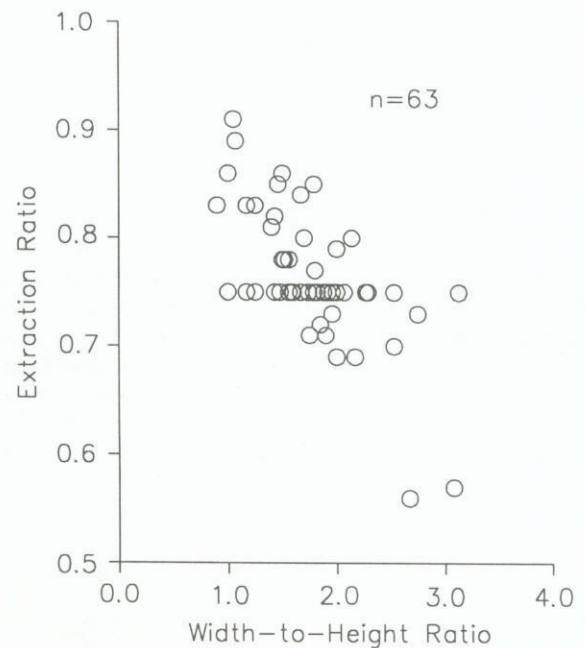


Figure 3 - Comparison between square pillar width-to-height ratios and the extraction ratios for development mining.

Figure 3 shows the relationship between the extraction ratio for square development rooms-and-pillars and the width-to-height ratios for these same pillars. In general, the extraction ratio decreases as the width-to-height ratio increases. This is expected because both ratios consider the pillar width as a factor. The safety concern is that as the extraction ratio increases the pillars receive higher levels of stress, while these pillars in a sense are becoming more slender. Obviously, slender pillars are inherently less stable.

Depending on the years of mining, an underground stone mine may have a few dozen to several thousand pillars. The vast majority of these pillars are adequately

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sized and currently stable. However, pillar design is seldom explicitly addressed. Pillar stability should be more closely examined under: 1) excessive stress levels, 2) adverse geologic conditions, and 3) increasing time.

Stress Levels

Generally, stone pillars are less stable if overburden is substantial because of the higher stress. Pillars are also less stable as the width-to-height ratio decreases such as in benching operations. Stress levels within pillars can be approximated by using the tributary area theory (Brady and Brown, 1985):

$$\sigma_a = \sigma_o \times \frac{(r + w) \times (r + l)}{w \times l}$$

where: σ_a = average post mining vertical stress
 σ_o = premining vertical stress

Pillar stress levels are affected by the overburden and the relationship between the area supported by the pillar and the area of the pillar. This relationship is illustrated by comparing the post mining vertical stress levels as the overburden and the extraction ratio increase. Figure 4 shows the relationship between average pillar stress, overburden, and extraction ratio for a 13.7 m (45ft) wide opening with square pillars. As overburden and extraction ratios increase, the stress levels rise exponentially. Incremental changes in overburden result in a proportional change in the average pillar stress levels at specific extraction ratios. Increasing extraction ratios produce an exponential rise in the average pillar stress levels at specific overburdens. At most underground stone mines, overburden averages 80 m

(262 ft), thus excessive stress levels would not be expected to result in pillar failure. The relatively shallow depth of most underground stone mines is why pillar shape and size are often overlooked as a safety issue during mine design and development.

Pillar Strength

The most generally accepted techniques for determining pillar strength, defined as the ultimate load per unit area of a pillar, use empirical derivations based on survey data from actual mining conditions. The power of the empirical method is that specific failure mechanism need not be considered. The failings of the method stems from an inability to extend these formulae beyond the specific material properties, sizes, shapes, and overburdens found in the survey data. Bieniawski (1984) wrote that the strength of mine pillars is dependent upon three elements: 1) the size or volume effect (strength reduction from a small laboratory specimen of rock to full size mine pillars); 2) the effect of pillar geometry (shape effect); and 3) the properties of the pillar material. For non-coal pillars, empirical formulas have largely been derived from some form of the following power formula:

$$\sigma_p = \sigma_m \times \frac{w^a}{h^b}$$

where σ_p = Pillar strength (ultimate)
 σ_m = Material strength
 h = pillar height
 a, b = constants derived from laboratory or field experiments

This formulation considers both material strength and pillar shape to calculate pillar strength.

Material Strength: In these equations, the material strength (σ_m) of a nominal size pillar is generally approximated by reducing the uniaxial compressive strength (σ_c) of the material from laboratory testing of small cylindrical or cubed shaped specimens. Laboratory test samples typically over-estimate rock material strength values because larger flaws or fractures are exhibited as specimen size increases. At some point specimen size becomes large enough so that further reductions in material strength become insignificant. This is often referred to as rock mass strength. Bieniawski (1984) suggested that cubic coal specimens from 0.9 to 1.5 m (3 to 5 ft) are of a critical size and are representative of rock mass characteristics. Hedley and Grant (1972) used an equivalent material strength value representative of a 0.3 m (1 ft) cube of quartzite. Others have used reduction factors ranging from 40 to 80 % to determine material strength from uniaxial compressive strength values.

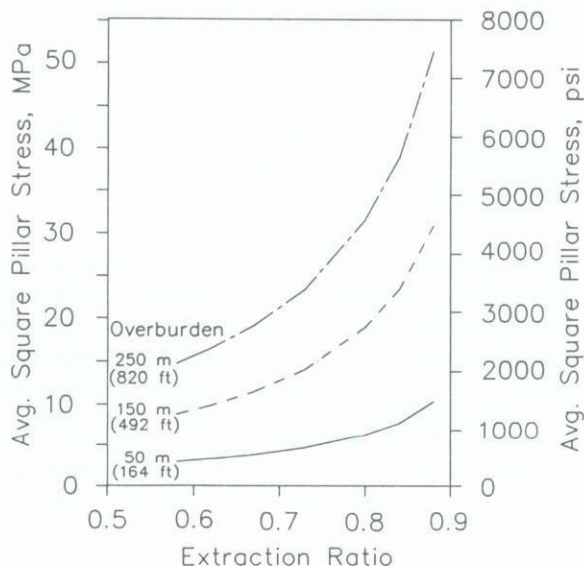


Figure 4 - Relationship between average pillar stress levels, extraction ratio, and overburden for a 13.7 m (45 ft) wide opening with square pillars.

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Pillar Shape: Several pillar strength equations have been expressed as a power function of the pillar's height and width. Salamon and Munro (1967) derived the following equation after analyzing 125 case studies from South African coal mines:

$$\sigma_p = 1320 \times \frac{w^{0.46}}{h^{0.66}}, (\text{psi})$$

In this equation a material strength (σ_m) equivalent to 9.1 Mpa (1320 psi) was used because coal has a relatively low material strength compared to most industrial minerals and metal mine rocks. Hedley and Grant (1972) extended this power function to describe the strength of quartzite pillars in deep uranium mines near Elliot Lake, Canada:

$$\sigma_p = 26000 \times \frac{w^{0.5}}{h^{0.75}}, (\text{psi})$$

The equation is similar to Salamon and Monroe with the exception of a higher material strength value for the stiff uranium host rock. In this case the material strength of uranium pillars is close to 20 times greater than for South African coal. An alternative application of a pillar strength formula was suggested by Hardy and Agapito (1975). From a study of oil shale pillars in western Colorado, the appropriate pillar strength formula was inferred to be of the form:

$$\sigma_p = \sigma_c \times \left(\frac{v_p}{v_s} \right)^{-0.118} \times \left(\frac{w_p}{h_p} / \frac{w_s}{h_s} \right)^{0.833}$$

Where: σ_c = Uniaxial compressive strength of sample
v = Volume of pillar/sample

This method involves determining the uniaxial compressive strength of a specimen where subscripts p and s refer to pillar and specimen, respectively.

Power functions produce a very distinctive relationship between strength and width-to-height ratio (Figure 5). At low width-to-height ratios (< 1.0), pillar strength rises rapidly. At higher width-to-height ratios strength increases occur at diminishing rates. In other words, at some point the pillar would begin to display some plastic behavior. Barron referred to this as pseudo-ductile behavior (Barron, 1984). The occurrence of pseudo-ductility in coal pillars has been debated for years. It seems unlikely that stiff, brittle materials like stone or other hard rocks would display this same type of plastic behavior. In fact, there has been a recognition that at width-to-height ratios greater than 4 or 5, strain hardening behavior can occur (Salamon and Wagner, 1985, Stacey and Page, 1986).

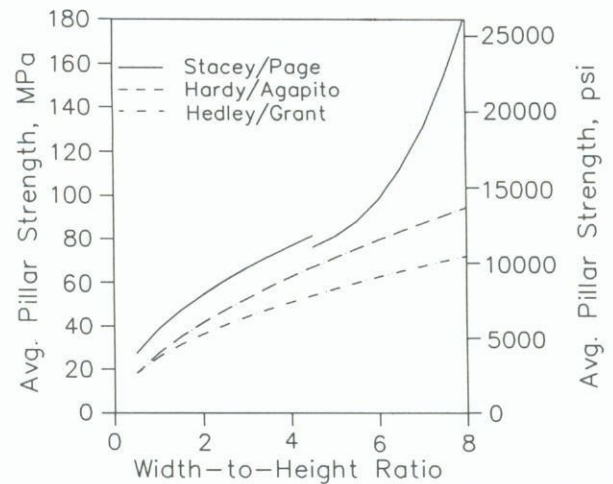


Figure 5 - Comparison between pillar width-to-height ratio and average pillar strength for several different empirical equations based on a power function. Note: material strength was normalized for each equation for comparative purposes.

The approach suggested by Stacey and Page (1986) takes this behavior into account by producing exponential rises in pillar strength at higher width-to-height ratios:

for $w/h < 4.5$

$$\sigma_p = k \times \frac{w^{0.5}}{h^{0.7}}, (\text{MPa})$$

for $w/h > 4.5$

$$\sigma_p = k \times \frac{2.5}{V^{0.07}} \times \left(0.13 \times \left(\left[\frac{\left(\frac{w_{\text{eff}}}{h} \right)^{4.5}}{4.5} \right] - 1 \right) + 1 \right), (\text{MPa})$$

Where: $k = \sigma_m \times \text{DRMS}$ (Design Rock Mass Strength)
DRMS = σ_c adjustment factor (Stacey and Page, 1986)
 $V = w_{\text{eff}}^2 \times h$
 $w_{\text{eff}} = 4 \times A_p / R$
 A_p = plan area of the pillar
R = perimeter

Here the pillar strength follows a power function for relatively low width-to-height ratio (< 4.5) and then begins to follow an exponential rise above width-to-height ratio > 4.5 (Figure 5).

The application of these formula to underground stone is problematic. It seems unlikely that increasing the pillar width-to-height ratio would result in a gradual

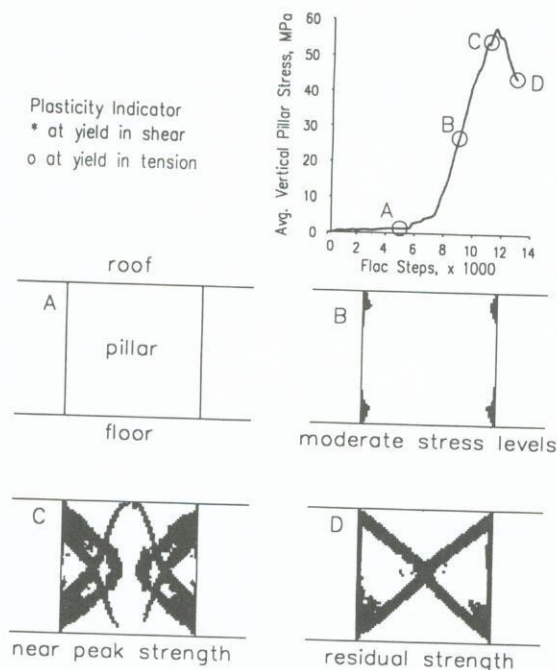


Figure 6 - An elastic-plastic model, which produces progressive failure patterns, can exhibit strain-softening behavior within full scale pillars.

decrease in the rise in pillar strength. In fact, in practice just the opposite appears to occur. Pillar stability is most endangered at low width-to-height ratios. As typical stone pillars reach a width-to-height ratio >1.5 , they begin to exhibit an almost indestructible character. The Stacey and Page (1986) formula appears to accommodate both the traditional strength flattening at moderate width-to-height ratios (~ 4 to 5) as proposed by Salamon and Munroe (1967) and the exponential rise in pillar strength at high width-to-height ratios. Unfortunately this strength adjustment occurs at width-to-height ratios that are well outside the ranges of all stone mines surveyed.

STONE PILLAR BEHAVIOR - Progressive Failure

The above techniques do not provide a totally realistic picture of stone pillar behavior. Additionally, a large database of pillar specific information of past successes and failures hasn't been established to aid in constructing a reliable pillar design technique for stone mines. In the absence of this information, numerical simulations can produce a potentially useful means to test engineering methods. The simulation used for this study was the 2 and 3-dimensional finite difference code (ANON, 1998). The 2-dimensional calculations were performed under plane-strain conditions, so the model sample is equivalent to a long pillar. It is assumed that individual elements in the model behave in an elastic-perfectly plastic behavior, however, the overall pillar

behavior can include strain-softening and strain-hardening. Model shapes mirrored those observed in the field ranging from a very slender pillar with a width-to-height ratio of 0.4 through intermediate pillars with a width-to-height ratio of 1.4 . With slender shapes, pillars failed rapidly by yielding from the ribs inward to the pillar core, indicating a relatively low strength structure. As pillars become more squat, elevated horizontal confinement dramatically increases pillar strength. Model symmetry was constructed so as to simulate a reoccurring pattern of rooms and pillars of equal dimensions. The roof and floor material were modeled to be elastic.

The model pillars had a moduli of 41.4 GPa ($6,000,000$ psi), an angle of internal friction of 40° , and a cohesion of 6.9 MPa ($1,000$ psi). The model pillars were subjected to simulated loading conditions by moving distant boundaries in the roof and floor together at very slow rates. This had the effect of gradually loading the pillars through several distinct strength phases. During the early loading phase the modeled pillar displays relatively elastic characteristics (Figure 6, Points A to B), where the pillar deformation is proportional to increases in average vertical stress levels within the pillar.

During this phase dominated by elastic behavior, minor yielding of the pillar edges begins to occur. Continued progressive failure of the pillar's outer perimeter produces an hour-glass shaped elastic core. It also has the effect of moving the peak vertical stress away from the pillar edge and toward the pillar center (Figure 6, Points B to C).

The maximum pillar strength was achieved when the highest vertical stress levels in the elastic core were supported by the maximum horizontal confinement available to the pillar (Figure 6, Point C). Beyond this point, any additional load to the pillar resulted in a rapid loss of strength. The zone of plastic yield extended throughout the pillar producing a residual pillar strength (Figure 6, Point D) which was considerably less than the maximum pillar strength.

THE EFFECT OF EXCESSIVE STRESS ON STONE PILLAR STABILITY

Pillar failure due to excessive stress levels was not frequently observed during field visits to underground stone mines. In fact, there were only seven development pillars at four mines that were observed to have undergone progressive failure due to excessive loading (Table 2). In Case 1 (Table 2) a pillar with a width-to-height ratio of 1.0 was inadvertently reduced from 9.1 to 6.1 m (30 to 20 ft) under less than 30 m (100 ft) of overburden. This pillar had an average vertical stress of less than 7 MPa (1000 psi). Failure of the pillar perimeter due to crushing left only a narrow core of

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broken rock (photo on right, Figure 1). It was surmised that additional stresses were applied to the pillar either as a result of its unique position near the highwall or due to some undetected geologic characteristic.

In Case 2, the initial mine development had occurred with random room-and-pillar methods. One failed pillar was observed that was narrower than the surrounding pillars. This pillar had a width-to-height ratio of 1.25 while adjacent pillars were in excess of 2.0. Here again it was difficult to determine what level of stresses caused failure since the overburden was very low.

Table 2 - Characteristics of development pillars that failed

Case	Observed failed pillars	Pillar width, m (ft)	Pillar height, m (ft)	w/h ratio	extraction ratio	Remark
1	1	6.1 (20)	6.1 (20)	1.0	0.94	Reduced pillar size
2	1	6.1 (20)	4.9 (16)	1.25	0.92	Smallest pillar in a non-regular mining area
3	1	5.5 (18)	6.1 (20)	0.9	0.86	High overburden stress and reduced pillar size
4	4	3 to 6.1 (10 to 20)	7.3 (24)	0.42 to 0.83	0.9 to 0.83	High overburden stress

In Case 3, a pillar with a width-to-height ratio of 0.9 was inadvertently reduced from 9.1 to 6.1 m (30 to 20 ft) under a high overburden condition of approximately 275 m (900 ft). Average vertical stresses on this pillar could have exceeded 28 MPa (4000 psi). The failed pillars had shear surfaces that began and terminated at the pillar-roof and pillar-floor intersections and propagated inward in a convex shape (photo on left, Figure 1).

In Case 4, the overburden ranged from 230 to 260 m (750 to 850 ft), producing an average vertical stress that was generally less than 14 MPa (2000 psi) on pillars with openings that ranged between 15 to 18 m (50 and 60 ft) in width and width-to-height ratios that averaged 2.1. In several areas within this large mine, pillars of reduced width with width-to-height ratios ranging from 0.83 to 0.42 were subjected to average vertical stresses that could have reached 35 MPa (5000 psi). At least four of these pillars exhibited a very distinctive concave failure surface that resembled an "onion skin". As the failure process continued, the pillar size was reduced, thereby increasing the extraction ratio and resulting in higher stress and more pillar deterioration.

As underground stone production expands, mining depths (overburden) are expected to increase. Presently six mines in the Valley and Ridge Province of Pennsylvania, Virginia, and Tennessee and two mines in Kentucky are mining at depths between 250 to 600 m (800 to 2000 ft). Many more mines will soon be

encountering their first 100 to 150 m (330 to 500 ft) overburdens. As depth of mining increases the potential for excessive stress levels that adversely affect pillar stability will also increase substantially.

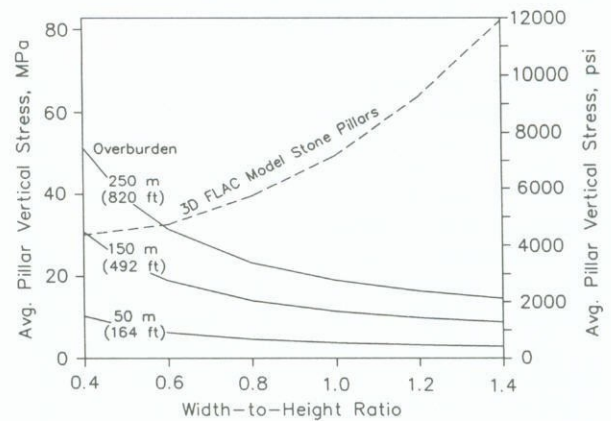


Figure 7 - Changes in stone pillar strength at width-to-height ratios ranging from 0.4 to 1.4.

In the absence of specific design information for underground stone mines, the following pillar design guidelines related to excessive stress levels are proposed. These guidelines consider the previous numerical simulations as a means of examining how pillars of various shapes will be effected under different overburdens. Figure 7 shows the changes in average vertical stress conditions at different overburdens and different width-to-height ratios using the tributary area theory. The solid black line represents the strength of a modeled pillar, free of discontinuities, as its shape changes from a slender pillar ($w/h=0.4$) to an intermediate pillar ($w/h=1.4$). This modeled pillar has a stiffness of 41.4 GPa (6 million psi) and a failure envelope defined by a friction angle (ϕ) of 40° and a cohesion of 6.9 MPa (1000 psi). These values represent typical material strength characteristics of Loyalhanna Limestone of Pennsylvania and West Virginia.

In this example, our model pillar with a width-to-height ratio of 1.4 could potentially accommodate an average vertical stress of approximately 80 MPa (12000 psi). Clearly a pillar of this strength, free of discontinuities, could withstand all of the extraction ratio and overburden conditions shown in Figure 4. Conversely a slender pillar, with a width-to-height ratio of 0.6 and 250 m (820 ft) of overburden, might fail. Note that the FLAC model strength curve has a shape very different from those curves determined by the empirical design techniques illustrated in Figure 5. At low width-to-height ratios pillars are low in strength. As width-to-height ratios increase to 1.0 and beyond, the pillar strength increases considerably. At a width-to-height ratio of greater than 1.5, pillars free of geologic discontinuities are unlikely to fail. The shape of the pillar strength curve shown in Figure 7 is defined by the stiffness of the material and shape of the failure

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envelope, which is defined by the friction angle and the material cohesion used in the model.

THE EFFECT OF DISCONTINUITIES ON STONE PILLAR STABILITY

The stability of a stone pillar is highly influenced by overburden stresses and the occurrence of geologic discontinuities. Observations of pillar conditions have shown that the presence of geologic discontinuities represents a more likely potential safety concern for mine workers than excessive stress levels. When a room is heightened, the area of the rib increases, which elevates the potential for exposure of geologic discontinuities and mining induced damage intersecting the rib. Characteristics of geologic discontinuities, such as bedding planes, slips, faults, and joints, include frequencies, persistence, straightness, strike, dip, and material properties. Mining induced damage can result from drilling, blasting, or scaling operations, and is an extremely important factor in designing safe, stable pillars, but is not considered in this report.

Bench Mining Case Studies

The best way to analyze the influence of geologic discontinuities on pillar stability is to evaluate the pillar performance in benching operations where pillar shapes are generally slender. A comparison of pillar shape, room width, and pillar height of 35 square bench pillar designs is shown in Figure 8. There appears to be no clear correlation between pillar height or room width and

pillar failure. Indeed, three of the four designs with some failed pillars had only moderate pillar widths and heights (Figure 8, Points B, C, and D). These operations were all mining in a formation which is known to have a higher than normal occurrence of large, angular geologic discontinuities. The fourth design with failed pillars (Figure 8, Point A) was used in a formation not known to have a high occurrence of geologic discontinuities. Here pillar failure was probably associated with the slenderness of the pillar ($w/h=0.5$).

There are several factors that make the character of the geologic discontinuity significant:

- Persistence - the length of the discontinuity must be on the same scale as the pillar it is effecting if strength is to be impacted,
- Dip - the dip of the discontinuity can dramatically affects pillar strength (Figure 9),
- Frequency - the spacing between discontinuities is very important in determining the potential for failure in a large mining area,
- Material Properties - the properties of the discontinuity can be used to assess the magnitude of strength reduction, and
- Orientation - the orientation of the discontinuity is important when the pillars are rectangular, in that, strength will be most affected when the orientation of the discontinuity is aligned with the long axis of the pillar.

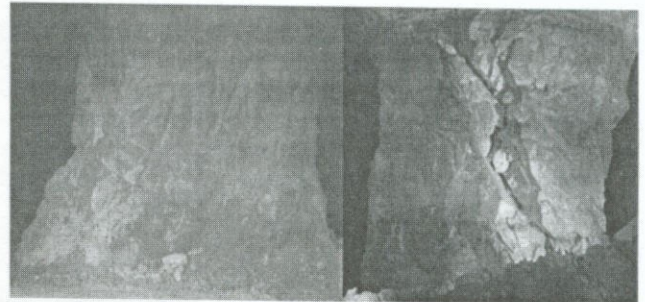


Figure 9 - The left pillar has a discontinuity dipping at approximately 20° while the right pillar's discontinuity dips at approximately 60°.

Characteristics of Discontinuities

Several of these factors can be considered by utilizing the analytical procedure discussed by Farmer (1983):

$$\sigma_1 = \frac{2C_d + 2\sigma_3 \tan \phi_d}{(1 - \cot \beta \tan \phi_d) \sin 2\beta}$$

where

σ_1 = Vertical strength,

σ_3 = Confinement,

C_d = Cohesion along the discontinuity,

ϕ_d = Friction angle along the discontinuity, and

β = Dip of discontinuity measured from the vertical axis.

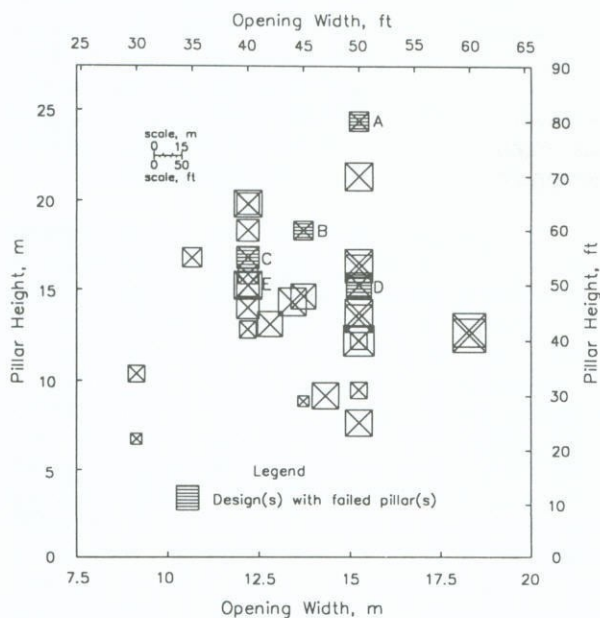


Figure 8 - Comparison of pillar shape, opening width, and pillar height of 35 square bench pillar designs. Note that changes in square size are proportional to changes in pillar sizes.



Figure 10 shows how sample strength varies for different discontinuity dip angles under a range of discontinuity material properties. This analysis is instructive in demonstrating the relative magnitude of strength reduction based on discontinuity dip angle. It also shows how friction angle and cohesion along the discontinuity individually affect unit strength. However, this technique can have limited value in analyzing stone pillar behavior with discontinuities because it treats the material as one uniform elastic mass.

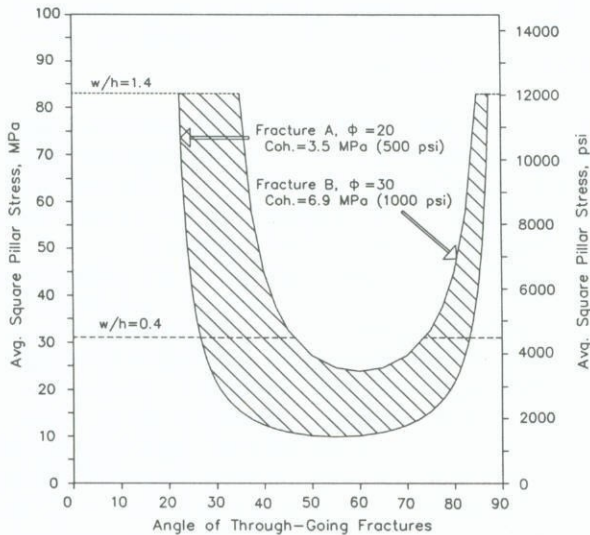


Figure 10 - The affect of angular geologic discontinuities on vertical strength using an analytical technique.

An alternative approach is to introduce simulated discontinuities into the previously discussed finite difference elastic-plastic pillar model and rotate the interface through a series of angles. In this way variations in material properties, discontinuity dips, and pillar shapes can be evaluated. This was accomplished by utilizing the ubiquitous joint model in the FLAC 3D program. The ubiquitous-joint model is an anisotropic plasticity model that includes weak planes of specific orientation embedded in a Mohr-Coulomb solid. In this model, yield may occur in either the intact rock or along a joint (discontinuity), or both, depending on the stress state, the orientation of the joint plane, and the material properties of the intact rock and joint plane.

Parametric analysis of the effect of discontinuities on pillar strength was conducted by varying the dip and material properties within the ubiquitous joint model. These simulations passed discontinuities through modeled pillar shapes that ranged from width-to-height ratios of 0.6 to 1.2. Three distinct pillar behaviors were observed based on the dip of the discontinuities. Figure 11 shows the strength profiles for pillars with a stiffness of 41.4 GPa (6 million psi) and a Mohr-Coulomb failure envelope defined by a friction angle of 40° and a discontinuity friction angle of 25°. The lowest pillar

strength occurred at a discontinuity dip angle of 57.5° (Figure 11). It should be noted that the point of lowest strength is defined in the ubiquitous joint model by the following relationship:

$$\beta_{\min} = 45 + \frac{\phi_d}{2}$$

where:

$\beta_{\min}$ = Angle of through-going discontinuity which produces the lowest average vertical peak stress,
 ϕ_d = Internal angle of friction for the discontinuity.

How well this model characteristic fits field conditions needs to be evaluated further. The highest pillar strength occurred with a discontinuity dip of 0° and gradually decreased as discontinuity dip angle increased. As the discontinuity dip angle increased above 57.5°, pillar strength began to increase again; however, the original strength for the intermediate to squat pillar shapes was not reestablished.

For all model shapes, and for a given set of material properties, pillar strength associated with the discontinuity dip of 90° was the same. In the case of the example shown in Figure 11 that value was approximately 29 MPa (4200 psi). It should be noted that this characteristic may be model driven and not representative of field conditions. The ubiquitous joint model would attempt to force discontinuities through columns of equally sized grid elements. While this behavior may be indicative of successive column failure through a pillar with numerous equally spaced vertical joints, it may not be indicative of pillars affected by variably spaced joints.

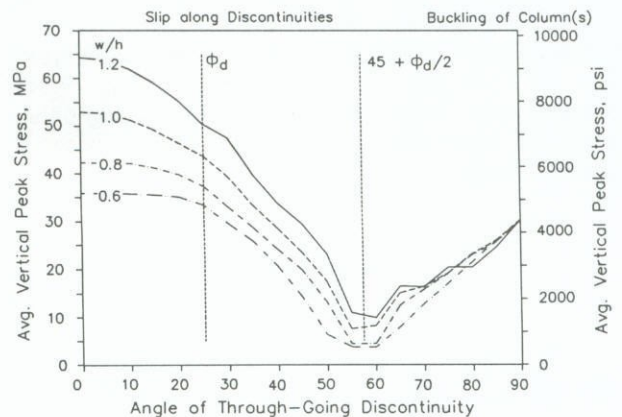


Figure 11 - Changes in average vertical peak stress as discontinuity dips are varied from 0 to 90° for four different width-to-height ratio model pillars.

Stress versus strain plots for these numerical simulations are shown in Figure 12. For discontinuity dips between 0 and 45° the material displayed an elastic-plastic behavior. When this model was run with

The intent of this report is to create an awareness in relation to mine worker safety of the potential for pillar failure, particularly in operations that bench. As underground stone mining expands and depth of overburden increases, consideration of the appropriate size and shape of pillars should be an integral part of the overall mine design. Using data gathered from in mine visits, mine maps, discussions with operators, and numerical simulations, stone pillar design issues were investigated and general guidelines were discussed. The important pillar design issues and guidelines can be summarized as follows:

- Most stone pillar have relatively low width-to-height ratios, ranging from 0.54 to 3.13, and high extraction ratios, ranging from 0.56 to 0.91. This results in many pillars having shapes that are slender with relatively large adjacent mine openings.
- Excessive vertical stress levels are generally not a serious problem in stone mines because the pillar material is often very strong and the overburdens are typically very low (average overburden = 80 m [262 ft]). However, there are a few known cases where pillars have failed due to excessive stress levels. These cases were associated with local unintentional pillar size reduction, overburdens greater than 200 m (630 ft), width-to-height ratios less than 1.25, or extraction ratios greater than 0.83. As depth of mining increases, the potential for excessive stress levels that adversely affect pillar stability will also increase substantially.
- The strength of slender pillars is best understood by examining models which allow for shape variations and progressive failure through elastic-plastic or strain softening behavior. Models which met this criteria were examined in the paper and demonstrated that stone pillar strength should not follow a linear relationship with pillar shape. Empirical strength formulas for metal/nonmetal mines produce power curves which produce higher pillar strengths at low width-to-height ratios (<1.0) and lower pillar strengths at moderate to high width-to-height ratios (>1.5).
- Bench mining produces slender pillars. Where geologic discontinuities are present, the potential for pillar failures increases. The persistence, dip, frequency, and material properties of these discontinuities control pillar strength.
- A strong correlation was found between the material properties and dip of discontinuity and the modeled pillar strength. As discontinuity dips increased from 0 to $45 + \phi_d/2$, pillar strength gradually decreased. When the dips were equal to $45 + \phi_d/2$, pillars exhibited a very unstable behavior, losing considerable levels of their original strength. As vertical orientations were approached, columns defined by discontinuities can control pillar behavior.

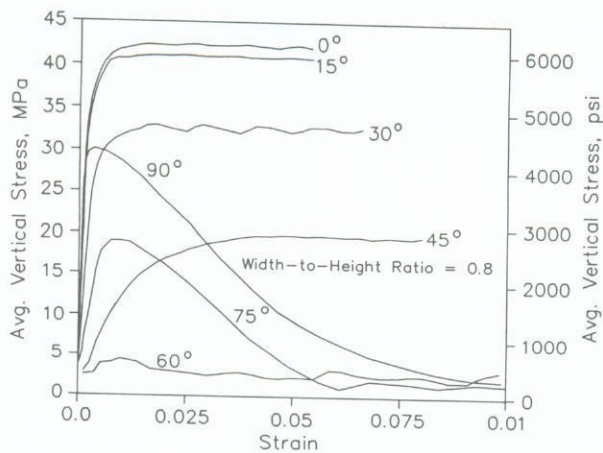


Figure 12 - Stress versus strain for model pillars (width-to-height ratio = 0.8) with discontinuity dips ranging from 0 to 90°.

a discontinuity dips of 60°, the model pillar exhibited very low strength. Finally, strain softening material behavior occurred as discontinuity dips increased to 90°. The conclusion of this effort is that both observation and numerical data suggest that when discontinuities are present at a particular angle and composition, they can control the behavior and strength of pillars.

The impact of this analysis is detailed in Figure 13. The strength of pillars with various shapes and different discontinuity dips are very sensitive to changes in extraction ratio and overburden. For example, a discontinuity dipping at 60°, under approximately 50 m (165 ft) of overburden, with a width-to-height ratio of 0.8, and passing through the entire 15 m (50 ft) high pillar, could fail if its properties were equivalent to material with a friction angle of 25° and a cohesion of zero. If this same discontinuity dips at 45°, the pillar might not fail until almost 250 m (820 ft) of overburden is encountered. Figure 13 is presented to illustrate the significant impact that pillar shape, overburden, and discontinuities can have on stone pillar strength and is not meant to be a design guideline.

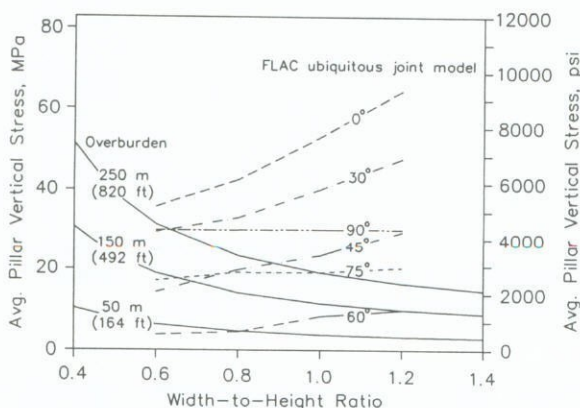


Figure 13 - Relationship between pillar shape, overburden, discontinuity dip and pillar strength.

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The pillar design guidelines developed through the observational and numerical simulations discussed above will require further field confirmation. This information has been presented so that mine planners, operators, and workers can recognize the potential hazards that exist when designing stone pillars. This approach can help to form a part of a comprehensive pro-active mine safety ground control plan for underground stone mines.

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Calibration of the Analysis of Longwall Pillar Stability (ALPS) Chain Pillar Design Methodology for Australian Conditions

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ABSTRACT

This paper summarizes the results of a research project whose goal was to provide the Australian coal industry with a chain pillar design methodology readily usable by colliery staff. The project was primarily funded by the Australian Coal Association Research Program and further supported by several Australian longwall operations.

The starting point or basis of the project was the Analysis of Longwall Pillar Stability (ALPS) methodology. ALPS was chosen because of its operational focus; it uses tailgate performance as the determining chain pillar design criterion rather than simply pillar stability. Furthermore, ALPS recognizes that several geotechnical and design factors, including (but not limited to) chain pillar stability, affect that performance.

There are some geotechnical and mine layout differences between United States and Australian coalfields that required investigation and, therefore, calibration before the full benefits offered by the ALPS methodology could be realized in Australia.

Ultimately, case history data were collected from 19 longwall mines representing approximately 60% of all Australian longwall operations. In addition, six monitoring sites incorporated an array of hydraulic stress cells to measure the change in vertical stress throughout the various phases of the longwall extraction cycle. The sites also incorporated extensometers to monitor roof and rib performance in response to the retreating longwall face.

The study found strong relationships between the tailgate stability factor, the Coal Mine Roof Rating, and the installed level of primary support. The final outcome of the project is a chain pillar design methodology called Analysis of Longwall Tailgate Serviceability (ALTS). Guidelines for using ALTS are provided.

INTRODUCTION

In many cases, chain pillars in Australia have been designed solely with regard to pillar stability using a process similar to that used for pillars within bord-and-pillar operations. The bord-and-pillar approach is based on analysis of collapsed pillar cases from Australia and the Republic of South Africa (1) and applies a factor of safety in relation to pillar collapse. This approach is inappropriate for a number of reasons when designing chain pillars.

Australian chain pillars typically have minimum width-to-height (w/h) ratios >8 , which is approximately 4.5 standard deviations away from the mean of the pillar collapse case histories. In addition, the chain pillar loading cycle and active life are significantly different from those experienced by pillars within a bord-and-pillar operation. Finally, the goal of maintaining gate road stability is very different from that of avoiding a pillar collapse.

The need for a design method uniquely developed for Australian longwall chain pillars was clear. The original submission for funding by the Australian Coal Association Research Program (ACARP) stated that the calibration (to Australian conditions) of a proven chain pillar design methodology offered the least risk for a successful and timely outcome. It was assessed that the most comprehensive chain pillar design tool then available was the Analysis of Longwall Pillar Stability (ALPS) (2, 3). The primary consideration in selecting ALPS is that it uses gate road (i.e., tailgate) performance as the determining chain pillar design criterion. Secondly, ALPS is an empirical design tool based on a U.S. coal mine database; thus, it provided a ready framework for calibration to Australian conditions.

The aim of the project was to provide the Australian coal industry with a chain pillar design methodology and computer-based design tool readily usable by colliery staff. A further objective was to ensure that the methodology developed by the project had the widest possible application to all Australian

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coalfields by identifying where local adjustments and limitations may apply.

In formulating the design methodology, the primary goal was to optimize pillar size (specifically pillar width) so as to—

- Maintain serviceable gate roads such that both safety and longwall productivity are unaffected;
- Minimize roadway drivage requirements so as to have a positive impact on continuity between successive longwall panel extraction; and
- Maximize coal recovery.

In designing chain pillars, specifically with regard to satisfactory gate road performance, the following design criteria were proposed:

- The chain pillar must provide adequate separation between the main gate travel road and belt road, such that the travel road (tailgate of the subsequent longwall panel) will be satisfactorily protected from the reorientation and intensification of the stress field caused by the extraction of the first longwall panel.
- The tailgate (with a focus on the tailgate intersection with the longwall face) will be sufficiently serviceable for ventilation and any other requirements (setting of secondary support, second egress, etc).

BACKGROUND

ALPS was originally developed by Mark and Bieniawski (4) at The Pennsylvania State University. It was further refined (2, 3, 5) under the auspices of the former U.S. Bureau of Mines (USBM)¹.

The initial ALPS research involved field measurements of longwall abutment loads at 16 longwall panels at 5 mines. These measurements were used to calibrate a simple conceptualization of the side abutment, similar to models proposed by Wilson (6) and Whittaker and Frith (7). The side abutment (A) equates to the wedge of overburden defined by the *abutment angle* (β) (figure 1). The tailgate loading condition is considered to be some percentage of the side abutment, called the *tailgate abutment factor* (F_t). The U.S. field measurements found a range of abutment angles, from $\beta = 10.7^\circ$ to $\beta = 25.2^\circ$. A value of $\beta = 21^\circ$ and $F_t = 1.7$ was selected for use in design.

Because of the encouraging results obtained from the initial study, the USBM commissioned further research directed toward quantifying the relative importance of roof and floor quality and artificial support on gate road performance. The approach was to analyze actual longwall mining experience. Case histories

from 44 U.S. longwall mines were characterized using 5 descriptive parameters. Pillar design was described by the ALPS stability factor (ALPS SF); roof quality was described by the Coal Mine Roof Rating (CMRR) (8, 9). Other rating scales were developed for primary support, secondary support, and entry width.

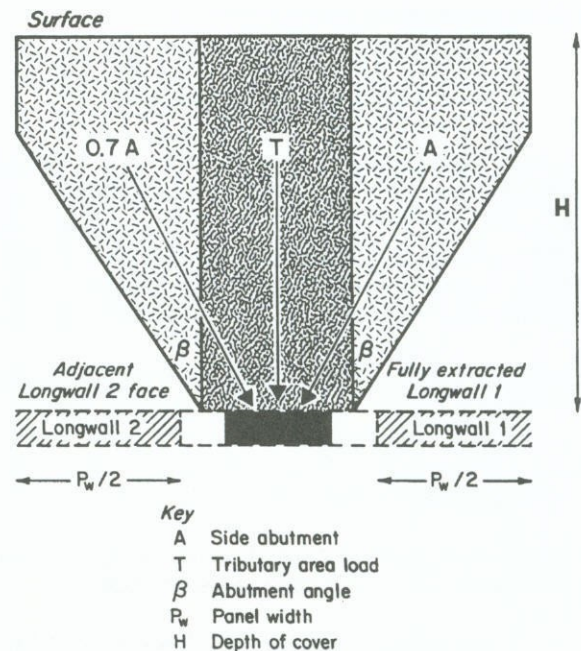


Figure 1. Conceptual model of the side abutment load.

Mark et al. (3) reported that statistical analyses indicated that in 84% of the case histories the tailgate performance (satisfactory or unsatisfactory) could be predicted correctly using only the ALPS SF and the CMRR. It was further stated that most of the misclassified cases fell within a very narrow borderline region. The analyses also confirmed that primary roof support and gate entry width are essential elements in successful gate entry design. The relative importance of the floor and of secondary support installed during extraction could not be determined from the data.

The following equation (relating the ALPS SF and CMRR) was presented to assist in chain pillar and gate entry design:

$$\text{ALPS SF}_R = 1.76 - 0.014 \text{ CMRR}, \quad (1)$$

where the ALPS SF_R is the ALPS SF suggested for design.

The Primary Support Rating (PSUP) used in ALPS was developed as an estimate of roof bolt density and is calculated as follows:

$$\text{PSUP} = \frac{L_b \cdot N_b \cdot D_b}{S_b \cdot w_e \cdot 84},$$

¹The safety and health research functions of the former U.S. Bureau of Mines were transferred to the National Institute for Occupational Safety and Health in October 1996.

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where L_b = length of bolt, m,
 N_b = number of bolts per row,
 D_b = diameter of the bolts, mm,
 S_b = spacing between rows of bolts, m,
 and w_e = entry (or roadway) width, m. (2)

PSUP treats all bolts equally and does not account for load transfer properties, pretensioning effects, etc.

NEED FOR CALIBRATION

Conventional longwall mines in the United States generally use a three-heading gate road system; Australian longwall panel design typically employs a two-heading gate road system with rectangular chain pillars separating these gate roads. A typical Australian longwall panel layout is presented in figure 2. Figure 2 also details the stages of the chain pillar loading cycle:

1. Development loading (calculated using tributary area concepts);
2. Front abutment loading, which occurs when the first longwall face is parallel with the pillar;
3. Main gate (side) abutment loading, when the load has stabilized after the passage of the first face;
4. Tailgate loading, when the second face is parallel with the pillar; and
5. Double goafing, when the pillar is isolated between two goafs.

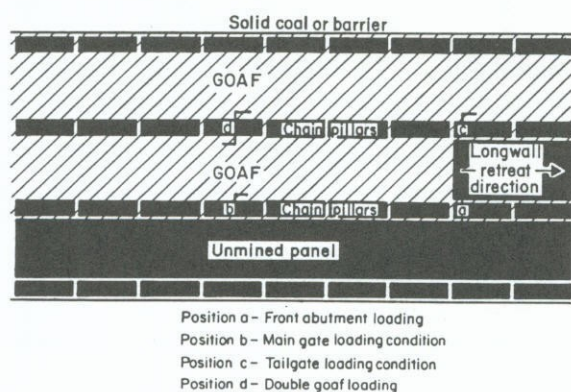


Figure 2. Stages in the dynamic loading cycle of longwall chain pillars.

It is during tailgate loading that the chain pillar (or cross section thereof adjacent to the tailgate intersection) experiences the greatest vertical loading during its "active life," i.e., the period where the chain pillar is playing its role in helping to maintain satisfactory gate road conditions. This project focused on tailgate performance (at the T-junction) as the design condition. The pillar stability factor in relation to the tailgate loading condition is designated as the "tailgate stability factor" (TG SF).

The project found that Australian chain pillars have an average length-to-width ratio of 3.2; crosscut centers on average are spaced at 100 m. The pronounced rectangular shape of Australian chain pillars may add strength to the pillar compared to a square pillar of the same minimum width. Mark et al. (10) reanalyzed the U.S. database using the Mark-Bieniawski rectangular pillar strength formula and found a slightly better correlation (in relation to the predictive success rate) than using the Bieniawski equation. In addition to the Bieniawski equation, this project assessed both the Mark-Bieniawski rectangular pillar formula (11) and the squat pillar formula (12) in relation to the correlation between the pillar stability factor and the CMRR.

In Australia, the significant impact of horizontal stress on coal mine roof stability is well documented (13, 14). The in situ horizontal stresses should not have a significant direct influence on tailgate roof stability due to the presence of an adjacent goaf. However, there is an indirect influence in terms of the degree of damage done to the roof during the initial roadway development and then to the main gate travel road and cut-throughs during longwall retreat. The effect of the in situ horizontal stress field on gate road serviceability (particularly on roof stability) is not taken into account directly by the ALPS methodology and was considered in more detail by the ACARP project. Finally, the project aimed to verify the applicability of the ALPS loading parameters β and F_t to Australian conditions.

MEASUREMENTS OF AUSTRALIAN ABUTMENT LOADS

The project measured changes in vertical stress across (and within) chain pillars at six collieries to determine whether the ALPS approximations should be refined. Three sites were located in the Bowen Basin Coalfield in Queensland (Central, Crinum, and Kenmare Collieries), two were in the Newcastle Coalfield (Newstan and West Wallsend Collieries), and one was at West Cliff Colliery in the Southern Coalfield (figure 3). Each monitoring site included an array of hydraulic stress cells (HSCs) generally located at midseam height to measure the changes in vertical stress. Most sites also included extensometers to monitor roof and rib performance. A general instrumentation layout is shown in figure 4.



Figure 3. Location of the field instrumentation sites.

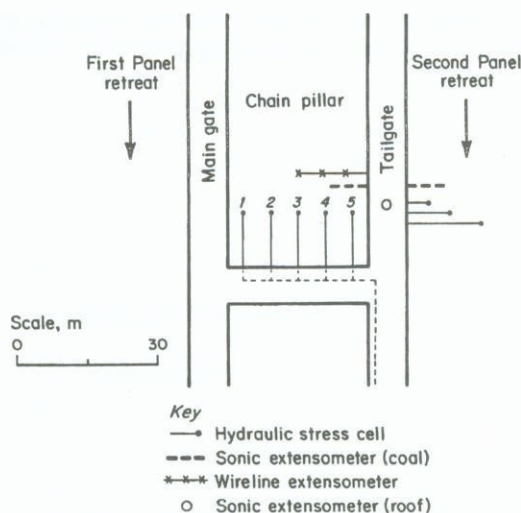


Figure 4. Instrumentation layout at a typical stress measurement site.

The six sites add considerably to the ALPS abutment load database. They include a much wider range of cover depths and width-to-depth ratios than the original U.S. data. There is also much more variety in the geologic environments. In addition, because the stress readings could be made remotely, monitoring was possible subsequent to the passing of the second longwall face. Of the 16 original U.S. panels, there were sufficient data to characterize the side abutment load in only 6, and only one panel provided data on the tailgate abutment factor. In contrast, data on both the side and tailgate loads were obtained from all six Australian monitoring sites. Details on the sites have been published elsewhere (15, 16).

Measurements of the main gate side abutment loading are used to calculate the abutment angle; measurements of the tailgate abutment (when longwall 2 is parallel with the instruments) are used to calculate the tailgate abutment factors. Figure 5 shows that the abutment angles at the three Queensland sites and from Newstan Colliery clearly fall within the range of

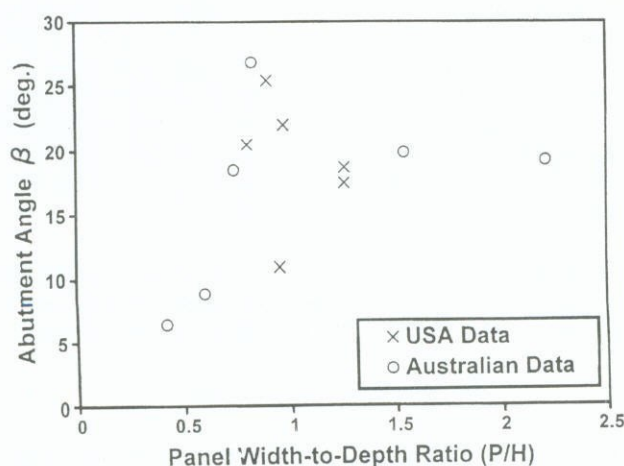


Figure 5. Abutment angles determined from stress measurement.

the U.S. data. However, the abutment angles calculated for the two deepest mines, West Wallsend and West Cliff, are the smallest of any in the database. The overburden at these two mines (and at Newstan Colliery) also contains the massive sandstone and sandstone/conglomerate strata commonly associated with the Newcastle and Southern Coalfields. The low width-to-depth ratio, along with the strong overburden, may be affecting the caving characteristics of the gob.

The Australian measurements of the tailgate abutment factor also show a high variability, with the mean at 1.3 in relation to an ALPS-style analysis. Figure 6 plots the development of the change in load during tailgate loading (as a multiple of the side abutment) against face position. It clearly indicates that the nature of the loading behavior at Central, Crinum, and Kenmare Collieries closely approximates that proposed by ALPS. However, the tailgate loading behavior at Newstan Colliery and particularly at West Wallsend Colliery reveals that the *double goaf load* is significantly greater than twice the measured main gate side abutment load. These data clearly suggest that a *much* greater portion of the main gate abutment load is distributed onto the adjacent unmined longwall panel than calculated on on theoretical grounds (figure 2).

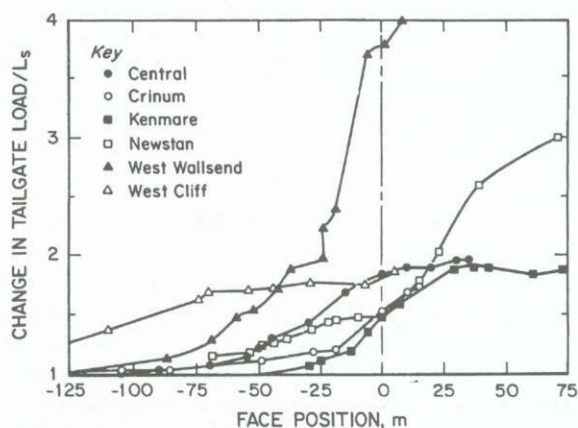


Figure 6. Development of abutment load at six monitoring sites.

The stress measurements collected by the project were supplemented by data from similar investigations previously conducted by other collieries, which were gratefully made available to the project (16). In general, the supplementary field data support the observations made from the project data. In Bowen Basin collieries, the loading behavior closely approximates that proposed within ALPS. For New South Wales collieries with strong, spanning overburden and a low width-to-depth ratio, the measured side abutment angles were significantly less than 21° .

In summary, it seems that an tailgate abutment factor $F_t = 1.5$, in conjunction with an abutment angle of $\beta = 21^\circ$, is a reasonable and generally conservative approximation of the actual tailgate load for most Australian mines. The exceptions are two collieries and one locality (containing three collieries) within the Australian database, where there is sufficient evidence

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to suggest that site-specific loading parameters are more applicable. These are the Central and West Wallsend Collieries, and the deepest collieries within the Southern Coalfield (South Bulli, Tower, and West Cliff Collieries). For Central Colliery, the appropriate loading parameters seem to be $\beta = 26^\circ$ and $F_t = 1.6$. With regard to the three Southern Coalfield collieries, the recommended loading parameters are $\beta = 10^\circ$ and $F_t = 1.5$, which also apply to areas associated with West Wallsend Colliery that are unaffected by the *near-seam* sandstone/conglomerate channels. In areas where thickening of the channel occurs, it is assessed that the abutment angle of $\beta = 10^\circ$ should be maintained, while F_t should be increased to 3.5.

Two other variables can influence the calculation of pillar stability factors: *in situ* coal strength (S_t) and the *overburden* density (γ). A comprehensive study in the United States recently concluded that uniaxial compressive strength tests on small coal samples do not correlate with *in situ* pillar strength (17). That study and others in Australia and the Republic of South Africa (1) found that using a constant seam strength works well for empirical pillar design methods. Accordingly, the *in situ* coal strength is taken to be 6.2 MPa, as used in ALPS.

In some Australian mines, there is so much coal in the overburden that the overburden density is significantly reduced below the $\gamma = 0.25 \text{ MN/m}^3$ that is typical for sedimentary rock. Dartbrook and Kenmare Collieries have undertaken satisfactory analyses of their overburden and have determined that $\gamma = 0.22 \text{ MN/m}^3$ and 0.23 MN/m^3 , respectively.

INDUSTRY REVIEW

The aim of the industry review was to construct a historical database of gate road and chain pillar performance. During the course of the project, 19 longwall mines (a cross section from the 5 major Australian coalfields) were visited. Underground inspections were conducted at each that incorporated a subjective assessment of gate road performance while documenting the relevant details in relation to panel and pillar geometry, roof and floor geology, artificial support, and *in situ* stress regime. Brief summary reports were then forwarded to each mine to confirm the accuracy of the recorded data. The complete data set has been summarized elsewhere (16).

The U.S. database included the Secondary Support Rating (SSUP), which is described as a rough measure of the volume of wood installed per unit length of the tailgate (3). It should be noted that 59 of the 62 cases (i.e., 95%) within the U.S. database used standing secondary support (predominantly in the form of timber cribbing) along the tailgate. In the Australian database, less than 50% (9 out of 19) mines routinely installed standing secondary support along the tailgate. In the context of this study, standing secondary support refers to timber cribbing, the Tin Can system, Big Bags, etc., and does not include tendon support (cable bolts or Flexibolts) installed within the roof. Because of the variety of secondary supports used, a yes/no variable was used instead of a quantitative rating. Figure 7 illustrates the relationship between SSUP, CMRR and depth within the

Australian database. The presence of adverse horizontal stress conditions was also described by the yes/no variable HORST.

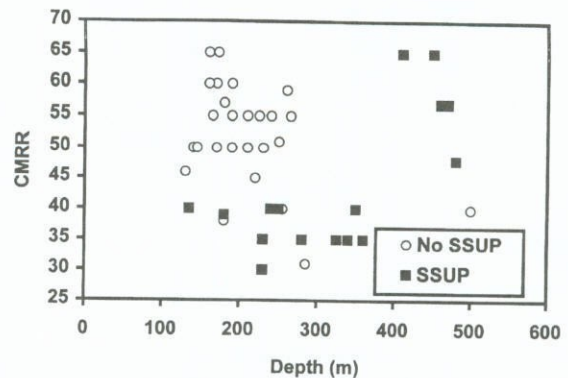


Figure 7. Relationship between SSUP, CMRR and depth of cover within the Australian data base.

STATISTICAL ANALYSES

The same statistical technique used with the U.S. ALPS database, that of discriminant analysis, was used with the Australian data. Discriminant analysis is a regression technique that classifies observations into two (or more) populations. In the case of the ALPS data, the classified populations are tailgates with satisfactory and unsatisfactory tailgate conditions.

An initial change that was made with the Australian data was to include "borderline" tailgates with the unsatisfactory cases. This modification is consistent with the Australian underground coal industry's desire to have in place strata management plans that design against both borderline and unsatisfactory gate road conditions. It also adds to the otherwise small pool of unsatisfactory cases available for analysis.

In their analysis, Mark et al. (3) were not able to quantify the effect of standing secondary support on tailgate conditions. However, because nearly every U.S. case used some standing support, SSUP is basically *intrinsic* to the design equation (equation 1). Because less than 50% of Australian mines use secondary support, it seems reasonable to assume that tailgates that presently incorporate standing secondary support would become unsatisfactory if it were removed. A major modification was to include all collieries utilizing standing secondary support in the modified-unsatisfactory category of tailgate conditions. The final database includes 50 case histories with 29 modified satisfactory and 21 modified-unsatisfactory cases.

Numerous analyses were conducted to determine the best design equation. Ultimately, the most successful design equation relates the required TG SF to the CMRR, as shown in figure 8:

$$\text{TG SF} = 2.67 - 0.029 \text{ CMRR} \quad (3)$$

Equation 3 correctly predicted the outcome of all except seven case histories, for a success rate of 86%. Comparing equation 3 to the U.S. design equation (equation 1), it may be seen that the TG SF is generally more conservative than the

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ALPS SF for weaker roof, but the TG SF decreases more rapidly than the ALPS SF as the roof becomes stronger.

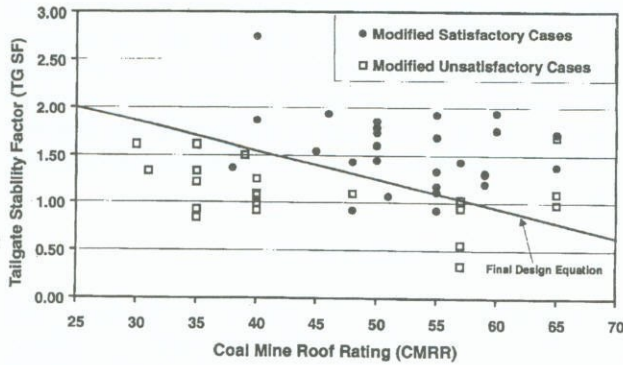


Figure 8. The final design equation relating the CMRR to the TG SF.

Another strong relationship that was evident in the case histories was between the primary support and the roof quality. Figure 9 plots the PSUP against the CMRR, and the best-fit regression is of the following form:

$$\text{PSUP} = 1.35 - 0.0175 \text{ CMRR} \quad (4a)$$

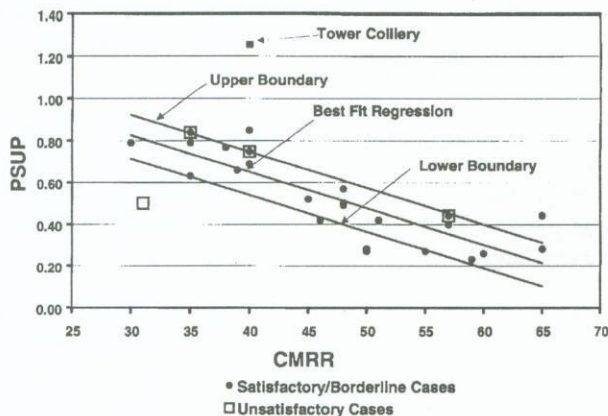


Figure 9. Design equations for primary support based on the CMRR.

It seems that Australian mine operators have intrinsically adapted their primary support patterns to the roof conditions and operational requirements. Mark et al. (3) reached a similar conclusion for the United States.

Upper- and lower-boundary equations (4b and 4c, respectively) relating CMRR to PSUP have also been proposed and are illustrated in figure 7:

$$\text{PSUP}_U = 1.45 - 0.0175 \text{ CMRR} \quad (4b)$$

$$\text{PSUP}_L = 1.24 - 0.0175 \text{ CMRR} \quad (4c)$$

Equation 4c may be applicable, for example, when the mining layout is not subject to adverse horizontal stress conditions

and/or standing secondary support is planned as part of the colliery's strata management plan.

Mark et al. (3) also found a strong correlation between the CMRR and the entry width. No such correlation was seen here.

It is interesting to note some similarities and differences between the U.S. and Australian databases. For example, overall roof quality seems to be reasonably similar in the two countries. The mean CMRR in the United States is 53.7 with a standard deviation (SD) of 13.9; this compares with an Australian mean of 49.5 and SD = 10.0. However, the mean Australian PSUP is 0.49 (SD = 0.23), which is approximately twice that of the U.S. database.

Studies by Mark (18) and Mark et al. (19) suggest that the horizontal stress levels in the two countries are comparable. It seems that philosophical differences are more likely responsible for the different levels of primary support. Most Australian coal mines have an unwritten (sometimes written) policy of no roof falls; U.S. multientry mining systems seem more tolerant of roof falls. Also, most Australian coal mines have an antipathy toward standing secondary support for reasons associated with a two-entry gate road system. It seems that the main way in which Australian operations prevent poor tailgate conditions is to install substantial primary support on development. Therefore, in Australia one would expect a strong relationship between the level of primary support and a reliable roof rating system. This is exactly what transpires, which adds to the credibility of the CMRR.

Additional statistical analyses tested whether the accuracy of ALPS could be improved by replacing the original Bieniawski formula with another pillar strength formula. It was found that the Mark-Bieniawski formula had virtually no impact on the classification success rate, while the squat pillar formula significantly reduced classification success rate. The conclusion was to remain with the original Bieniawski formula used in the "classic" ALPS.

ANALYSIS OF TAILGATE SERVICEABILITY (ALTS)

The chain pillar design methodology proposed by the project is referred to as "Analysis of Longwall Tailgate Serviceability" (ALTS). The design methodology recognizes the impact of ground support on tailgate serviceability and incorporates guidelines in relation to the installed level of primary support and the influence of standing secondary support on the design process.

A design flowchart (figure 10), Microsoft® Excel Workbook, and user manual have been developed. The spreadsheet workbook (*ALTS Protected.xls*) was formulated to facilitate the computational components of the design methodology.

The ALTS design process should only be employed in designing chain pillars that are subject to second-pass longwall extraction. If the chain pillars under consideration are not to be

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subject to second-pass longwall extraction, then an alternative pillar design method should be employed based on pillar stability and outer gate road serviceability requirements. The monitored chain pillar loading behavior (conducted as a part of the project) will assist in estimating the main gate load for design purposes.

The recommended chain pillar width (rib to rib) is contingent upon an appropriate level of primary support. That level of primary support (i.e., $PSUP_L$ to $PSUP_U$) is dependent on (1) the orientation of longwall retreat in relation to the magnitude and direction of the major horizontal stress and (2) the use of standing secondary support along the length of the gate road.

The database is able to identify situations where it is likely that standing secondary support may be required. However, there are insufficient data at this stage to make numerical recommendations for the SSUP similar to those made for the TG SF and PSUP. Appropriately qualified personnel should assess the type, level, and timing of SSUP installation.

CONCLUSIONS

The following main goals of the project were achieved:

- To establish a chain pillar design methodology that has widespread application to Australian longwall operations; and
- To quantify the probable variance in the chain pillar loading environment between collieries and mining localities and to incorporate this variance within the design methodology.

In addition, the study has been able to propose definitive guidelines with regard to the installed level of primary support and to conduct a subjective analysis regarding the impact of standing secondary support on the design process. This provides the Australian coal industry with a truly integrated design methodology with regard to tailgate serviceability that has been able to address the main factors controlled by the mine operator.

The initial benefit from this project is that mine managers and strata control engineers will be able to identify where chain pillars can be reduced in size and where increases may be necessary. They can make these decisions with the confidence that a credible Australian database is the foundation for the design methodology.

This project has identified that there is an opportunity for some mines that do not currently incorporate the routine installation of secondary support along their tailgate to make significant reductions in chain pillar width. It is an operational decision whether a reduction in pillar width is more or less beneficial to production output and costs than the introduction of secondary support along the length of the tailgate. This project simply highlighted that the opportunity exists.

The chain pillar monitoring exercises conducted at collieries under deep cover or with strong roof have found that the

abutment load may be overestimated by using a generic abutment angle of $\beta = 21^\circ$. However, the aggressive tailgate loading behavior monitored at West Wallsend Colliery (figure 5) provided a warning, which emphasized the need to use great caution before making any sweeping changes to a proven chain pillar design tool. Although the way in which the load manifested itself at West Wallsend was significantly different from that proposed by ALPS, the resultant tailgate load was quite similar.

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Figure 10. Design flowchart for the Analysis of Longwall Tailgate Serviceability (ALTS).



Mine Convergence When Using Mobile Roof Supports in Pillar Recovery

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ABSTRACT

To date, there has been limited formal research dealing with the use of Mobile Roof Supports (MRS) for pillar recovery. This paper, which is a portion of a larger project, presents convergence data from 4 different mine case studies.

MULSIM was used to create a numerical model. The resulting information from this model was then compared to the actual measured values.

MULSIM was also employed to analyze the industry-accepted pillar/barrier layout for mining underground reserves with the Joy Highwall Mining System. This layout was examined for both a low cover and a high cover extreme.

INTRODUCTION

Retreat mining using mobile roof supports (MRSs) to remove pillars left during panel advancement is becoming more common as the better coal reserves are being mined out. Other countries in the world have been using this technology effectively to recover pillars in some of their best coal seams. In this decade, the U.S. coal industry has recognized their usefulness and is using MRSs not only to increase a mine's recovery, but also to boost its productivity. With low coal prices, companies are trying to increase recovery and thus decrease the cost of exploration, acquisitions, and development per ton of coal.

Pillar recovery through secondary mining has been used in the coalfields for many years. Applications of MRSs in pillar recovery systems have allowed for extraction of pillars in areas where the use of conventional timber supports was previously thought to be too dangerous or too expensive. MRSs have allowed the mining of areas with a broader range of roof conditions, have increased the productivity of the retreat section, and have increased the safety of the workers.

METHOD OF OBTAINING CONVERGENCE DATA

Convergence readings at a noted time were found by subtracting a measured height of the mine opening from a baseline mine height reading. The base line reading was measured while pillaring activities were far from the measurement site, thus not affecting the reading.

The convergence measurement system consisted of a fixed point on the roof and the floor where accurate readings of the mine height (or pillar height) could be taken. Each convergence station was discrete, to minimize the possibility of mine personnel tampering.

The University of Kentucky Mining Engineering Department extensometer was used for measurements. The extensometer was modified to allow for measurements of up to 9 feet. All of the mines at which data were collected had mining heights within this range. After the instrument was calibrated for 6 feet, the 3 foot extension was firmly attached to the bottom, increasing its measurable range.

In order to measure convergence only, all points were located as close to the ribs as possible. This was to minimize the influence of roof sag and floor heave from introducing error into the convergence measurements. The stations were set in a straight line across the section. Where possible, two stations were placed in each entry, one on the left pillar's rib and one on the right pillar's rib.

Measurements of convergence were taken up to the point where all of the pillars immediately in by the convergence stations were removed. For safety reasons, no measurements were made at the stations when pillaring activities were being performed in the row of pillars containing these stations. As the activities approached the convergence stations, the frequency of recording the readings was increased. On the final day, readings were taken at 1 hour intervals and this continued until mining began in the row of pillars containing the stations. At that point, readings were no longer taken.

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For numerical modeling of convergence, the US Bureau of Mine's MULSIM/NL, a multi-seam stress simulator, was used. The results of numerical modeling were compared with measurements obtained from each mine. The MULSIM/NL input that was used was based on "good engineering judgement." The parameters were not adjusted to create a model that mirrors the actual readings.

CASE STUDIES

Mine A

This mine was located in the West Virginia Lower Kittanning seam. The seam height varied from 60 inches to 90 inches with 40 to 250 feet of cover, which was made up primarily of sandstone. The mine floor consisted of a hard gray shale. The mine was normally operated five days per week, two shifts per day. It produced close to 4000 raw tons per day on retreat, with approximately 50% reject.

Figure 1 shows a 1" = 200' scale portion of the mine map showing the line of pillars in which the convergence stations were placed. The cover in this area ranged from about 70 feet to 180 feet with an approximate average of 140 feet. All entries and crosscuts were around 20 feet in width and the pillars were 60 ft x 60 ft center-to-center distance. According to this mine's approved pillar recovery, all pillars would be removed except for the farthest left pillar, which was used for the gob bleeder entry.

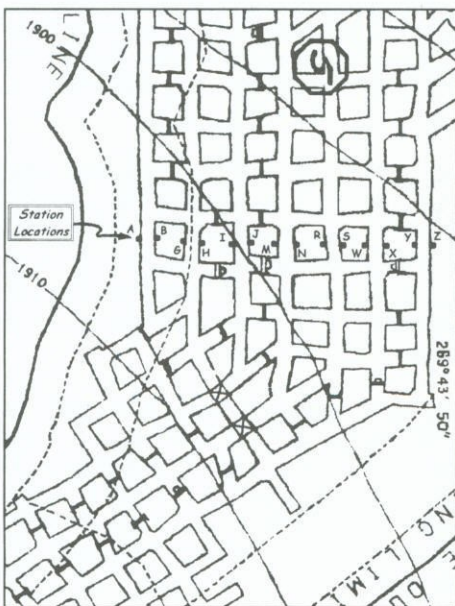


Figure 1. 1" = 200' scale portion of Mine A's map with the location of the measurement sites.

On April 14, 1994, fourteen convergence stations were installed in the locations shown in Figure 1. Pillar extraction activities had already started about one month prior to this date, but the section was still approximately 1/2 mile from the area of the mine containing the convergence stations. On May 5, 1994, the line of pillars containing the convergence stations was removed.

From the readings for Mine A, the convergence was calculated based on the original April 14 mine height readings for each station at various time increments. The difference between the base line reading for a particular station and a daily reading yields convergence for that day.

From this information, the graph of Figure 2 was constructed to show the average convergence for the stations in a straight line across the panel, versus the average time that the readings were taken. As can be seen on this graph, the most common trend was for the stations to show an increased mine height (negative convergence) from the base line (day zero) readings through the first readings taken on day 21 (the last day) of this project. The first readings on day 21 were taken when this mine was finishing the row immediately inby the stations. Therefore, the trend for the stations was for stress relief and consequently, negative convergence from day zero to the beginning of day 21.

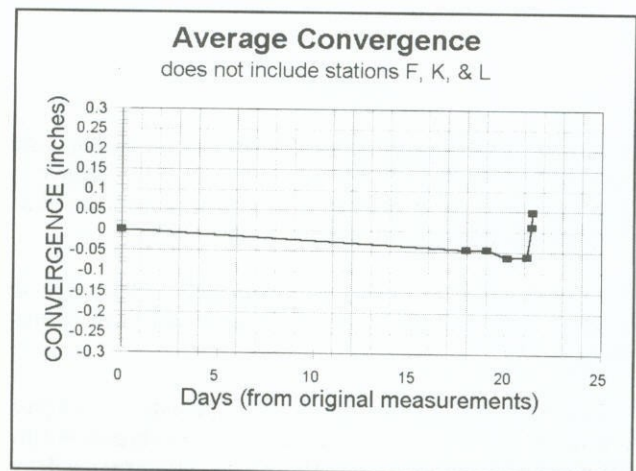


Figure 2. Graph of convergence vs. time for the average of the convergence stations for Mine A.

Under normal circumstances, as a pillar recovery section retreats, the outby pillars should take weight and thereby converge. Also, the surface should subside in a gentle troughing manner based on some angle of draw for the material overlying the coal seam.

Upon first inspection of this interpreted data, one is likely to say that the resulting information at this mine was contrary to convention. This would be true if this mine was a deep cover mine with a homogeneous roof with a contiguous structure. This mine, however, was a shallow mine with a massive sandstone top that was intersected by numerous hill seams. This introduces a roof structure that is comprised of discrete elements, which reacts differently to changing stress fields during secondary mining. By examining the magnitude of the convergence, it can be seen that these readings are zero (far less than even a 1% variation in height). The data that was collected and analyzed points to virtually no movement in the top, and the micro-scale movement that was measured was an increase in height.

The reason for this near zero convergence was the multitude of hillseams in the top. Under normal massive sandstone roof

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conditions, where there are not any hillseams, the roof behaves in a cantilever fashion. During the extraction of pillars in this type of condition, the roof would have a tendency to hang. The massive sandstone top would tend to act as a very rigid cantilever beam, and therefore transfer the stress caused by the pillaring to the outby pillars.

The hillseams, which are cracked all the way to the surface, cause a natural stress relief for this mine. When the pillars are removed, the top falls almost immediately. Stress transfer to the outby pillars is basically zero, because the falls are vertical (following the hill seams) and directly and immediately transferred to the surface. Therefore, there was virtually no cantilever action existing in this mine. A pillar supported the strata immediately above it (plus part of the strata overlying the adjacent entries), while having almost no association with adjacent pillars or the strata that they support. When a pillar was removed, the remaining unsupported roof fell in a large unbroken chunk. Subsidence was immediately transferred to the surface; therefore the readings were basically zero convergence.

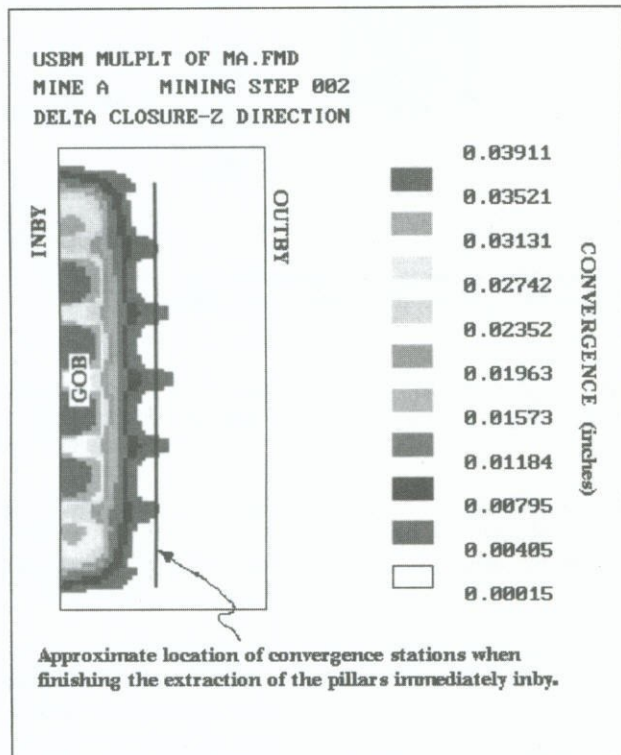


Figure 3. "3-Dimensional" output file from MULSIM that shows the projected convergence caused by pillar extraction.

MULSIM was then run to validate the actual convergence readings taken at Mine A. The behavior of the "hill seams" was modeled with a large "gob-height factor," which is a ratio of the gob height to the seam thickness. Figure 3 shows the "3-D" type of output of the convergence that reflects the mining of the pillars immediately inby the convergence stations. By examining this figure, it can be seen that the projected convergence across the panel at the reference line is very small. MULSIM interprets the convergence to be between 0.00405 and 0.00795 inches in

entries 2 through 6 and between 0.00015 and 0.00405 inches for entries 1 and 7. This projected convergence is negligible, and it corresponds well to the measured convergence for the May 5, 1994 readings at Mine A.

Mine B

This mine was located in the West Kentucky Number 13 Seam. The seam height varied from approximately six feet to eight feet throughout the panel area where the convergence stations were placed. This section was operated an average of six days per week, two shifts per day, with the crews "hot seating" (crews were relieved at the face by the oncoming crew).

Figure 4 shows a 1"=400' portion of Mine B's total mine map, displaying the line of pillars along which the convergence stations were placed. As can be seen, the panel was made up of eight entries with two different size pillars. The far left and right pillars in line with the convergence stations were 100 ft x 70 ft center-to-center distance, while the five middle pillars were 100 ft x 100 ft center-to-center distance.

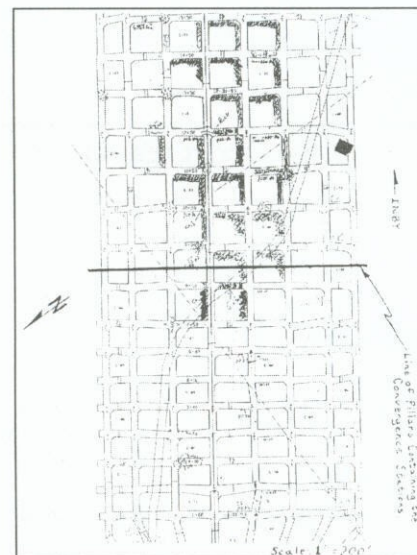


Figure 4. 1" = 400' map of the pertinent portion of Mine B.

This was a below-drainage mine with a vertical depth of approximately 907 feet, as interpreted from a surface drill hole. The immediate roof was comprised of 6 feet of gray shale.

This mine practiced partial pillar extraction and employed the partial fender method. The secondary mining method approved at Mine B was to take a maximum of 20 foot deep lifts on two sides of five of the 100 ft x 100 ft pillars. The external rows of pillars were left untouched to establish the bleeder system.

On June 6, 1994, the convergence stations were installed in the locations shown in Figure 4. This area was 26 breaks (2600 feet) outby the active pillaring section. As was common throughout this mine, the top was in extremely poor condition. The immediate roof was a weak, slicken-sided shale. Falls of the immediate roof were quite common throughout the entire mine.

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Additional, secondary roof control (installed outby the active sections) was added quite often throughout this mine. Although roof conditions at the test site were difficult, they were average for this mine. The immediate floor in this area was comprised of approximately two or three inches of fire clay. Some of the fire clay that originally existed under this coal seam was cut and removed by the advancing section more than one year earlier. Below the fire clay was a hard shale floor. No floor heave was observed in this area, but the mine floor was littered with chunks of slicken-sided shale that had fallen out of the roof.

The goal of this retreating section was to maximize coal production; extraction was not the primary concern. This meant that if there was a fall in an entry that blocked the access to one or both sides of a pillar, then it was skipped. The section finished pillar recovery in this panel on the second shift on June 18, 1994.

From the mine height readings for Mine B, convergence was calculated for each station. This information was used to construct the average convergence versus time for the row of stations for this mine (see Figure 5). As can be seen on this graph, the common trend was for the stations to remain close to zero convergence through day 7. From day 7, there was an increasing positive convergence (decreased mine height), which is congruent with expectations.

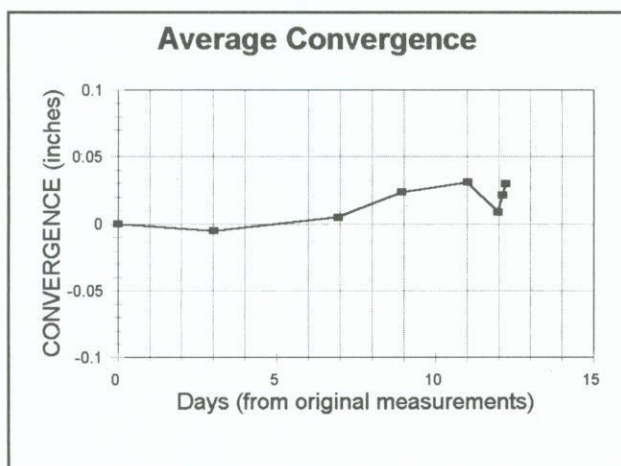


Figure 5. Convergence vs. time for the average value of the convergence stations for Mine B.

The convergence trend from day 11's readings to the first set of readings on day 12 (the last day) is negative. This shows an increased mine height (negative convergence) for that period, showing stress relief. This was most likely caused by a fall occurring and relieving the transferred stress. From day 12 to the end of the project, the convergence trend returns to normal, positive convergence.

The largest absolute (positive or negative) individual station convergence was approximately 0.08 inches (Station C), and the largest average absolute convergence was approximately 0.03 inch. These readings can be considered zero (far less than even a 1% variation in height). The data that was collected and analyzed points to virtually no movement in the top; the micro-

scale movement that did occur followed convention (positive convergence). The reason for this near zero convergence was because the pillars were not being completely extracted.

MULSIM was then run to validate the actual convergence readings taken at Mine B. Figure 6 shows the "3-D" type of output of the convergence that reflects the mining of the pillars immediately inby the convergence stations.

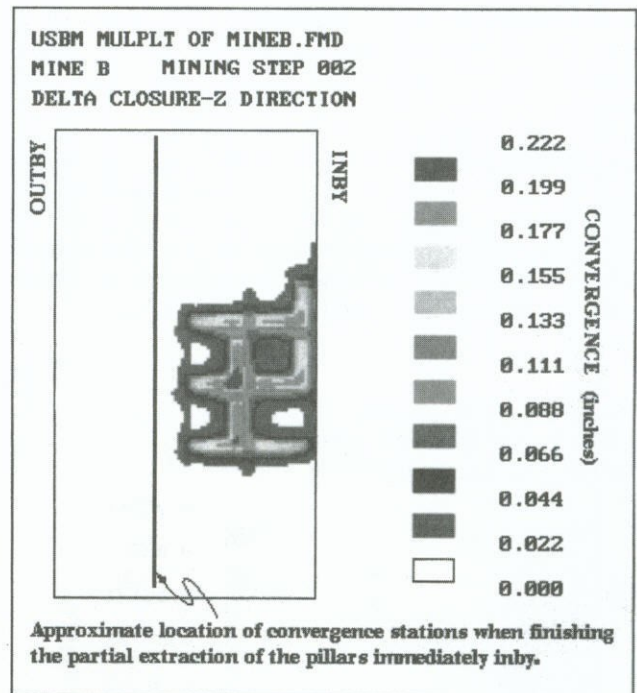


Figure 6. "3-Dimensional" output file from MULSIM that shows the projected convergence caused by partial pillar extraction for Mine B.

The maximum convergence predicted by MULSIM is approximately 0.01 inches across the line of pillars. The field measurements correspond fairly well to the MULSIM output, thus validating the model for the conditions present at Mine B.

The lack of convergence in both the results of the MULSIM output and the field measurements can be attributed mainly to the large stumps of coal left from the partial extraction method employed at this mine. Their size was sufficient to carry the increased loading resulting from the partial pillar extraction over the short time when measurements were taken.

Mine C

This mine was located in the West Virginia Coalburg Seam. The seam height varied from approximately eight to nine feet throughout the panel area in which the convergence stations were placed. This section was operated five days per week, two shifts per day.

Figure 7 shows a 1"=800' portion of Mine C's total mine map, with the two lines of pillars in which the convergence stations

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were placed. In the first line, there were 11 entries containing ten 60 ft x 80 ft center-to-center distance pillars. The second line had nine entries with the same panel configuration.

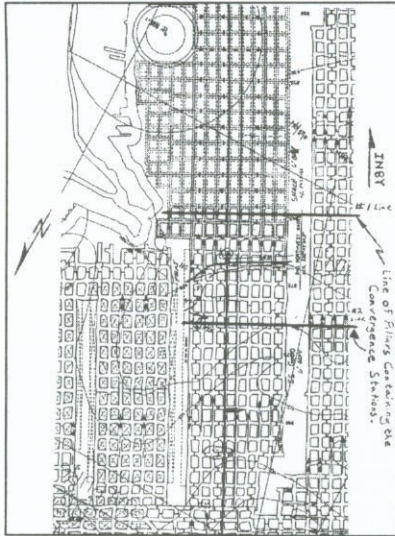


Figure 7. 1" = 800' portion of Mine C's map showing both lines of convergence stations.

This was an above-drainage mine with a vertical cover ranging from 150 feet to 475 feet. An approximate average cover of 300 feet was above both lines of convergence stations. At the first line, the roof was comprised of a hard shale, approximately 15 feet thick. Above this horizon, the roof was mainly layers of sandstone with spars, greatly spaced layers of coal and shale. At the location of the second line, the immediate roof was composed of sandstone with some major joints, "hill seams." This sandstone was the same sandstone bed that was above the hard shale at the first line of stations. The hard shale was not present over the second area due to geologic changes and a fault that existed between the two lines of stations. This allowed the monitoring of the same type of pillar extraction technique in differing geologic conditions.

This mine conducted total secondary extraction of mine pillars. The recovery method used in this panel was the Fishtail (Xmas Tree) Method for rectangular pillars. On November 2, 1994, the convergence stations were installed in the two lines shown in Figure 7. Both areas were a great distance from the active retreating section, operating at the far inby end of the panel.

Figure 8 depicts the average convergence for the stations in Line #1 across the panel, versus the average time that the readings were taken. Close to zero convergence was measured through day 37. From day 37, an increasing positive convergence (decreased mine height) was observed as expected.

The largest absolute (positive or negative) individual station convergence was approximately 0.23 inch, and the largest average absolute convergence was approximately 0.08 inch. From the perspective of the affect on stability, these readings are negligible. Mine C convergence was larger than that of Mines A and B; however, the change in the pillar rib height was still far

less than 1% of the original height. The collected data indicates no movement of the roof. The small scale movement followed convention (positive convergence).

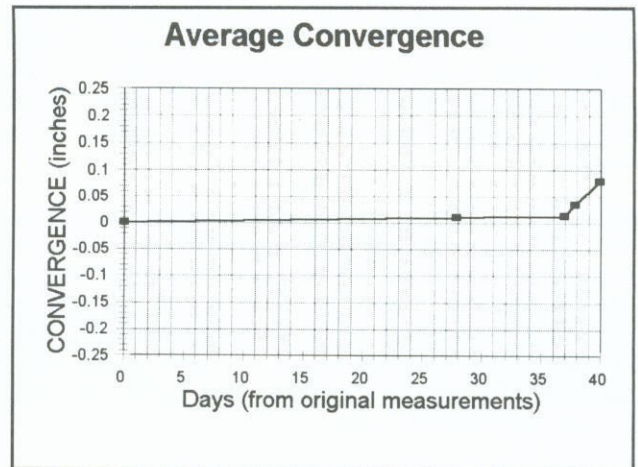


Figure 8. Convergence vs. time for the average of the convergence stations for Mine C, Line #1.

Figure 9 shows the average convergence versus time for Line #2 (outby convergence stations). Again the measurements remain close to zero convergence through day 37. From day 37, there is an increasing positive convergence (decreased mine height), as was expected.

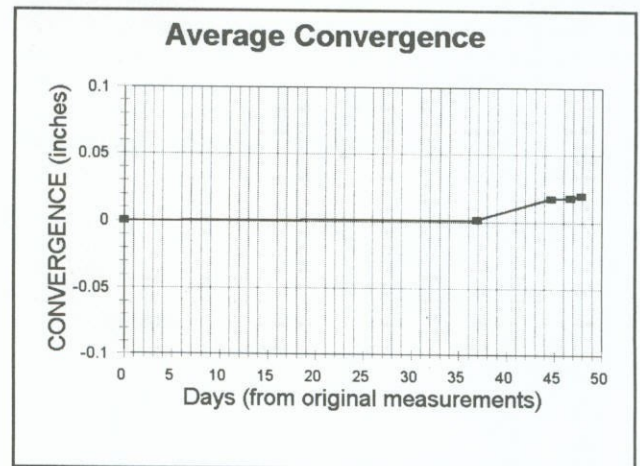


Figure 9. Convergence vs. time for the average of the convergence stations for Mine C, Line #2.

The largest absolute (positive or negative) individual station convergence was approximately 0.07 inch, and the largest average absolute convergence was approximately 0.02 inch. This indicates essentially no movement in the roof. Again, the micro-scale movement recorded followed the trend observed in other mines (positive convergence).

The roof strata for Line #2 resembled the roof strata that was experienced in Mine A. The immediate (and main) roof was a massive sandstone that contained joints, "hillseams." These

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joints were present due to the fault that was immediately inby these points and because this area of the panel was located in the head of a valley. The authors' practical experience has found that hill seams in the roof are more prevalent when mining around the head of a valley. These hill seams were comparable to those observed in similar geographic locations. They were slightly colored by iron oxides from ground water seepage. Such coloration indicates that the fractures reach the surface, which supports the assumption that cantilever roof action will be minimal.

Unlike Mine A, though, the roof falls in the gob were not as massive. In Mine A, the roof fell in huge blocks along the near vertical hill seams as the pillars were mined, while in Mine C, the roof fell in large broken pieces of sandstone. The difference in the character of caving was because the "hill seams" and other stress relief fractures contained in the roof of Mine C were more erratic in their orientation (not vertical). Still, the fractures acted as a natural stress relief for the roof structure.

Figure 10 shows the "3-Dimensional" plot of "Delta Closure-Z Direction." This is a plot of the Mulsim projected convergence for the face area of Mine C, Line #1. The diagram contains a reference line that shows the approximate location where the convergence stations for Line #1 were located when the last pillar was extracted in the row immediately inby the stations. The projected convergence across Line #1 was between 0.02 and 0.04 inches. The measured convergence and the projected convergence are fairly close. The approximate, projected average convergence across the panel is 0.04 inches, and the actual, measured average convergence was approximately 0.08 inches. Even though there is a 100% difference, the numbers are very small. The difference can be attributed to the error in the field measurements and to the error in the creation of the Mulsim model (definition of material properties, definition of the roof behavior and properties, etc.).

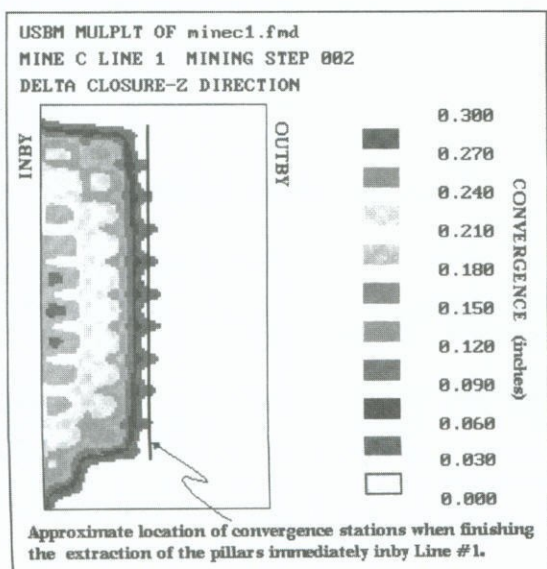


Figure 10. "3-Dimensional" output from Mulsim that shows the projected convergence caused by pillar extraction inby Mine C, Line #1.

Figure 11 shows the "3-Dimensional" plot of "Delta Closure-Z Direction" (convergence) in the area around the convergence stations in Line #2 of Mine C. A reference line shows the approximate location where the convergence stations in Line #2 were located, when the last pillar was extracted in the row immediately inby the stations. The projected convergence across Line #2 is between 0.017 and 0.035 inches, which corresponds well with the December 20, 1994, actual field convergence for these stations, thus validating this model for the conditions present.

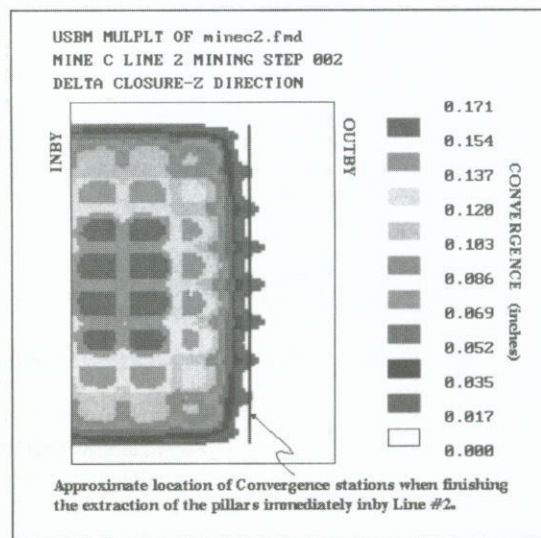


Figure 11. "3-Dimensional" output from Mulsim that shows the projected convergence used by pillar extraction inby Line #2, Mine C.

Joy Highwall Mining System

The purpose of this section is to analyze the mining industry's accepted pillar/barrier layout for the Joy Highwall Mining System. The industry currently uses the same mine layout for both high-cover and low-cover highwall mines. In reference to slope geometry ranges encountered in Eastern Kentucky, two sets of conditions were selected for analysis. Mulsim was used to run simulations on both a high-cover, steep-mountain terrain (Case 1) and a low-cover, gentle-mountain terrain (Case 2) highwall mine layout. Both mining scenarios used the same standard, pillar/barrier configuration. Also, both are modeled as mining reaching 1000 feet into the mountain, which is the maximum mining depth of the Joy Highwall Miner.

Two separate highwall mining scenarios were modeled using Mulsim. Figures 12 and 13 show cross sections of the two conditions that were modeled, which were a steep mountain slope at an angle of 33° above the horizontal (Case 1) and a gentle-mountain slope at an angle of 16° above the horizontal (Case 2). The overburden was divided into three zones. Zones 2 and 3 were run on Mulsim for the two different terrains. The reason for dividing the mine layout into zones was so that different portions of the same mine could be modeled for a representative overburden depth. This had to be done because

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MULSIM analyzes mines with flat, horizontal surface features.

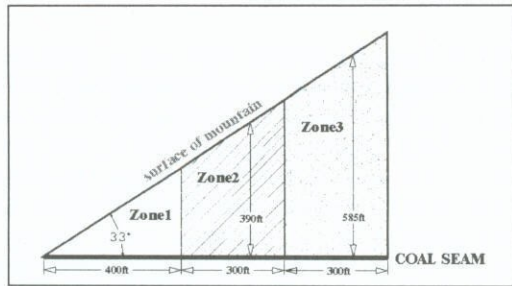


Figure 12. Cross section of steep-mountain terrain, divided into three overburden zones.

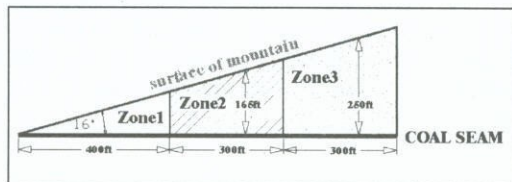


Figure 13. Cross section of gentle-mountain terrain, divided into three overburden zones.

Zone #1 was not modeled on MULSIM. This "outby" 400 foot portion of the mine is under low-cover and close to the outcrop, and therefore, mining stability is governed mainly by the presence of hill seams and not by the depth of the overburden. Zone 2 was the middle, 300 foot portion of the mine. This zone was modeled at an overburden depth of 390 feet (see Figure 11) for Case 1 and 165 feet (see Figure 13) for Case 2. Zone 3 was the final 300 feet of mining, under the greatest mining overburden thickness. The overburden for Case 1 was 585 feet and Case 2 was 250 feet. These overburden thicknesses correspond to two thirds of the maximum thickness in the particular zones. The seam thickness was assumed to be 48 inches.

The industry accepted mine geometry is to mine five, 10 foot wide entries with a 4 foot wide strip of coal left between each entry. After every set of five entries is mined, a 10 foot wide barrier pillar is left. This geometry cycle is then repeated. The mine layout modeled in MULSIM was constructed to reflect the situation in the 5-entry mining cycle, for which the mine's stability conditions are tested. This is when the highwall miner has mined to the 1000 foot depth, in the last (5th) entry in the five-entry cycle.

The MULSIM output plots for the normal stress are shown in Figures 14 (steep-mountain terrain) and 15 (gentle-terrain). By examining these figures, it can be seen that the normal stress of the steep-terrain is approximately twice as large as that for the gentle-terrain. Based on purely stress analysis, this shows that the gentle-terrain mine is over-designed, assuming that the new technology in the directional control of highwall miners ensures the ability to maintain parallel pillars of uniform thickness.

The long coal pillars that are left to support the top in the shallow cover, gentle-terrain mine, could be downsized. They

should develop the same stress as the pillars in the mines under steep terrain, and the benefit is the increased recovery of the coal reserves. The current recovery for the accepted mine geometry is approximately 65%. If the small pillars were decreased by 2 feet in width (2 foot total width), the recovery would be increased to approximately 74%.

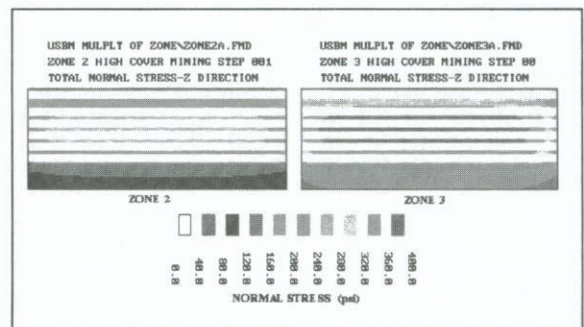


Figure 14. MULSIM normal stress plots for Zone #2 and Zone #3 for the steep mountain terrain.

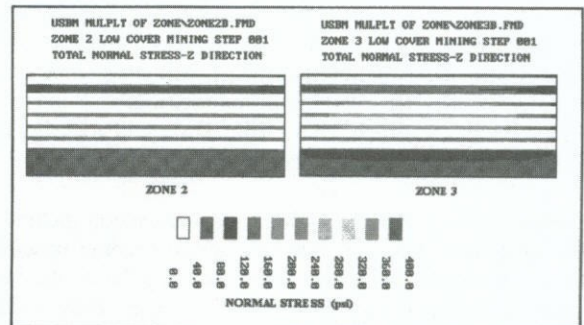


Figure 15. MULSIM normal stress plots for Zone #2 and Zone #3 for the gentle mountain terrain.

CONCLUSIONS AND RECOMENDATIONS

The important finding from both the field measurements and numerical analysis was that the convergence was very small. This showed that the mining systems used were well suited for their conditions. This minimal strata movement showed that the face area, where personnel were working, was not approaching a hazardous mine stability condition. MULSIM was found to be a fairly good tool for predicting the convergence of the pillars for the mining conditions found at the three mines. Also, MULSIM seems to be an effective design tool for modeling the conditions resulting from pillar extraction.

Larger convergence numbers should be attainable in full extraction, deep-cover mines. In this type of condition, though, one must keep in mind that the arching effect of the roof strata will limit the magnitude of the convergence readings, unless the extracted panel is wide enough so that the pressure arch intersects the surface. Therefore, narrow deep-cover panels will likely show smaller convergence than a wide deep-cover panel.

The full capabilities of Mobile Roof Supports (MRSs) have not been fully utilized by most mines. The mines that were studied



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seemed to use the MRS units to extract pillars, without planning the advance mine pillar configurations to maximize recovery and the utilization of the MRS units. If the pillar configurations were modified to make pillars longer, retreat mining with the MRS units would be able to extract a higher percentage of the total reserves. It also reduces the expenses for roof support and thus, decreases the cost per ton. This would slow the advance rates of the continuous miner development sections, but the MRS retreating section should more than make up for such slower advancement. If conditions allow, the best plan is to drive long pillars that are approximately 40 feet (actual dimension) wide. This allows for the MRS section to extract the pillars using the Fishtail Method. The Wongawillie and the Sigma-Ben methods of retreat mining seem to be the best methods for utilizing the MRS units. It is essentially an inexpensive shortwall method of mining. The problem associated with this system does not stem from stability, but from U.S. and state laws that govern the maximum length of entries prior to turning a crosscut.

Mine B used a partial pillar extraction method. Mining observations and measurements, as well as carried out analysis on MULSIM, proved that stability conditions were maintained at the appropriate level. The Mine B system decreased the pillar size but did not change the pillar shape. The authors believe that partial pillar extraction is potentially more dangerous than full extraction and should be avoided. The above statement applies especially for the practice of pillar splitting.

The problem that can occur with a partial extraction method is that the resultant pillars are overstressed and become unstable. Partial extraction can cause an initially stable pillar to become, after splitting, a borderline stable/non-stable pillar. The problem develops after some time, when one of these partially extracted pillars suddenly fails. The stress is transferred rapidly to the neighboring pillars, which have also been partially extracted. These pillars are unable to withstand the new load and they fail. The result is a "domino effect" and a failure of all of the pillars, which can cause a dangerous air blast.

The recorded cases of rapid failure of an area of partial extraction happened when pillar splitting was used. Pillar splitting creates a problem that other partial extraction methods do not have. Pillar splitting leaves two small (narrow) pillars. These two small pillars have their width to height ratio decreased (w/h), which is related to the confinement of the core of the pillar. The panel stability is not only decreased by reducing the amount of supporting coal, but also by reducing the pillar strength as controlled by the w/h factor.

The best method of retreat mining is to fully extract all of the material. This allows for caving to take place, and subsequently, stress relief occurs in a controlled environment.

One should pay extra attention to the intersections during pillar extraction. Intersections have a larger critical roof span than the entries. These potentially hazardous intersections might be supported with cable or truss bolts (Vervoort and Prohaska, 1991), which could improve the stability of the area by supplying more confinement to the roof strata. Such reinforcement should be considered during the design of the strata control, so that this

supplemental support can be added during the section advance, by the section roof-bolting machine.

The MULSIM program predicted the mining conditions well for the pillar extraction sections where measurements were taken. Therefore, its application seems to be warranted in other areas of mining technology. The Joy Highwall Mining System is one such area. It has been demonstrated through the performed analysis how current accepted standards of Highwall Mining layouts can result in an over designed mine, causing under utilization of some coal reserves. As mining continues, more and more of the easy reserves will be depleted. Methods of extending a coal reserve's life are becoming increasingly more important, and the Mobile Roof Supports and Joy Highwall Miners are becoming more commonly used mining equipment applied in present mining systems. Full study of roof and floor behavior and improved methods of predicting this behavior is recommended to enhance both mine safety and mine profitability.

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The Advance and Relieve Mining Method: A Horizontal Stress Control Technique

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ABSTRACT

Sacrificial entries, roof slotting, and other tactics have been used to combat high horizontal stresses during roadway development in U.S. coal mines. In Australia, the "pillar extraction on the advance" mining method has been successfully employed as a horizontal stress control technique. The concept of advancing and relieving mine workings through pillar extraction developed from observations that some mines experience difficult ground conditions when advancing and retreating the first panel in a new reserve. However, conditions dramatically improve in subsequent panels. Apparently, the creation of a gob area stress relieves the adjacent panel workings. After much debate and close scrutiny, the first U.S. roof control plan to advance and relieve was approved in 1998, for the Sargent Hollow Mine in Wise County, VA. The major concern was that this mining practice places section personnel in by of the gob. The tentative approval permitted barrier pillar slabbing and removal of the adjacent development pillar during panel development. The intent of the plan was to provide a "stress shadow" for the advancing faces and outby workings to reduce the excessive number of roof falls. Sargent Hollow successfully extracted their first panel by pillaring on advance and retreat mining using posts. Mobile roof supports were employed to extract pillars while advancing the second panel. This paper summarizes Sargent Hollow Mine's experiences with the advance and relieve mining method.

INTRODUCTION

Several U.S. coal mine operators have come to the realization that underground mining conditions are highly dependent upon roadway orientation. For example, in the Eastern U.S., virtually all longwall panels are oriented approximately east-west, and very rarely north-south. Another

situation, sometimes referred to as "first panel syndrome," has also plagued numerous longwall and room-and-pillar mines. With first panel syndrome, extremely adverse ground conditions are experienced when developing and retreating the first panel in a new reserve. However, at many of these operations, favorable ground conditions were experienced in subsequent panels. Over the past two decades, researchers have determined that adverse ground conditions associated with both of these phenomenon are caused by high horizontal stresses (1, 2).

In mine workings subjected to high horizontal stresses, many operators have also experienced roof damage in the entry which intersects the horizontal stressfield first. However, satisfactory ground conditions were encountered in the adjacent entries. This fact, coupled with first panel syndrome, indicates that horizontal stresses can only be fully transmitted through intact rock. Roof falls, fractured ground, and gob areas apparently stress relieve or "stress shadow" nearby workings (3). Based on these observations, mine operators have employed sacrificial entries or caving chambers (4), roof slotting, and other tactics to stress shadow adjacent workings. As the name "sacrificial" implies, the entry which enters the stressfield first, sustains roof damage and is sacrificed for neighboring entries (figure 1). This strategy has been employed to help insure stability in headgates, tailgates, and other critical entries. With roof slotting, a cutting machine is used to cut a vertical slot into the roof near mid-entry. This technique was tried at a Northern West Virginia operation. The 15-cm (6-in) wide slot closed 7.5 cm (3 in) within two days after cutting it. In an Illinois coal mine, the 15-cm (6-in) wide slot converged 7.5 cm (3 in) within 7 days of cutting it (5). The pillar extraction on the advance mining method has also been employed to combat ground control problems associated with high horizontal stress in Australia (6). With this method, pillars on one side of the panel are extracted as the panel is developed. The intent of the plan is to stress shadow the advancing faces and outby workings in order to reduce the occurrence of roof falls.

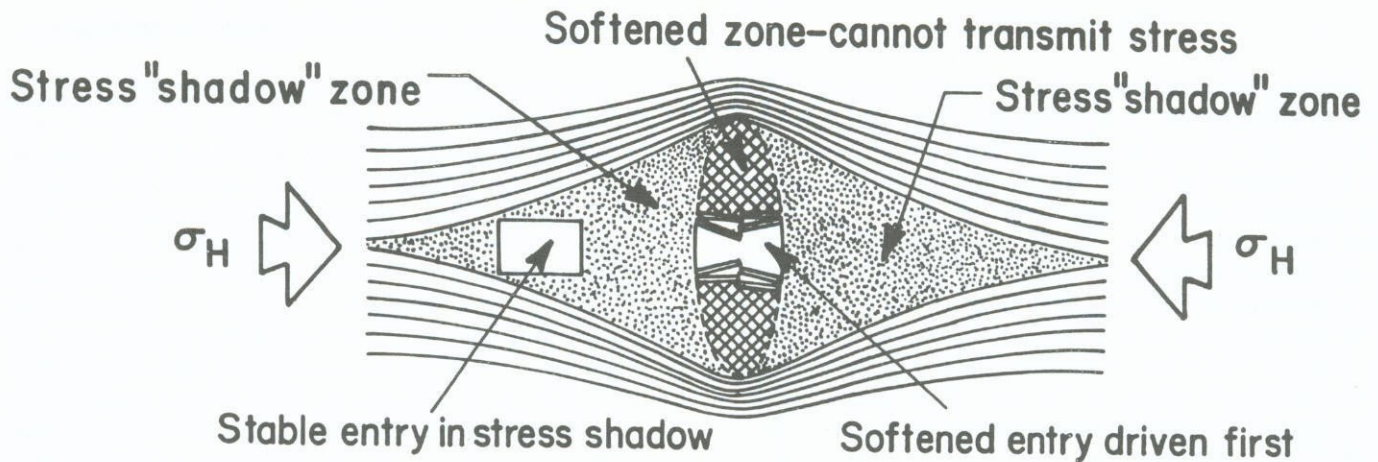


Figure 1. Development of a stable entry in the stress shadow of a sacrificial entry (2).

Over the past two decades, researchers from the National Institute for Occupational Safety and Health (NIOSH) and the former U.S. Bureau of Mines have investigated the problems associated with high horizontal stresses in underground coal mines (7). During this time, numerous case histories documenting successful and unsuccessful mining designs and methods to mitigate stress related problems have been determined (2, 8). Another opportunity to collect an unique case history developed when a series of unintentional roof falls and related injuries occurred at the Sargent Hollow Mine. At the request of the mine operators and the Mine Safety and Health Administration (MSHA), NIOSH personnel conducted underground geotechnical evaluations to determine the causative factors of these falls. The initial mine visit indicated that the

weak roof strata was being subjected to and damaged by high horizontal stresses. After discussions, the mine operators and the Virginia Department of Mines, Minerals, and Energy (DMME) personnel, and Sargent Hollow Mine operators expressed an interest in employing the novel advance and relieve mining method. Subsequently, the mine operators requested NIOSH assistance in determining the maximum horizontal stress direction, and also to both monitor and evaluate the mining methods effectiveness. It was readily apparent that if this mining method proved to be successful in reducing the occurrence of stress related roof falls, that its applicability to other mines facing similar conditions might prove to be wide ranging.

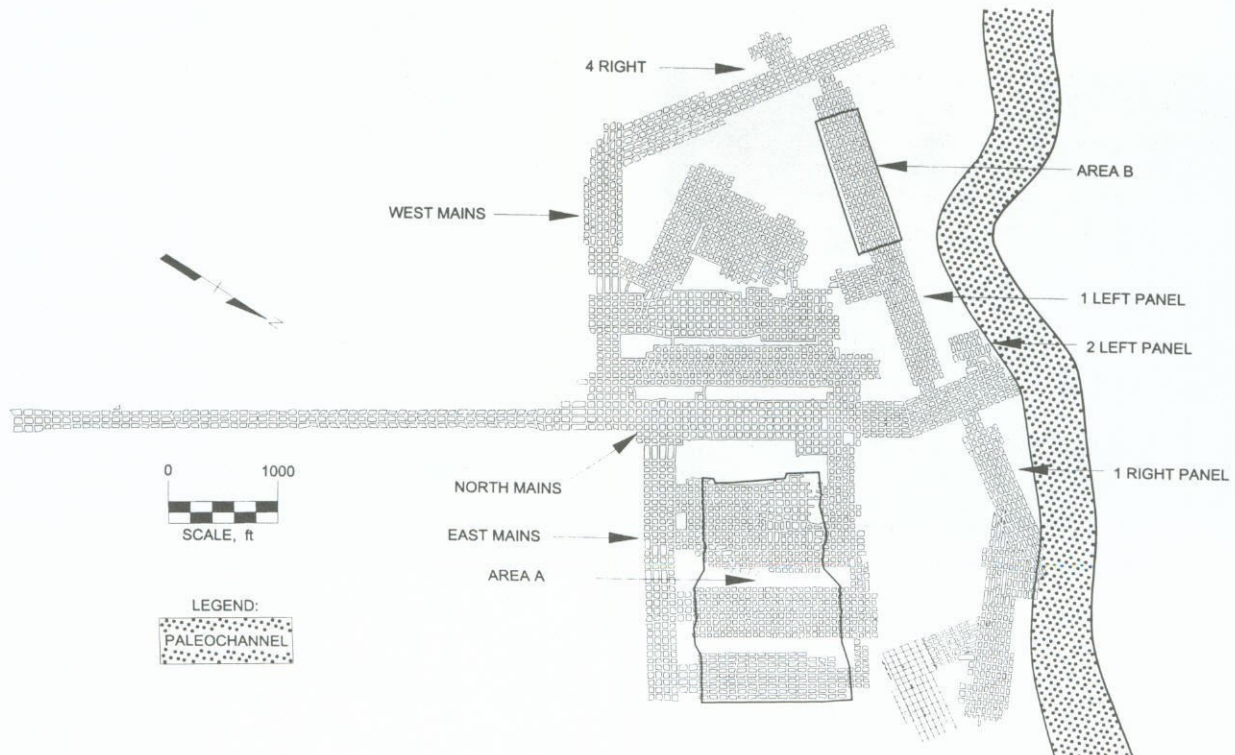


Figure 2. Sargent Hollow Mine map.

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MINE SETTING

Sargent Hollow mine is located in Wise County, VA. The mine is extracting the 1.1-m (42-in) thick Blair Coalbed. The Blair Coalbed is notorious for having poor roof conditions, and several operators have been unsuccessful in their attempts to mine adjacent reserves. In order to access the mineable reserves, mine operators drove four, 1,525-m (5,000-ft) long mainline entries (figure 2) from the highwall through coal and rock. The roof rock at Sargent Hollow is highly variable. In portions of the mine, the immediate roof is a rooted mudstone (underclay) overlain by a 2.5-10-cm (1-4-in) thick rider coal. In other areas of the mine, a more competent shale occurs immediately above the coalbed. The Coal Mine Roof Rating values (9) for the former is typically in the low 30 range, and in the low 40's for the latter. Considerable channel activity also occurred contemporaneously with both peat and roof sediment deposition at Sargent Hollow Mine. This activity is evidenced by in-seam partings (splits), alternating thin layers of sandstone, shale, and coal (stackrock), and paleochannels (washouts or wants). Each of the above mentioned roof rock types, except for the massive sandstone channel deposits, are highly susceptible to horizontal stress damage. This was evidenced by the frequency of unintentional roof falls, and the regular occurrence of cutter roof failures (figure 3), ribline guttering, and stress roof potting. High horizontal stress conditions were also suggested by "first panel syndrome" which occurred in the first of a series of panels driven off the East Mains (figure 2, Area A). Overburden in Area A ranged from 91 to 206 m (300 to 675 ft). Under the deepest cover, the Analysis of Retreat Mining Pillar Stability Factors (ARMPS SF) were 1.1 or greater (10).



Figure 3. Cutter roof failure.

ADVANCE AND RELIEVE MINING METHOD

Due to the high frequency of unintentional roof falls and related injuries, discussions were held with roof control specialists from MSHA, DMME, and NIOSH to consider the advantages and risks associated with an advance and relieve mining method. The overlying concern was that if a gob fall over-rode the breakers into the working faces, the incident could jeopardize the safety of section personnel. Although a similar concept had been successful in Australia, it had never been formerly tried in the U.S. On June 2, 1998, a tentative approval, pending results of an in-mine evaluation by MSHA and DMME, to advance and relieve was granted to Sargent Hollow Mine. The test site was located in 1 Left Panel off the North Mains (figure 2, Area B). In 1 Left Panel, eight entries were driven on 15.2-m (50-ft) centers as shown in figure 4. The crosscuts were driven on 21.3-m (70-ft) centers, and the roadways were approximately 6.1-m (20-ft) wide. Because of the section's ventilation plan, entries were advanced from right (number 8 entry) to left. In the roadways, primary support consisted of 1.8-m (6-ft) tensioned rebar bolts (# 6 rebar grade 60) which were installed with a 0.6-m (2-ft) equivalent of grout. In the intersections of number 1 and 2 entries, and other areas where conditions warranted, 2.4m (8-ft) tensioned rebar bolts (# 6 rebar grade 60) were used. Cover over 1 Left Panel varied from 198 to 274 m (650 to 900 ft).

NIOSH personnel stress mapped (8) several miles of underground workings to determine the horizontal stressfield direction. Knowledge of this direction is critical in order to determine an optimum panel orientation and correct pillar plan (figure 4). Based on this mapping, the maximum horizontal stress (σ_H) direction was determined to be ENE, which is in agreement with the regional stressfield (11). Figure 4 illustrates one preferred panel orientation with respect to the horizontal stressfield, to maximize the workings being advanced into the stress shadow zone. As indicated in figure 4, the number 1 entry will intersect the stressfield first, and therefore, pillaring should be conducted on the left side of the panel to produce the stress shadowing effect. Figure 5 depicts the approved timber plan and cut sequence for Sargent Hollow's advance and relieve plan. As shown on figure 5, five lifts, outby to inby, were taken from the number 1 entry to slab the barrier pillar. Similarly, three lifts are then removed from the adjacent development pillar from the number 2 entry. A final lift is then taken from the development pillar from the inby crosscut. Prior to barrier and production pillar extraction, the roof control plan stipulates that the working faces must be developed at least two crosscuts inby the area to be pillared, as shown in figure 4.

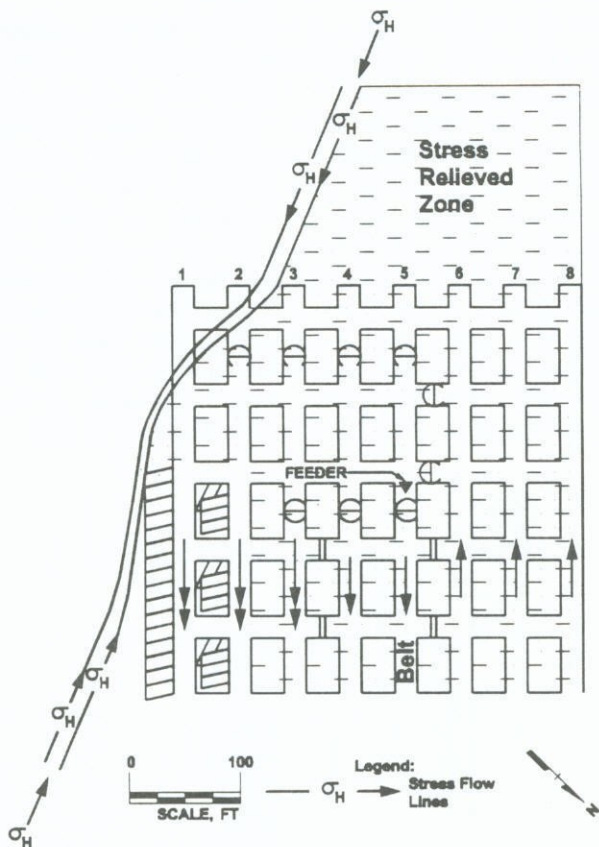


Figure 4. Layout for 1 Left Panel showing stress relief zone.

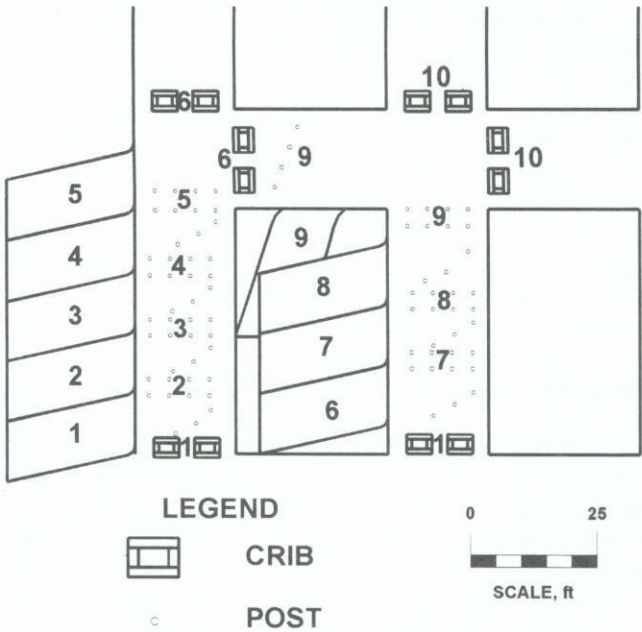


Figure 5. Timber plan for 1 Left Panel.

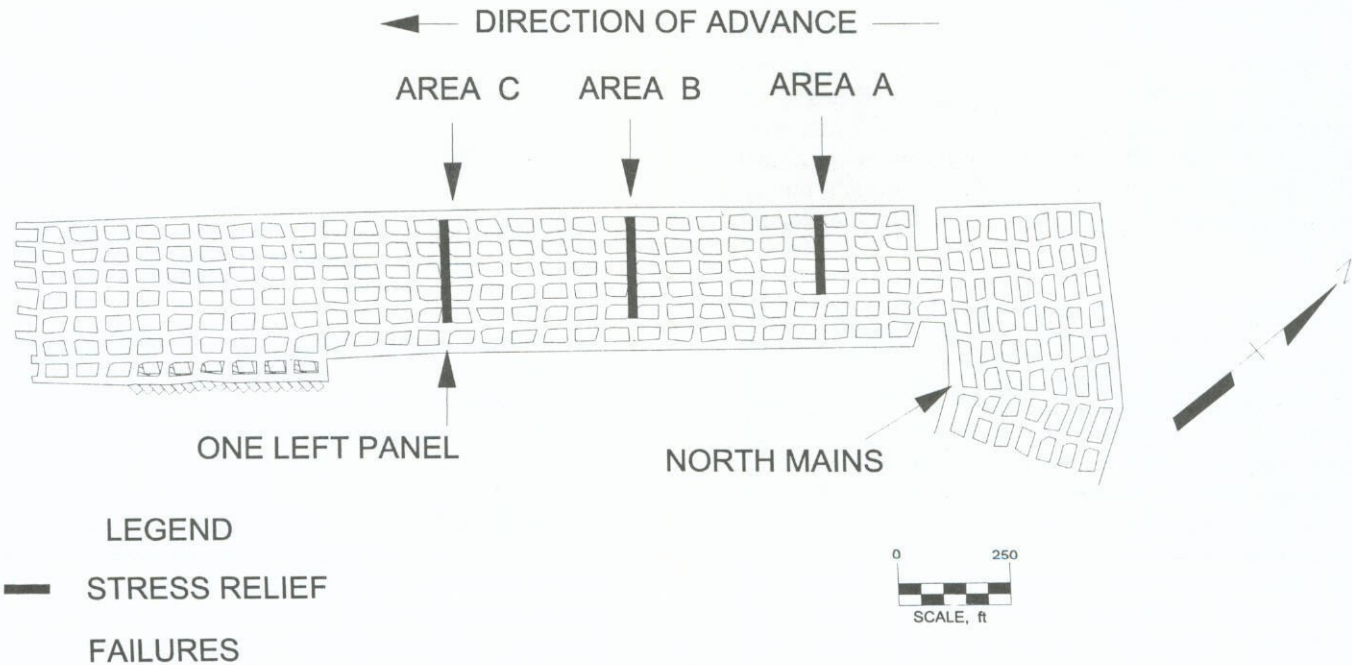


Figure 6. Crosscut stress relief failures in 1 Left Panel.

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As was expected, the number 1 entry sustained considerable damage on development, causing difficult conditions during barrier slabbing because the entry was located in a stress concentration. Conversely, excellent ground conditions were generally encountered on the right side of the panel, especially in the number 8 entry, because the workings were advanced into the stress shadow zone. One interesting note is that prior to instituting the advance and relieve plan, 1 Left Panel experienced incremental increases in loading after each successive row of crosscuts were established. The section continued to load up until it had been advanced eleven crosscuts, at which time, the workings located eight crosscuts outby began deteriorating and breaking up (figure 6, Area A). The area was immediately spot bolted using 2.4-m (8-ft) bolts, and the crosscuts were heavily cribbed. Some cribbing was also necessary in the adjacent entries. Tension cracks and cutters continued to develop, and excessive roof spalling also occurred. The roof appeared to be broken up and failed, and was most probably resting on the heavily loaded cribs. Shortly afterward, the entire section appeared to be stress relieved. The section was advanced eight more crosscuts and the same pattern of loading conditions and roof damage occurred again. This time the damage occurred in the workings located six crosscuts inby of the first area affected (figure 6, Area B). When the scenario repeated itself a third time (figure 6, Area C), it became obvious that some type of periodic horizontal stress buildup was developing. Periodic horizontal stress buildup, in one form or another, has also been observed in other coal mine operations. NIOSH personnel were on the section to observe the next cycle. However, the outby crosscut workings did not load up or fail as was expected. The most likely explanation was that the newly implemented advance and relieve plan had de-stressed the outby workings. In fact, the pillaring had been started just two crosscuts outby the crosscut location of where the next damaged workings were expected. The ground control problems which occurred in the number 1 entry and in the crosscuts were bothersome, but not overwhelming. After the advance and relieve mining method was implemented, overall mining conditions at Sargent Hollow improved, and the roof control plan was approved for further use.

After intersecting 4 Right Panel off the West Mains (figure 2), 1 Left Panel was retreat mined. An ARMPS SF of 1.0 or greater was maintained during panel retreat. Development of 2 Left Panel (figure 2) began in October 1998. Even more favorable ground conditions were anticipated in this panel because of the stress shadowing provided by the adjacent gob in 1 Left Panel. However, rapidly changing geological conditions proved to be the unknown variable at Sargent Hollow. A thick in-seam parting and poor roof conditions associated with a nearby paleochannel were encountered, and mining was terminated in the 2 Left Panel. Mining then began in the 1 Right Panel (figure 2). Once more, very favorable mining conditions were experienced with the advance and relieve mining method. Two of the reasons for this success was increased pillar recovery and more rapid panel advance due to mobile roof support (MRS) usage. MRS's were used in lieu of turn posts and cribs during pillar extraction (12). Observations of the gob area where the

timber plan had been used in 1 Left Panel (figure 5) indicated that only partial caving was occurring in the immediate roof rock to an approximate height of 1.5 - 1.8 m (5-6 ft). It was obvious that the pillar remnants, posts, and cribs were inhibiting complete caving. With MRS's, only eight entry breaker posts are required to slab cut the barrier per crosscut, and total of 16 entry and crosscut breaker posts are necessary to extract the production pillar (figure 7). It was evident from observations back into the gob that more complete and higher roof caving was occurring in the 1 Right Panel. In fact, roof caving intersected the sandstone paleochannel at one location. Gob caving was so complete that the channel was broken into large units, some of which were 1.8-m (6-ft) thick. Figure 7 indicates the cut sequence for pillars driven on 28-m (90-ft) crosscut centers.

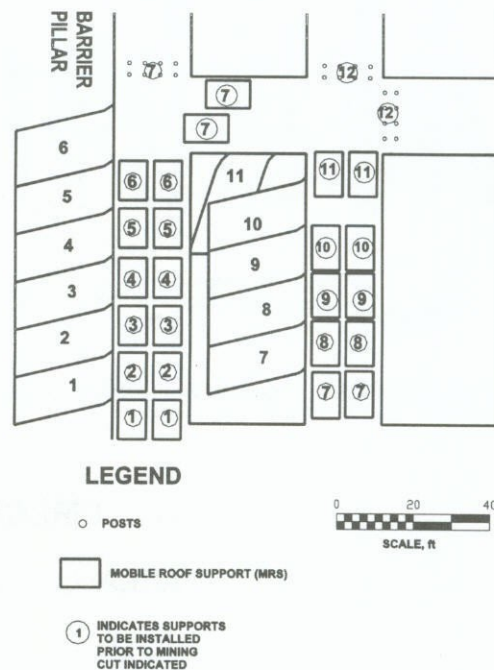


Figure 7. Approved plan for single development pillar extraction using MRS's.

Development continued in 1 Right Panel until another thick in-seam parting and poor roof conditions associated with a nearby paleochannel were encountered. The panel was then reoriented to a direction adjacent and parallel to the trend of the parting and the channel (figure 8, Area A). During this realignment, barrier and production pillar extraction were halted. As a result, the section began experiencing severe loading, and additional time was lost setting cribs. In an attempt to stress relieve the workings, a modified roof control plan approval was granted to extract the pillars shown on figure 8 in Area A. Unfortunately, pillar removal failed to mitigate the problem, and four large falls occurred over a 24-hour period. After turning the panel, the advance and relieve plan was reinstated. Semi-favorable mining conditions were encountered for the next four rows of crosscuts. At this point, difficult ground conditions started occurring in the number 1 entry and pillar recovery was

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incomplete. In January 1999, Sargent Hollow received an approval which allowed for the extraction of the barrier and two production pillars on advance (figure 9). The reasoning behind this plan was that enhanced stress shadowing would occur on the section because a wider pillared out area would induce higher roof caving. Because of the location of the power center, it was necessary to develop another entry on the left side of the panel to provide the additional pillar to be removed. A less favorable panel orientation with respect to the maximum horizontal stress direction coupled with the time required to develop the additional heading proved costly. As previously mentioned, pillar recovery was incomplete immediately outby, and the number 1 entry was already beginning to load up. By the time pillaring could begin, section personnel deemed it unsafe to try to recovery the two pillars next to the barrier as shown on figure 8, Area B. This only aggravated the problem and the section began experiencing severe loading immediately outby of the working faces. Cutters and tension cracks started ripping the roof, and the pillars began spalling and popping indicating that

they were taking weight. Section personnel continued to address the problem by cribbing extensively until conditions permitted barrier and production pillar extraction. Within a matter of minutes after extraction was completed, NIOSH and mine personnel observed a noticeable improvement in ground conditions, giving further credence to the advance and relieve plan. It became readily apparent after this experience that mining operations must remain on cycle, failure to systematically stress relieve the workings through pillar extraction invited unnecessary problems. It is important to note that after 1 Right Panel was reoriented, optimum directional mining with respect to the maximum horizontal stressfield was diminished because the section was driven approximately parallel to the maximum horizontal stressfield. This less than desirable orientation reduced the number of working faces being advanced in the stress relief zone, and also placed the crosscuts in an unfavorable direction (nearly perpendicular) with respect to the stressfield. The adverse conditions encountered underground confirmed that this mining direction produced unacceptable results. The cover over 1 Right Panel ranged from 190 to 297 m (625 to 975 ft).

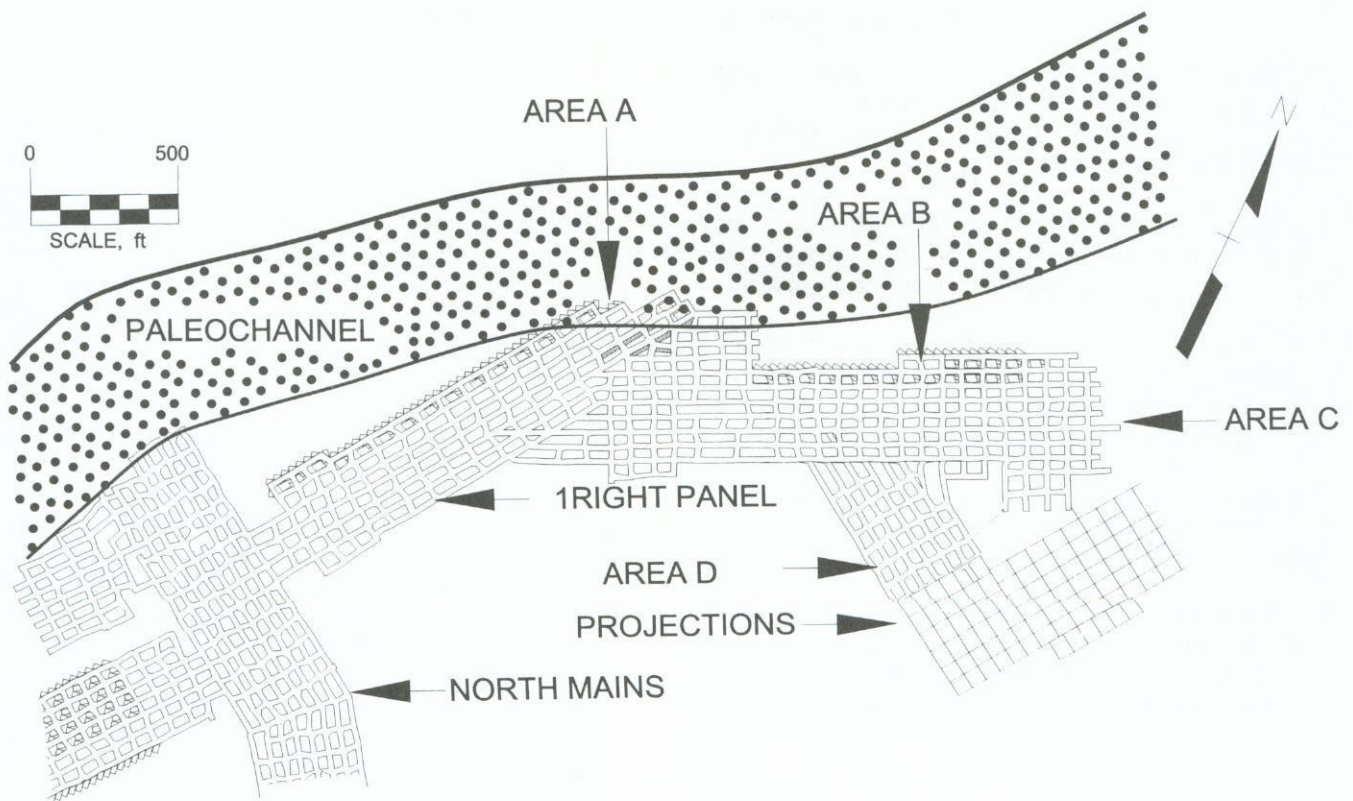


Figure 8. Layout for 1 Right Panel.

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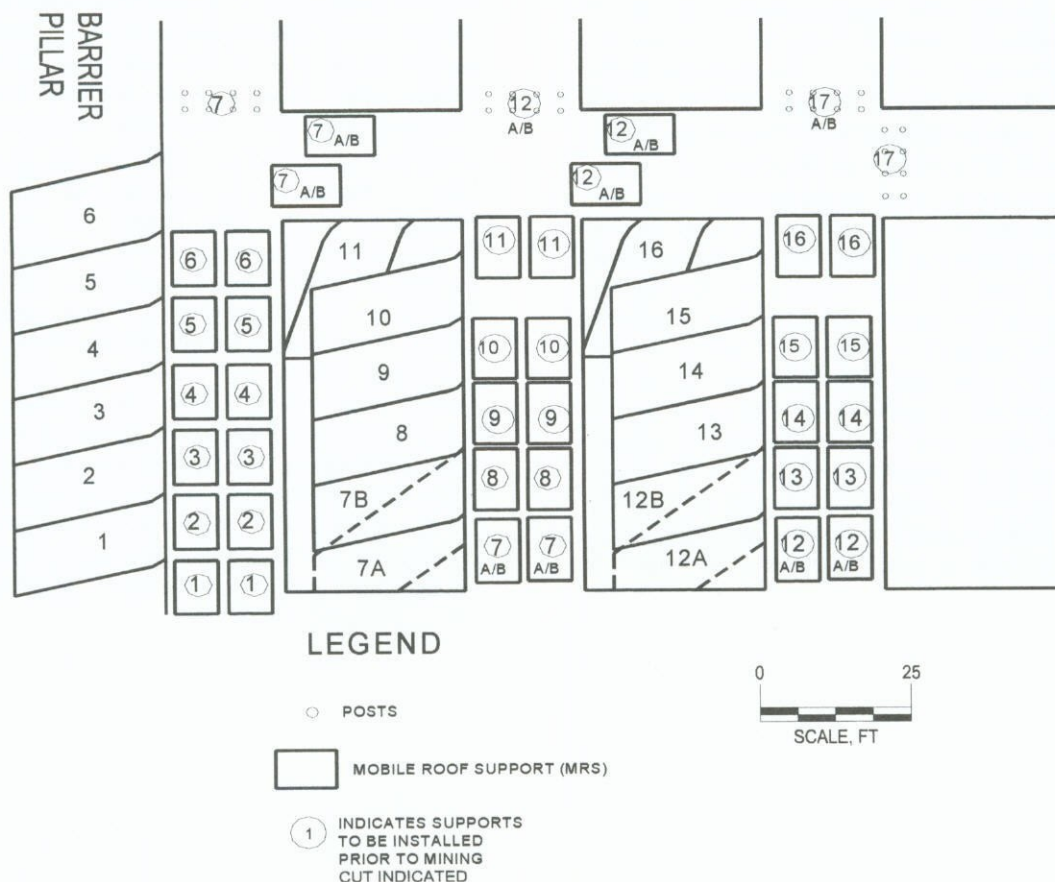


Figure 9. Approved plan for dual development pillar extraction using MRS's.

It should be indicated that MSHA personnel also conduct an independent underground study of the advance and relieve plan conducted at Sargent Hollow Mine (13). The most significant findings of this investigation include:

1. Difficult mining conditions occurred in several areas where the depth of cover exceeded 850 ft. It is probable that vertical stress also contributed to poor ground conditions.
2. Barrier and single pillar extraction may have been beneficial in improving ground conditions at Sargent Hollow Mine. It is not clearly evident that the favorable mining conditions which occurred in the areas where the advance and relieve mining method was used were due solely to stress relief. Changes in roof lithology and/or higher pillar stability factors in low cover areas could have been the primary factors for improved conditions.
3. Roof deterioration index and stress mapping indicated that conditions worsened when dual production pillar extraction was employed. Dual pillar extraction might be detrimental. One reason for this might be added abutment load transfer.

DISCUSSION

Difficult ground conditions associated with a paleochannel in the roof was most probably a key factor in halting 1 Right Panel development (figure 8, Area C). In fact, oppressive mining conditions occurred in each location where the channel was encountered (2 Left Panel, North Mains development, and 1 Right Panel, Area A). Poor mining conditions in proximity to paleochannels are attributable to two factors. First, channel margin geology is typically composed of weak rock types (14) which are extremely susceptible to high horizontal stresses. Second, there is a general consensus that where the roof strata are subjected to high horizontal loads, that stress concentrations occur along the stiffer sandstone intrusion (15). Although this has not been confirmed by direct measurements in the U.S., British work indicates that horizontal stresses are higher in stiffer strata such as sandstone (16). Many coal mine operators have experienced problems with channels similar to those at Sargent Hollow, even where dealing with small channels. As shown in figure 8, Area D, 1 Right Panel was reoriented away from the channel margins to access the reserves. After the panel is turned, the advance and relieve plan will be reinstated in a more favorable orientation with respect to the maximum horizontal stressfield. In order to keep the section advancing in a timely manner, only one production pillar will be removed on development. Where the mining was terminated in 1 Right

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Panel (figure 8, Area C), the cover was 297 m (975 ft) and the pillars had an ARMPS SF of 1. Although this stability factor is lower than the suggested 1.5, analyses of case histories in the ARMPS data base in other mines where the overburden exceeded 200 m (650 ft) indicate that pillars with equivalent stability factors have provided satisfactory ground conditions. NIOSH is currently investigating this and other issues pertaining to deep cover pillar recovery.

As indicated earlier in the paper, NIOSH and mine personnel stress mapped miles of workings to determine the maximum horizontal stress direction. Micro stress damage features such as bolt hole shifting and cutter and pot orientations were inconclusive in determining the maximum horizontal stress direction in both 1 Left and 1 Right Panels. The ENE compressional stressfield which was identified in 1 Left Panel was based on two macro features. Namely, the failed roof zones which primarily occurred in the crosscuts. The other factor which was used to establish the orientation was the stress concentration damage which developed in the number 1 entry. These same two stress damage macro features were also observed in 1 Right Panel (figure 8, Area B), where the sections orientation had been changed approximately 25° as compared to 1 Left Panel. Based on the differing orientations of the macro stress damage features in 1 Left and 1 Right Panel after reorientation, it appears that the maximum horizontal stress direction may not be well defined over this portion of the mine property. As previously indicated, in order to maximize the workings being advanced into the stress shadow zone, the panel should be correctly oriented with respect to the maximum horizontal stressfield. The above mentioned facts suggests that maximum stress shadowing may not have occurred in the working faces. However, the outby workings were stress relieved, and this was most probably a contributing factor in the noticeable reduction of outby roof falls.

Another topic which warrants discussion in this paper is panel width. Obviously, the more narrow the panel, the faster it can be both developed and stress relieved. This has led Sargent Hollow Mine Operators to opt for single development pillar extraction using MRS's. As indicated earlier, Sargent Hollow began extracting two production pillars to induce higher caving in order to enhance the stress shadowing effect. The logic being that the stresses would flow above the caved zone and over the adjacent workings. However, this required that the panel be widened and stress damage could have occurred in the number 8 entry. It is important to remember that the stress shadowing effect is limited in extent. The horizontal stress levels in the immediate roof will return to their "pre-relieved values" at some point beyond the failed ground. Where dealing with high horizontal stresses, one should examine the problem as a three dimensional issue, and realize that there is a risk of stress damage occurring in the outside entries of wider panels. As for Sargent Hollow's experiences with dual production pillar extraction, the unfavorable panel orientation coupled with channel margin roof geology did not provide reasonable mining conditions to evaluate the plan's effectiveness. If these problems

were mitigated, the dual pillar extraction method may warrant further consideration.

One final matter which was debated is how many crosscuts the pillared out workings should lag behind the working faces. Clearly, the faces would be stress relieved sooner if pillaring were conducted one crosscut outby instead of two. The major concern however was, that a pillar line fall may override the breakers into the face area. This actually occurred, without incident, on two occasions at Sargent Hollow. Both of the incidents occurred after 1 Right Panel had been reoriented (figure 8, Area B). On one of these occasions, mine personnel allowed the roof to fall for additional stress relief because the heading was going to be discontinued.

CONCLUSIONS

1. Roof control problems associated with high horizontal stress were significantly reduced after initiating the single production pillar advance and relieve plan at Sargent Hollow Mine. Underground experiences indicated that timely panel advance and systematic pillar extraction were critical to the plan's success. Single development pillar extraction using MRS's proved to be the most effective method of keeping the section on cycle. However, this is not to say that equally effective, or even more effective, results might be achieved utilizing the dual pillar extraction method given the proper circumstances.
2. Prior to instituting an advance and relieve plan, mine personnel should stress map underground workings adjacent to the planned mining area to determine the maximum horizontal stress direction. This information is critical when determining both the panel orientation and the correct pillar plan.
3. Where dealing with high horizontal stresses, one should examine the problem as a three dimensional issue, and consider restricting panel width to minimize the possibility of stress damage occurring in the outside entries. It is important to note that the stress shadowing effect is limited in extent. The horizontal stress levels in the immediate roof will return to their "pre-relieved values" at some point beyond the failed ground. Also, narrower panels can be developed and stress relieved more rapidly which minimizes stress concentrations.
4. The face areas will be stressed relieved more quickly the closer the pillared out workings are to the faces. However, gob falls can override the breakers into the face area. Where conducting optimum directional mining with respect to the maximum horizontal stressfield under favorable geologic condition, additional study and experience is warranted to resolve whether the pillared out workings should lag one or two crosscuts outby of the faces.

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Localized Horizontal Stress and its Effect on Ground Control

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ABSTRACT

Regional high horizontal stress has long been recognized as an important factor affecting roof stability. However, localized high horizontal stress, although causing many ground control problems, has not received due attention. Through many ground control studies and underground experiences, it has been found that localized horizontal stress can be created by abrupt changes of overburden and stream valleys. The magnitude of localized horizontal stress depends on the valley configuration and overburden depth. Utilizing the Jennmar Finite Element Analysis System (JFEA), this paper analyzes the mechanism of the localized horizontal stress concentration based on two case studies.

INTRODUCTION

The effect of regional horizontal stress on roof stability is well known and documented. It is a very important factor in the design of mine layouts and roof support. However, ground control experiences have shown that in some mines, although the regional horizontal stress is not high, roof falls related to horizontal stress often occur in areas where the overburden abruptly changes or is stream valley associated. Many falls are often attributed to weak strata or slickensides related to the stream. But the directional roof cutter and roof fall indicate the existence of localized horizontal stress concentrations. Two of many cases, which Jennmar has studied in ground control experiences, will be presented in this paper. Utilizing the Jennmar Finite Element Analysis System (JFEA), it is found that localized horizontal stress concentration zones can be created under a stream valley area. The stress direction is usually perpendicular to the stream valley. The magnitude of the localized horizontal stress depends on the depth and width of the valley, seam overburden depth, and rock properties.

MINE # 1

Mining And Geological Conditions

Mine # 1 (Fig. 1) is currently mining the Elkhorn #3 seam with a continuous haulage system. The mining height is 6' with a 4"-16" parting in the seam. Each section consists of 6 or 7 entry system. The mine layout is shown in Fig. 1. The immediate roof typically consists of 0'-7' dark shale, with highly laminated sandy shale or sandstone above. The entry width is 18'-20' and the pillar is 70' x 75' center to center. This mine is under a hilly mountain area with overburden depth ranging from 200'-800'. Five major stream valleys lay over the mining area as shown in Fig. 1. Overall, regional horizontal stress is not a major concern for roof stability.

Roof Fall History and Analysis

Since the mine started in 1996, it has successfully controlled the roof with a 5', fully grouted, rebar bolt. However, as the mining progressed into the North and East area, a number of bad roof areas were encountered as shown in Fig. 1. Each area is described as follows:

Area 1: The 1st North Mains and 2nd North Mains had been developed without any major roof problems until mining approached stream Branch 1. As entries were developed closer to the stream, the roof deteriorated very quickly. Roof cutter developed at the left corner of the entry soon after mining and severe floor heave occurred. The roof falls in this area are shown in Fig. 1. Underground observations indicated that the horizontal stress appeared to be very high in this area.

Area 2: The roof in 1st West Mains had been in a very good condition until mining approached stream Branch 3. Roof sag and cutter was so severe under the

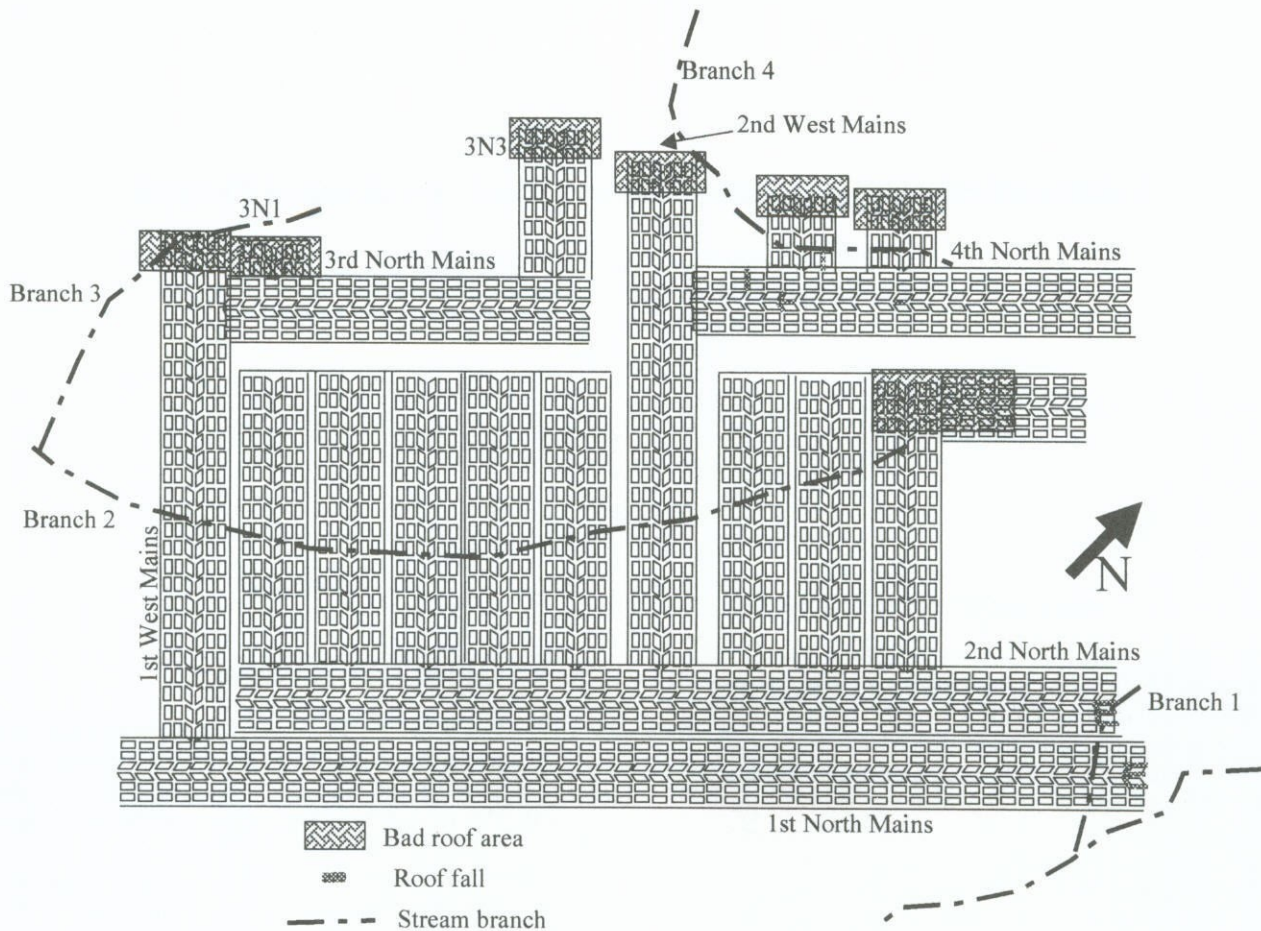


Fig. 1 Mine layout and roof fall history in Mine #1

stream area that the 1st West Mains turned right to develop the 3rd North Mains. After the 3rd North Mains was completed, 3N1 section was attempted. However, the roof condition was almost identical to the 3rd North Mains and the section had to be abandoned. The 3N3 section started successfully, but extreme roof cutter was encountered as it approached stream Branch 4. The section eventually was abandoned.

Area 3: As the 2nd West Mains approached stream Branch 4, the horizontal stress apparently increased in magnitude. Severe roof sag, cutter, and a roof fall occurred. The section then turned right to drive the 4th North Mains. Conditions did not improve very much. A number of roof falls occurred in the 4th North Mains as shown in Fig. 1. The two sections driven from the 4th North Mains encountered the same roof problems and were abandoned.

Area 4: The three bad roof areas described above are all related to the stream valley. However, under stream Branch 2, eight sections have been successfully developed and recovered. Only the last section, which is under the half-basin shaped stream end, encountered roof cutter and

bad conditions. It should be noted that the roof falls in Areas 1, 2, and 3 all occurred under the half basin shaped stream end.

The roof fall experiences have shown that the majority occurred under the influence of a stream valley. It was apparent that a high horizontal stress zone exists in the stream valley area, and especially in the half basin shaped valley end as show in Fig. 2.

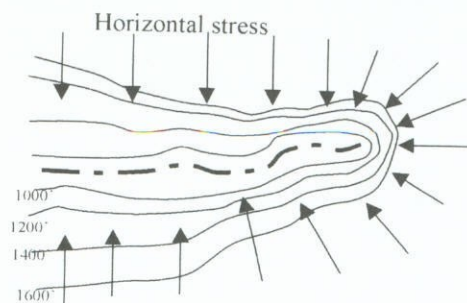


Fig. 2 Stress concentration around valley end area



Examination of core hole log data indicate the appearance of laminated sandstone with shale streaks at the 5' or lower horizon. This type of strata is normally associated with unstable roof. Shown in Fig. 3 is the roof strata column for both good and bad conditions.

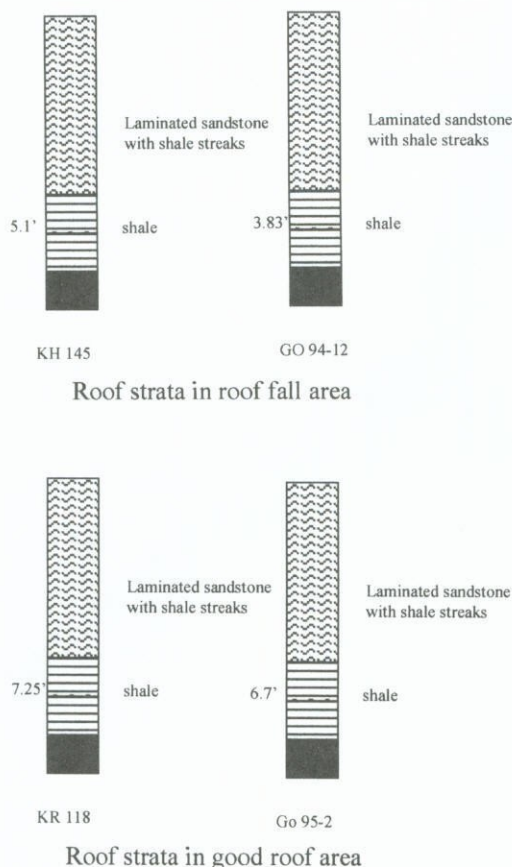


Fig. 3 Roof strata in roof fall and good roof areas

MINE # 2

Mining and Geological Conditions

Mine #2, shown in Fig. 4, is mining the Upper Freeport seam by room and pillar method. The mining height is 5-6' and the overburden depth varies from 150'-400'. The immediate roof (Fig. 5) typically consists of thinly laminated, light gray shale. Underground observations indicate clay content in the immediate roof varies. Several stream valleys lay over the mining area. Generally, the regional horizontal stress is minimal based on underground observations and experience from neighboring mines.

Roof Fall History and Analysis

Roof Fall Vs Topography Changes

Shown in Fig. 5 is the roof fall frequency versus the topography change along A Mains. It clearly shows that all the roof falls occurred under valley areas, most of which are associated with stream bottoms. From Wall 1 to Wall 45, the overburden depth is less than 200' with two major stream valleys. Stream Valley #1 is 50' deep and has two roof falls. Stream Valley #2 is 170' deep and has 9 roof falls. The overburden in all the roof fall areas is less than 200'. For the remaining part of A Mains, the overburden varies from 200'-320' with three major valleys. Valley #3 is 120' deep and has one roof fall. Valley #4 is 120' deep and has two roof falls. Valley #5 is 200' deep with two roof falls.

Shown in Fig. 6 is the roof fall frequency versus the topography changes along B Mains. Two major stream valleys are associated with B Mains. Under the first valley, one roof fall occurred. As the B Mains approached the second stream valley, two roof falls occurred and the roof conditions were so bad that development was forced to stop. The C Mains encountered the same roof problems as it approached the stream valley.

Sections B6, B5, and B4 have been successfully mined without any major roof problems. However, section B7, which is adjacent and parallel to the stream valley, experienced difficult roof conditions. Severe cutter roof and two major falls occurred in the entry. However, the crosscut generally remained in good condition.

Roof Falls Vs Strata Changes

Underground observations indicated the roof strata under the stream valley areas were generally weak with a high clay content. Higher clay content would make the roof more prone to moisture deterioration. Also, numerous slickensided clay veins were present with random orientation.

It can be summarized that localized horizontal stress related to the stream valley and weak roof strata are the primary reasons for the roof falls in this mine.

STUDY OF THE STREAM VALLEY EFFECT

Roof fall analysis in the two cases has demonstrated that the stream valley plays a very important role in roof stability. A finite element model was constructed to analyze the stream valley effect on stress concentrations. The in-situ horizontal stress applied to the model is 600 psi. The results are shown in Figs. 7 and 8.

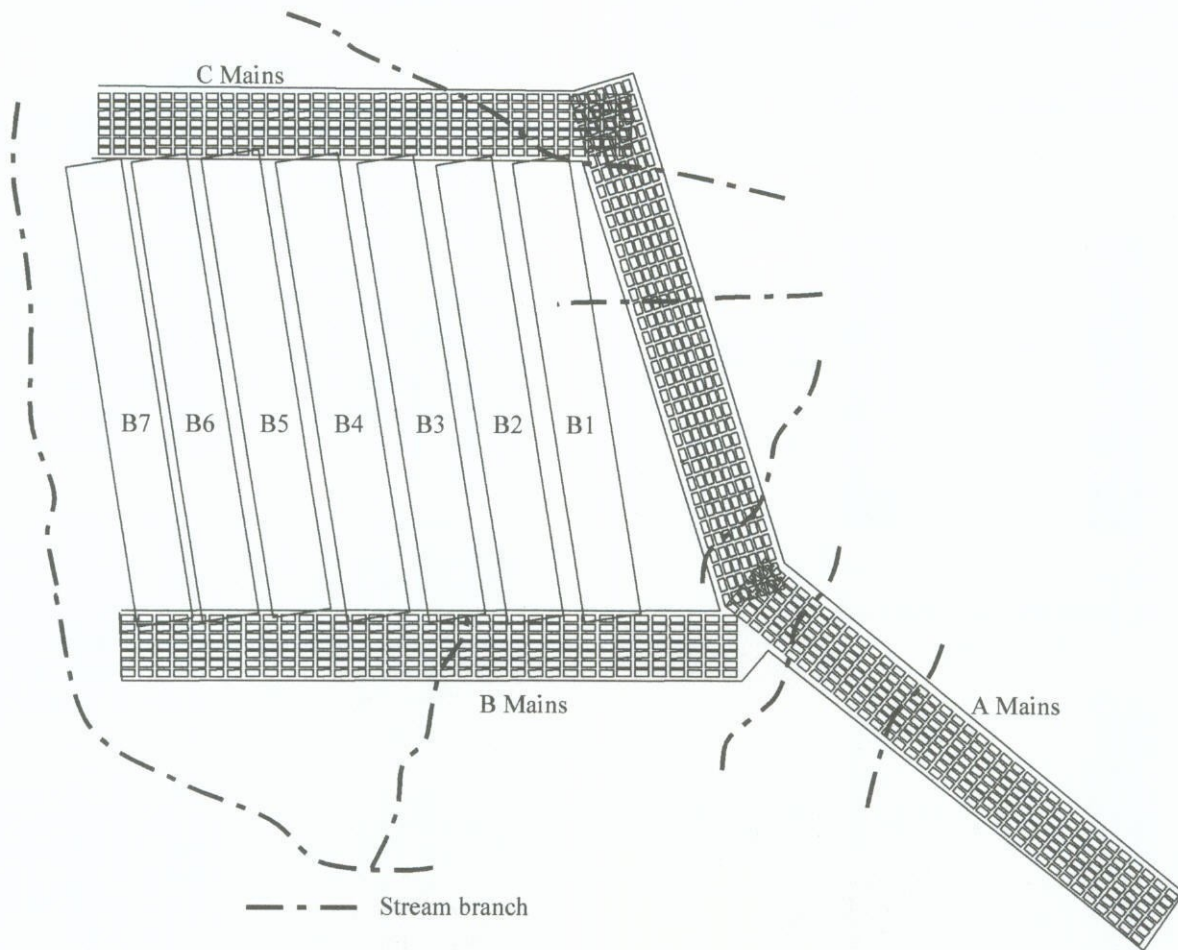


Fig. 4 Mine layout of Mine #2

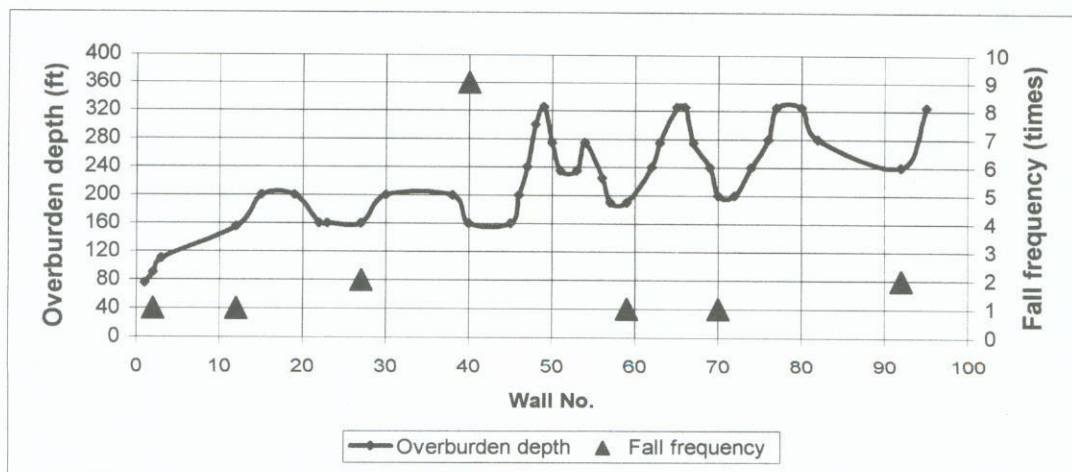


Fig. 5 Roof fall frequency vs. topography change along A Mains

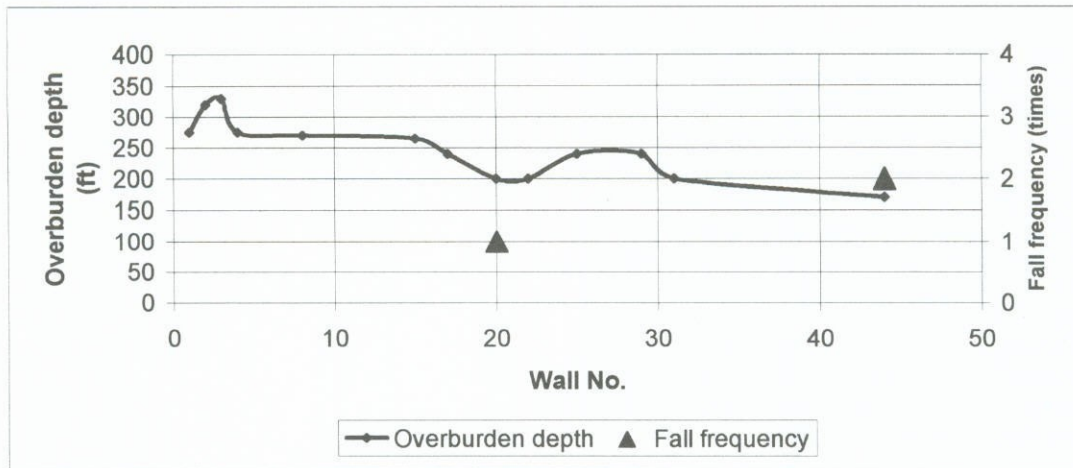


Fig. 6 Roof fall frequency vs. topography change along B Mains

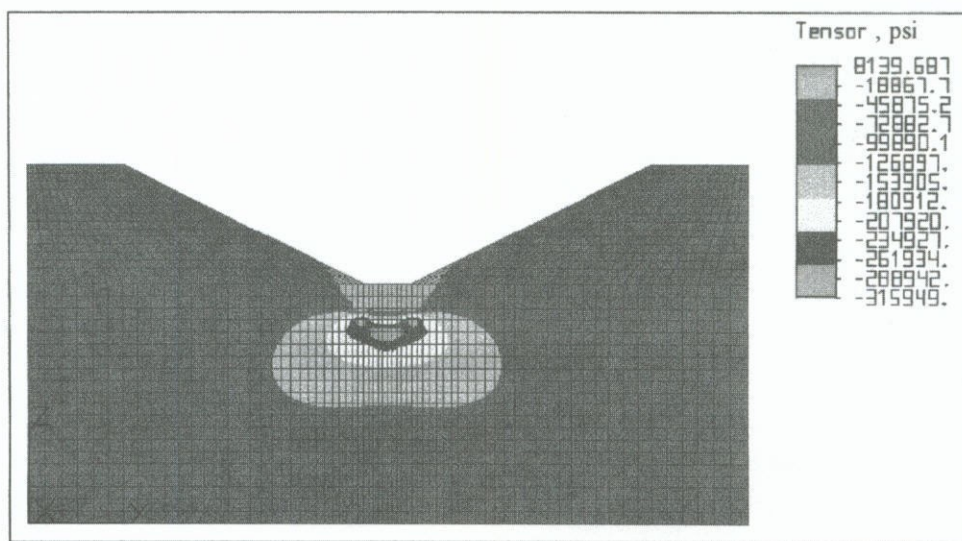


Fig. 7 FEM analysis results – stress concentration under a stream valley

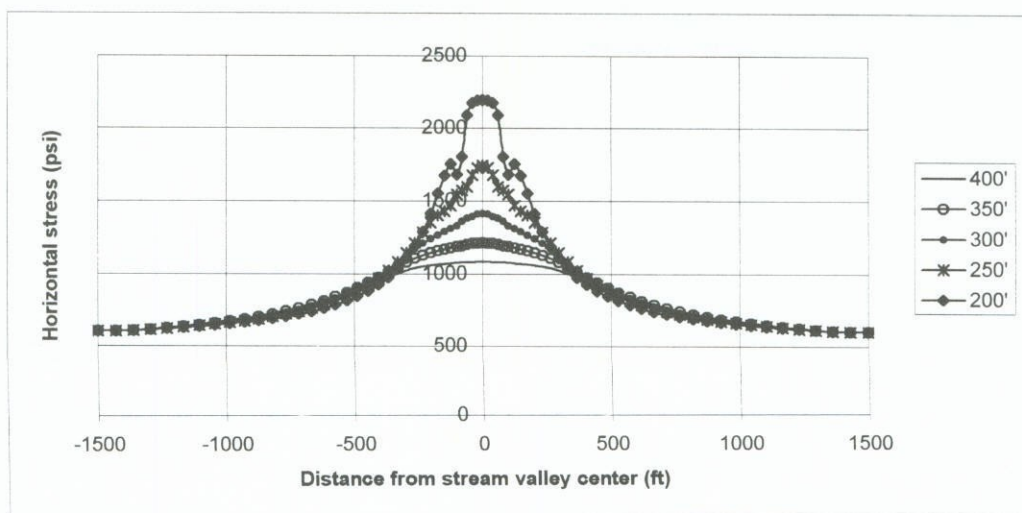


Fig. 8 Horizontal stress profiles under a stream valley



From Fig. 7, it can be seen that the horizontal stress is concentrated on an area under the stream valley. The magnitude of the stress concentration depends on the depth of the valley bottom and coal seam. If the coal seam is above the valley bottom, the horizontal stress is relieved and the roof will be more stable. From Fig. 8, it can be seen the deeper the coal seam is from the valley bottom, the lower the stress concentration. The maximum stress concentrations at different depths are listed in Table 1. The Stress Concentration Factor (SCF) is defined as the localized stress concentration divided by the in-situ stress. It can be concluded that the stream valley creates a localized high horizontal stress concentration zone and roof stability will be adversely affected in this zone.

Table 1: Localized stress concentration caused by a stream valley

coal seam depth from valley bottom(ft)	in-situ stress applied to the model (psi)	localized stress concentration (psi)	stress concentration factor (SCF)
200	600	2200	3.7
250	600	1750	2.9
300	600	1420	2.4
350	600	1220	2.0
400	600	1150	1.9

CONCLUSIONS

The case studies demonstrate that roof stability is adversely affected by a stream valley. The localized high horizontal stress concentration has been observed underground. The roof falls under the stream valley area are caused by the combined effect of weak roof strata and a localized horizontal stress concentration. Finite element analysis indicates that there is a high horizontal stress concentration zone under the stream valley. The direction of localized horizontal stress is parallel to the stream valley and its magnitude depends on the valley depth, valley width, roof strata properties, and the overburden depth of the seam.



Prediction and Control of Surface Subsidence over Abandoned Mined Lands in Carbonia (Italy)

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ABSTRACT

The investigated area lies close to a large town and covers an area of about 1km^2 . The mine was worked at depths ranging from 5 to 100 meters but mining operations actually ceased some 50 years ago. There are seven coal seams, each one to two meters thick, overlain by sedimentary rocks in a strongly faulted area. Different kinds of subsidence have occurred, crown holes, shallow depressions and hummocky ground. The results of rock mechanical characterisation (RMR method) in the area have been used to define three different types of numerical model (FLAC 2D). A large numbers of simulations has been performed to solve three different problems at three different scales: to determine the mechanical properties of the roof rock and overburden involved in the subsidence process, to assess the crown hole subsidence potential and to define the properties of the beam foundations.

INTRODUCTION

The area under investigation forms part of the Sulcis coalfield, located in SW Sardinia.

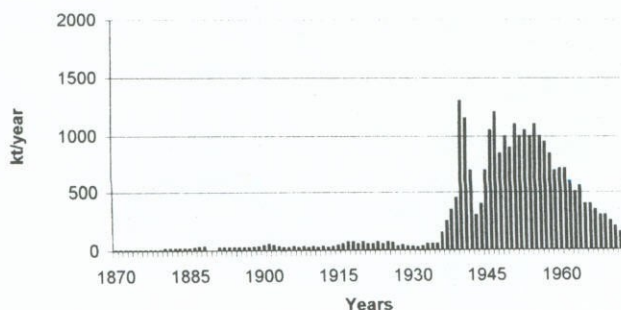


Figure 1. Coal production in the Sulcis basin.

The coalfield is stratified and extends over 35 km^2 , at depths

ranging from 5 down to 300 meters, with coal beds between 50 and 200 cm thick. Over the years, the coal deposits have been exploited by various mining companies. Coal production is shown in Figure 1 (1).

Of Tertiary age and producing coal technologically classifiable as sub-bituminous, the coalfield was discovered gradually and the upper parts began to be developed in the 1870's while mining in the deeper parts has not ceased altogether. The southern part of the coalfield is relatively shallow, becoming deeper northwards. The expansion of the extractive industry was accompanied by major urban development. Coal production and manpower reached a peak in the 1940's (Figure 1) with 1.3 Mt/year. It was during this period (December 1938) that construction of the town of Carbonia began in an area strongly affected by the consequences of near-surface mining, in particular subsidence and widespread mine waste dumps.

The present paper is concerned with the subsidence resulting from mining operations in the upper part of the deposits, and focuses in particular on the approach adopted to investigate the area and to predict further incidents.

The investigation method was strongly conditioned by the scant information at hand. No precise details were available about the properties of the coal strata, their depth and thickness, the mining technique used, the presence and dimensions of pillars, the presence of inclines, drifts and shafts. The possibilities offered by stress strain analysis can solve these problems by simulating different conditions.

GEOLOGY

The coal basin lies between the outcrops of the Paleozoic basement and the Cortoghiana and Serbariu-Sirai faults. The outcrops are composed, north-westwards, of a narrow strip of ignimbrites of Tertiary age and northwards of a minor outcrop of Quaternary eolian sands, while alluvium of various nature blankets the rest of the area (1).

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Underground, rock formations of Tertiary age overlap. From top to bottom we can recognise: the *Cixerri* sandstone, Productive seams and limestone *Miliolitico* Autc formations. The *Cixerri* formation, which outcrops only marginally in the study area, overlies the weak discordance the Paleocene-Eocene deposits of the Productive seams and has variable thickness of up to 60 meters. It is composed of an alternation of sandstone, siltstone and clays, with thick polygenetic conglomeratic lenses, that deposited prevalently in fluvial environments, with rare carbonate lacustrine deposits at the base. The clasts are polygenetic, of varying size and composed chiefly of Mesozoic limestone, metasandstone, metasilites, gneiss, granite.

The Productive seams succession, which does not outcrop in the area concerned, has average thickness of around 70 - 80 m and is composed at the base of miliolithic limestone of littoral environments, followed by marly limestone, lagoonal marls and shale. Interbedded with the palustrine sediments are numerous coal seams of variable thickness, which have been mined in the past. Tectonically, the Productive seams formation has S-W trending tabular geometry, strongly complicated by folds with axis generally inclined NW-SE, dipping S-E. Three major faults exist in the area, produced by tectonic movements during the late Tertiary, striking N-S (that interrupts the mine workings eastwards), NE-SW and E-W respectively.

SUBSIDENCE

Generally, for the coal mines, two specific features control the effects that the mine openings have on the surface: the large area involved and the presence of an overburden of weak and plastic sedimentary rock (2).

Because of the extensive area exploited, the voids created also affect a vast area and their stability cannot rely merely on the natural properties of the rocks. Furthermore, numerous shafts, raises and ramps were cut through the overburden rock to ensure access to various points, ventilation and haulage of the coal and these represent the weak point as far as stability of the overlying rock cover is concerned. The properties of the

sedimentary rock are responsible for the ease with which the collapse of the roof propagates up to the surface.

Two mining methods were used, room and pillar and longwall mining, and the type of backfill and conditions for emplacement varied accordingly. In the case at hand, initially the room and pillar method was used, particularly suitable for shallow and irregular coal beds. The longwall method was instead used for the deeper coal strata and in more recent times on account of the fact that the room and pillar system yielded low recoveries and required more advanced technology. In the room and pillar system a series of permanent supports (pillars) are left behind, so that once the coal has been mined out the rooms are stable. In the event of the roof or pillar collapsing this does not happen during working and in any case is localised. With the longwall system, the coal is removed from the panel and the roof is then allowed to cave in to ensure stability but sometimes the collapse propagates up to ground level. Thus, any subsidence will appear continuously and during mining operations.

Both systems have been used in the study area and consequently two kinds of subsidence have occurred. Large areas with ill-defined contours that formed during mining activity (approximately 50 years ago), and small areas of less than 100 m² with well-defined boundaries that have appeared at different times, some very recently.

The following types of subsidence have been recognised in the area (3): crown holes and shallow depressions. Indeed, numerous circular depressions have been recognised, a few of recent formation. Some of these follow a pattern parallel to the likely position of an adit; others have formed at the boundary of a panel. Various shallow depressions are visible on the ground. The most prominent of these can be observed in the northern part of the area see (Figure 8).

It may have been caused by the subsurface extraction of pillars or by the reactivation of geological faulting. Likely, both types of subsidence occurred during mining.

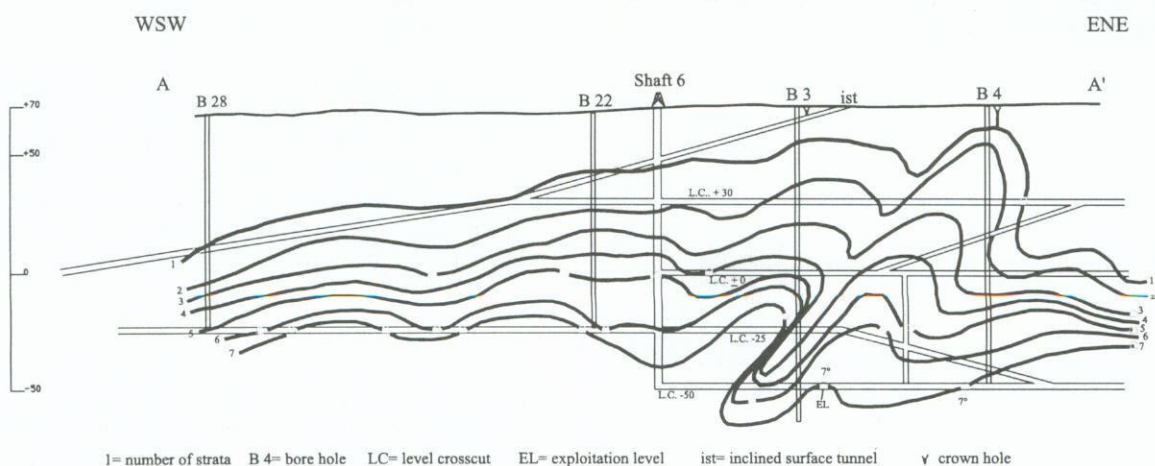


Figure 2. Main section of the coal mine

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INVESTIGATION METHOD

The area has been investigated using classical rock mass methods: collection and analysis of documents, in situ surveys, boreholes and geomechanical characterisation (4). The results obtained have been used to define a geomechanical model of the area and to formulate different numerical models for the three main problems, namely: to determine the mechanical properties of a large area formerly affected by subsidence, to assess the subsidence potential, to determine the characteristics of a suitable foundation system.

The simple formulation of a geomechanical model, even when combined with a geophysical characterisation of the area, is not sufficient for land-use planning purposes and design of buildings. Moreover, the classification systems of subsidence caused by underground coal mines (5), based on morphological aspects, only provides a qualitative description. The specific object of the present work is to show that to assess the possibilities of developing an area and to define the subsidence potential specific stress-strain analysis is required.

The reconstruction of the main section of the mine is shown in Figure 2. The geologic columns obtained from four significant boreholes are shown in Figure 3.

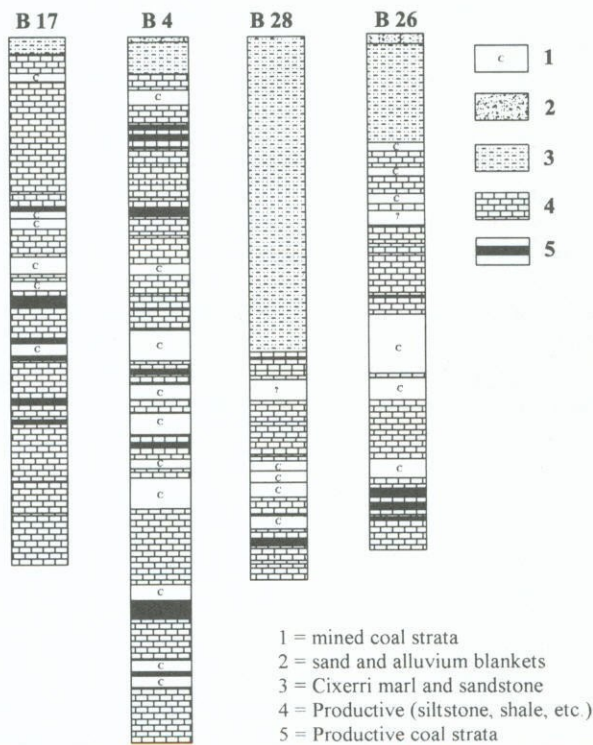


Figure 3. Four typical geological columns.

ROCK MECHANICS CLASSIFICATION

The rocks in the study area have been characterised using the RMR (Rock Mass Rating) index (6), whereby the mechanical

behaviour of a rock mass can be assessed on the basis of measurements and observations conducted on recovered cores of borehole.

Twenty boreholes were done, with cores recovery for a total depth of about 2000 m. The rocks were then classified, first dividing the rock masses into regions that differed significantly as to lithology. Within these classes the parameters requested by the classification were then calculated and the partial (due to the contribution of the single properties) and total RMR indexes were then obtained. The means calculated for each rock type and for the Cixerri and Productive formations to which these belong, are summarised in Table 1.

The properties related to the fracture of the rock (RQD, spacing and surface conditions) and to water circulation were determined continuously along the entire length of each borehole, while rock strength was evaluated by means of local tests (by Uniaxial Compression Tests or by Point Load Tests). The calculated means were assumed to hold for the same rock type also in zones and boreholes where mechanical strength tests had not been conducted.

All the classical properties have been considered in the discontinuity analysis: persistence, opening, roughness, backfill and weathering. Groundwater level has been measured for each borehole drilled to estimate the water circulation system. The following characteristics have been obtained: unconfined aquifer at a depth of 5-8 metres, currently in a steady state condition, sensitive to runoff, with moderate recharge time and surfacing in certain zones. The findings of geomechanical investigations pointed to a clear distinction as to rock type and geomechanical properties between the sedimentary rocks of the overburden (Cixerri) and the thin layers of the Productive formation. The former is more resistant and less prone to creep than the sedimentary rock of the Productive layers formation. They also form beds of greater thickness and furthermore when these fracture their bulking modulus is negligible.

Table 1. Mean RMR for lithological units and formations.

Rock	RMR _{lu}	RMR _f	SD	RSD
Cixerri sedimentary rocks		62	4	6%
- marl	61			
- sandstone	62			
Productive layered rocks		50	8	16%
- conglomeratic sandstone	61			
- silty sandstone	54			
- shale and marl	54			
- limestone and coal	49			
- limestone and marl	49			
- marly limestone	51			
- coal	42			
- marl and sandstone	45			
- marl and shale	44			
- siltstone and coal	48			
- siltstone and shale	53			

SD = Standard Deviation

RSD = Relative Standard Deviation

The results show that the overburden rock of the Cixerri

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formation is of good quality (Bieniawski class II), and exhibits substantially similar geomechanical properties (relative deviation of just 6%), though they differ significantly from the rock of the Productive formation.

The latter on the other hand is composed of thin layered rocks on average of fair quality (Bieniawski class III), but displays a certain variability. Data analysis shows that this variability may be attributed to layers near the coal seams that are thin and weak, and whose contact with adjacent rocks represents discontinuity surfaces subjected to normal and shear displacements.

Application of the RMR, gives the following mechanical properties of the rock formations.

Table 2. Rock mass mechanical properties obtained with RMR

Rock formation	Cohesion,	Internal friction	Elastic modulus
Cixerri	300 kPa	35°	250 – 500 MPa
Productive	200 kPa	25°	200 MPa

NUMERICAL MODELS

A series of investigations has been conducted in the study area and based on the findings a geomechanical model has been developed for simulating the following:

1. subsidence potential referring to a main section of the coal strata;
2. crown hole subsidence potential associated with mine openings not backfilled during mining or with near-surface inclines and shafts;
3. internal forces in a foundation beam subjected to minor subsidence.

Simulations were performed using numerical methods (code Flac 2D) under the hypothesis of a continuum material, subjected to plane strain and following the strength criteria of Mohr Coulomb. In addition, each of the simulations was performed as described below.

Main section of the mine

Of the total number of coal seams, only the first and second are considered in the model (see Figures 2 and 4). Coal mining is simulated removing for each stage blocks of about 20 m, proceeding from left to right and replacing the extracted material with backfill having low bulk and shear modulus (see Back. H. Seam in Table 3) corresponding to a roof caving in or a slightly compacted backfill. The plasticity zones for tensile strength obtained by the calculation sequence are substituted with vertical discontinuity surfaces having low shear stiffness (see discontinuities in Table 3). That portion of the coal-seam with vertical bedding has been mined using the cut and fill method and consequently the coal is replaced with fill material having greater bulk and shear modulus than the simply caving roof. The number of grids element is 250 x 100 the model dimensions 1250 x 50 m². The mechanical properties assumed in the model are shown in Table 3. The tensile strength is considered nil to allow for fracturation of the rocks (tensile cut-

off).

To respect the correct stress path, the following calculation sequences have been considered: pre-mining stress condition (to assess the original stress state, under elastic hypothesis), gradual working and backfill of the coal seams Nos. 1 and 2, mining and backfill of the vertical seams, formation of some vertical fractures through the overlying rock.

Table 3. Main section, mechanical properties of the materials.

Material	EM	PC	TS	C	FA	NS	SS
	MPa		MPa	kPa		Pa/m	Pa/m
Cixerri	500	0.20	0	300	35°	---	---
Productive	200	0.25	0	200	25°	---	---
Coal	100	0.35	Elastic model				
Back. H. Seam	10	0.35	0	10	30°	---	---
Back. V. Seam	25	0.35	0	10	30°	---	---
Discontinuities	---	---	---	---	---	10 ⁸	10 ⁷

EM = Elastic Modulus

TS = Tensile Strength

FA = Friction Angle

SS = Shear Stiffness

PC = Poisson Coefficient

C = Cohesion

NS = Normal Stiffness

Risk analysis of crown – hole subsidence

Four different situations have been examined corresponding to different depths of the coal bed (at 5, 10, 20 & 30 metres) and different overburden thickness and composition. The contact between the different rock types is defined by horizontal discontinuity surfaces having low normal stiffness (consequently the layers may become detached) and average shear stiffness (consequently they may undergo shear displacement). The rock types considered and their properties correspond to the geologic columns (see Figure 3) identified by core logging and boreholes analysis. The number of grid elements is 150 x 200 the model dimensions 60 x 60 m². The properties assumed in the model are given in Table 4.

Table 4. Risk of crown hole subsidence: material properties.

Material	EM	PC	TS	C	FA	NS	SS
	MPa		kPa	Kpa		Pa/m	Pa/m
Cixerri marl	500	0.25	100	100	35°	---	---
Cix. sandstone	250	0.25	50	50	35°	---	---
Roof layer	100	0.30	0	50	35°	---	---
Coal	100	0.35	Elastic model				
Backfill	10	0.35	0	10	30°	---	---
Discontinuities	---	---	---	---	---	10 ⁷	10 ⁸
Surface Soil	25	0.35	0	10	30°	---	---

The following stress paths have being considered: pre-mining stress condition, working of the first coal seams, emplacement of a timber support system and backfill of the mined out coal seam with fractured rock, removal of the support (presumed worn and inefficient), progressive collapse of the thin rock layers, progressive collapse of the overburden.

The evolution of roof collapse and mine opening is simulated by removing the elements under plasticity for a total displacement of more than 10 cm and when equilibrium has not been reached. After each verification stage the calculation sequence is resumed.

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Design of foundation beam

A 32 metre long beam was considered, with three possible cross-sections, stiffness and column spacing, each of which was subjected to three concentrated load conditions (nine cases). For each case four conditions were considered: soil stability or surface mine opening in contact with the foundation in three different positions (in the middle of the last column, on the cantilever, in the centre of the middle column). In all 36 cases were simulated. The foundation soil has average properties, shown in Table 5, corresponding to a weakly compacted, cohesionless sandy soil. The surface openings are 3 m wide and 1.5 m deep, corresponding to those now visible on the ground and to those simulated as described above. The number of grid elements is 200x100, the model dimensions 60x60 m². The properties assumed in the model are given in table 5.

Table 5. Model of foundation beam: material properties.

Material	EM	PC	TS	C	FA	NS	SS
	GPa		MPa	kPa		Pa/m	Pa/m
Concrete Beam	17	0.20	Elastic model				
Soil	0.025	0.35	0	10	30°	---	---

To respect the correct stress path, the following steps have been considered: pre-mining stress condition, driving the beam foundation into the surface soil under three different load conditions, the formation of a void corresponding to a hypothetical depression in three different positions, the calculation sequence to reach the equilibrium condition (unbalanced force less than 100 N) obtained for each model simulated.

RESULTS

The results obtained for each type of simulation are summarised below.

Main section of the mine

The vertical displacements in metres are shown in Figure 4. The following considerations can be made. Several coal seams generate combined effects only when the thickness of the interburden is less than 3 metres (see Figures 2 and 4).

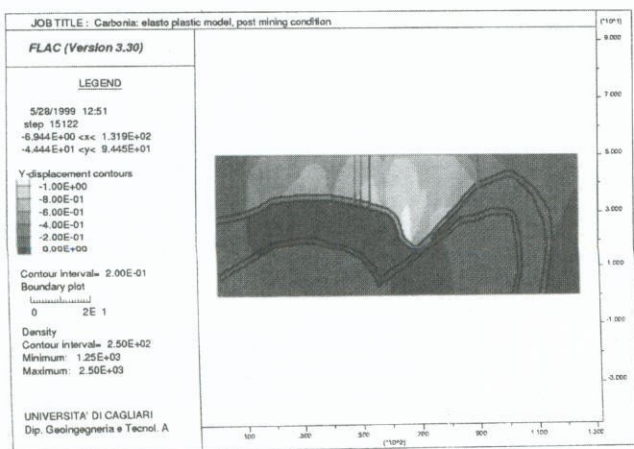


Figure 4. Main section of the mine: subsidence movements.

The area involved in the subsidence (ground level sinking of more than 20 cm) extends over about 800 m, but the dangerous zones for rigid and concentrated failures or angular distortion of more than 1/200 affect less than 300 m.

The simulation of a surface load on this area, in pre- and post-mining conditions, show that the mechanical properties differ significantly in terms of deformation modulus. For a load of 100 kPa load on 1 m² of foundation, a maximum vertical displacement of 4.5 cm was obtained for the post mining case against 3 mm for the pre mining hypothesis.

Risk analysis of crown – hole subsidence

The results obtained for the coal strata at depths of 5 m and 30 m underground are shown in Figures 5, 6, 7 and 8. From the four simulations done, the following considerations can be made. Coal seams less than 10 m deep, as well as zones where shafts or inclines are present, are defined as High Risk areas (HRa and HRb in Figure 8); while those over 30 m deep are considered not to be at risk (blank area in Figure 8).

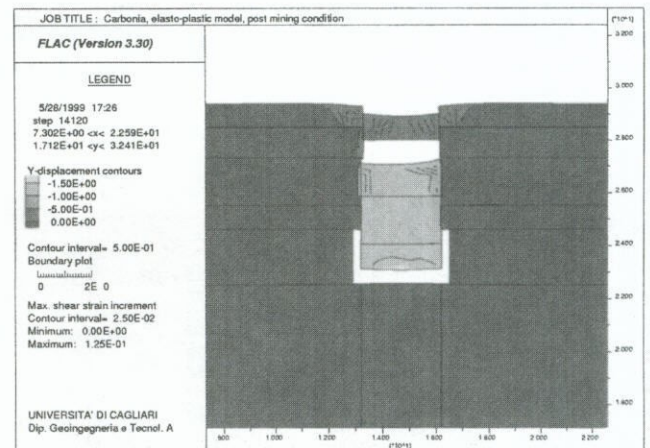


Figure 5. Risk of crown hole subsidence at a depth of 5 m. Vertical displacements and max. shear strain increments during roof collapse.

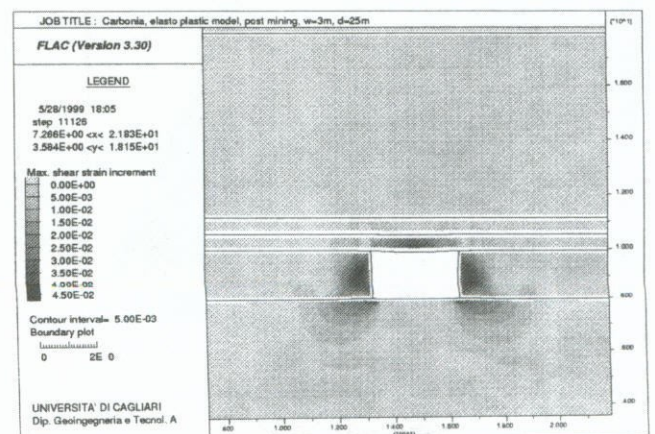


Figure 6. Risk of crown hole subsidence at a depth of 30 m. Max. shear strain increments at the first stage of instability.

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In intermediate conditions ($10 < \text{deep} < 30$) there exists a Moderate Risk (MR in Figure 8) that rock collapse will transmit through the overburden rock to the surface.

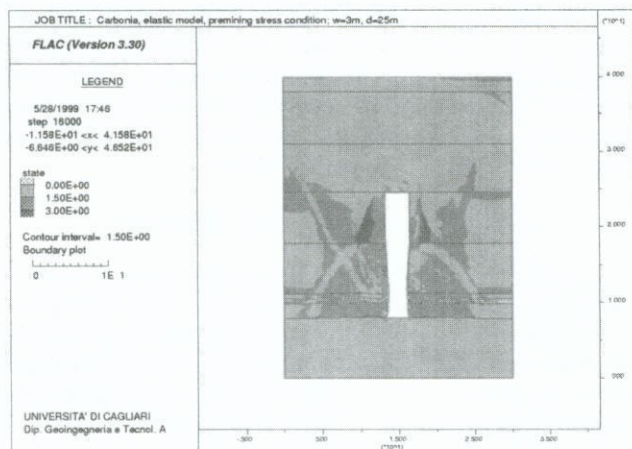


Figure 7. Risk of crown hole subsidence at a depth of 30 m. State of rock plasticity in equilibrium condition: $1.5 > \text{state}$ (no plasticity); $1.5 < \text{state} < 3$ (shear plasticity), $3 < \text{state}$ (plasticity for tensile strength).

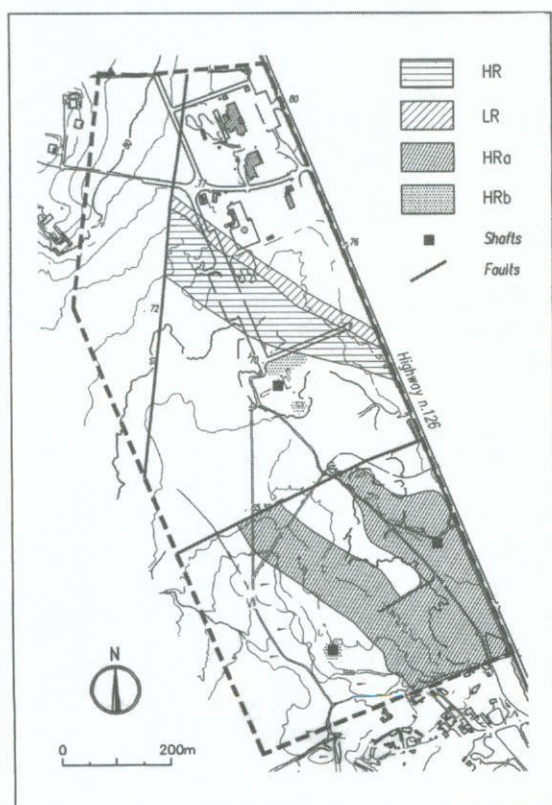


Figure 8. Risk of subsidence. Investigation area and subsidence risk. MR = Moderate Risk, LR = Low Risk, HRa = High Risk for crown hole HRb = High Risk for shafts and ramps.

Design of foundation beam

In all the cases examined the stress condition of the foundation beam is compatible with the strength characteristics of the reinforced concrete. As shown in Figures 9, 10 and 11, the stress condition induced in the underlying ground shows for a 20 m wide portal the possibilities of plasticity extending with consequent risk of instability.

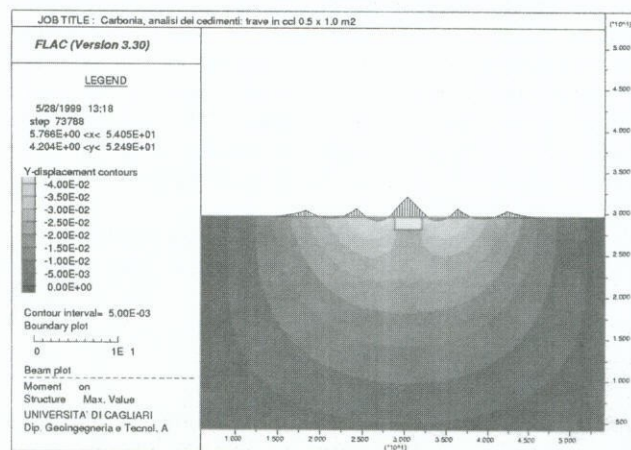


Figure 9. Stress strain analysis of foundation beam. 20 m wide portal and crown hole in the middle column. Vertical displacements produced.

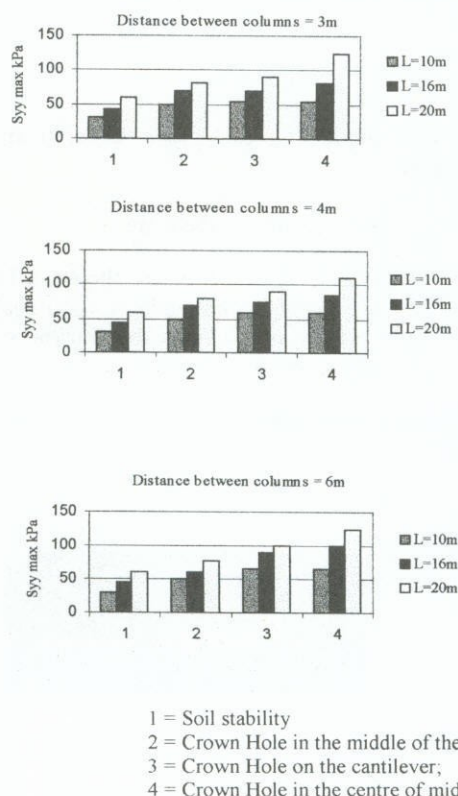


Figure 10. Analysis of foundation beam. Vertical stress component at the surface soil for 3 portal width (L).

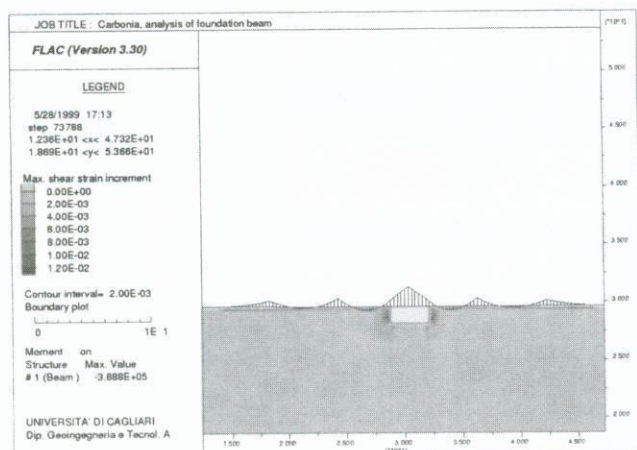


Figure 11. Stress strain analysis of foundation beam. Portal of 20 m wide and crown hole in the central column. Moments in the foundation beam and max. shear strain increment in the ground.

The overload transmitted to the ground due to the effect of load redistribution as a result of the surface caving in, only generates a plastic deformation hazard when the excavation corresponds to the middle column and under loads of 115 kN (3 metres spacing), 150 kN (4 m) and 220 kN (6 m).

The vertical stress component obtained in the 36 simulations is shown in Figure 10. In the light of the previous considerations, the value of $S_{yy} > 100$ kPa must be considered as a condition of plasticity development and, consequently, the hypothesis of a portal of 20 m wide must be ruled out for a beam foundation with 0.018 m^4 of inertia moment.

CONCLUSIONS

Based on the results of the simulations the following conclusions may be drawn.

Regarding the properties of the overburden, altered by subsidence, it has been shown that a major reduction of elastic modulus is produced, likely combined with a decrease in permeability coefficient.

As for the possibility of further crown hole subsidence, recognised by morphological analysis, it is shown that overburden properties can be correlated to the propagation of mine opening instability. In the case discussed above, the first coal seam, at a depth of 30 m, can be considered free of subsidence risk. This result has obviously been obtained neglecting the bulking modulus effect, whereby just a 10% increment in bulk is sufficient to completely fill a 2 m high mine opening at a depth of 20 m.

The interaction between a foundation structure and the subsoil strata can be evaluated using a suitable numerical model. For this particular problem it has been shown that suitable stiffness characteristics (inertia moment and resistant area) can be found to obtain a bridge effect between the two parts of the collapsed crown hole.

Lastly two considerations emerge from the simulations performed here. Generally speaking it can be maintained that the mechanical properties of the soil in an area affected by subsidence in the past, even if it has been developed, will be totally different. Therefore, in subsidence areas, precautions need to be taken as to type of foundation, building design and groundwater level control.

The second type of simulation has demonstrated the importance of rock composition. In the case of crown hole subsidence and for moderate or low subsidence risk (rock cover from 10 to 30 meter thick) an interesting solution may be provided by a beam foundation with appropriate stiffness.

One significant consequence of using numerical simulations is the possibility of extending the analysis to thin bedded rocks in neighbouring areas within the same coalfield.

ACKNOWLEDGEMENTS

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Effects of Mining on Underground Infrastructures in the German Hard Coal Sector

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ABSTRACT

The German hard coal production predominantly derives from underground multiseam mining activities. Beside common surface damages, "inner mine damages" on underground infrastructures (i.e. main roads, cross cuts and shafts) due to mining excavations occur to an increasing extent. As these "inner mine damages" dramatically increased because of the development towards high performance longwall mining technologies, this matter raised to be a severe problem concerning mining economics. To protect durable and long lasting underground infrastructures, precautionary measures such as suitable mine layouts, adapted planning of mining operations and moderate face advances have to be considered. Some examples of "inner mine damages" recorded by means of mine surveying methods are given in this paper. An empirical-mathematical model based on the principles of mining subsidence engineering methods will be introduced. This model allows to minimize "inner mine damages" and helps to support and optimize mining processes.

INTRODUCTION

Due to significant changes in the energy politics of the Federal Republic of Germany during the last years it was more and more necessary for the German hard coal industry to reduce the costs in the mines on the one side and on the other side to improve the efficiency of the individual working places clearly. Important prerequisites for that purpose are a very high state of the technical equipment and an optimum utilization of the existing infrastructure. In the end, that development lead to a smaller number of pits with at the same time relatively high output rates coming from only fewer longwalls. From the point of view of mining subsidence engineering, parameters as „face advance“ and „Rate of stress to strata due to mining“ became more and more important. Numerous examinations on selected

objects on the surface already showed that the new mining situation also lead to new knowledge concerning surface mining damages.

During the last years, also here the meaning of dynamic parameters (rate of face advance, stagnations and sequence) has been recognized and much more taken into account. For the influences on the mining operations themselves the situation is a little bit different: it is true that measurements are available for in-mine purposes, but until now they have not been examined very intensively and published under dynamic points of views as it is the case for examinations of the surface. With a higher prognosis safety and therefore an optimized dimensioning of the mining works negative influences to the sensitive infrastructure of a pit (maintenance and stability of the shafts, the rock drifts and headings, maintenance of the ventilation system and logistics etc.) could be better sized up. The costs would perceptibly be reduced if an under- or over-dimensioning of the supports was avoided. Here, intensive scientific research is necessary, since just the infrastructural bottlenecks and insufficiencies have a negative influence on the economic success.

For this purpose the momentary state of the convergence calculation, the extraction layout and the operational monitoring surveys derived from it is described here, taking especially into consideration the dynamic influence factors of modern underground mining technologies.

STATE OF THE CONVERGENCE CALCULATION

Planning of rock drifts

The convergence K describes the lost of height of any road cross section whatever in the center of the road which is composed of the relative roof sinking and the relative floor lifting ($SH = 0.5$ to $0.85 K$) (see Fig. 1,). These deformations and the resulting changes of the roadway

cross sections are chiefly caused by redistributions of the rock stress due to underground mining activities. To be able to estimate these influences already in the planning phase of the roads in question, in 1986 the „Decision Aids for Planning Rockdrifts ([1]) and Headings with regard to ([2]) Rock-Mechanical and Support-Technological Aspects“ were published. These recommendations for mining operations refer to the control and layout of the roads, to the pre-calculation of the deformations of the cross-sections and supports and they give advices for a technical influence. They are chiefly based on numerous calculations and measurements in the region of Western Germany's hard coal industry. Here are therefore now presented in short the

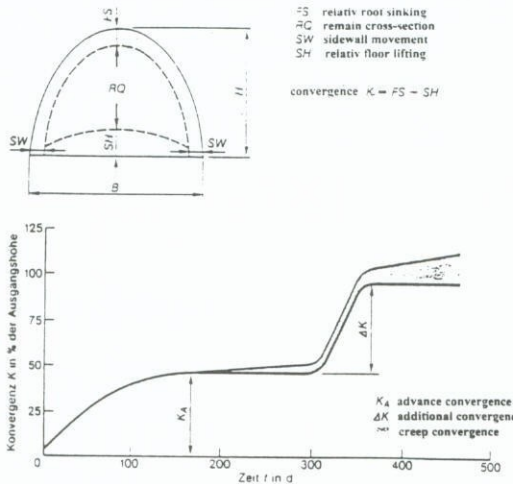


Fig. 1: Convergence of rock drifts

state of the convergence calculation and the resulting decision criteria for laying out the mining operations.

The overall convergence of a rock drifts can be split up as follows as per Figure 1:

1. Advance convergence K_A stopping approximately 150 days after the drifting.
2. A convergence caused by an increase of pressure ΔK produced by mining operations.
3. Convergence K_S due to subsidences because of underlying workings.
4. Creep convergence K_{Kr} due to long-lasting rock movements.

In the literature the following cases are distinguished as important planning aids ([1]):

1. A road should be driven as far as possible in untouched rock.
2. A road should be driven as far as possible with a change of pressure caused by the ribsides.
3. An already driven road is exposed to additional pressure, for example thru overlying workings.
4. An already driven road is being subject to underlying workings and therefore lowered.

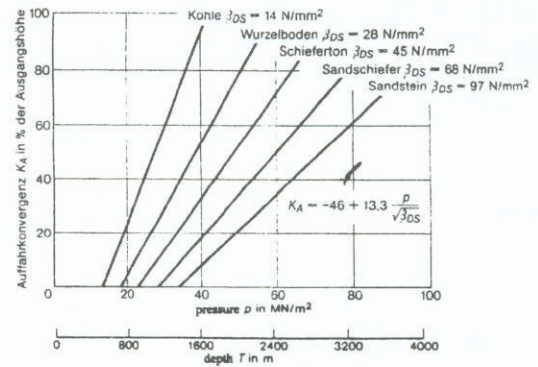


Fig. 2: Advance convergence K_A of rock drifts as a function of rock pressure p , depth T and hardness of the floor seams β_{DS} ([1])

For planning purposes the expected advance convergence K_A from Figure 2 can be estimated without any further calculation.

If, in addition to this, the convergence after an increase of pressure resulting from overlying or underlying workings is to be determined, also pressure change ΔP or the maximum mining-caused pressure p_{max} have to be taken into account. Important parameters are the structure of the bottom seams which is described by bottom indicator GL or the single-axe pressure resistance β_{ds} derived from it. Furthermore, the depth and the rock pressure resulting from it, the expected subsidence of the soil and the presumable deformation of the road cross section are important factors to pre-calculate the convergence.

Planning of headings

For the planning of mining roads the following planning system can be set up ([2]):

1. Evaluation of the rock structure, especially of the bottom with the help of profiles.
2. Determination of the road routing with especial regard to the rock pressure as a function of depth, extraction works and ribsides
3. Determination of the road exploitation and routing
4. Determination of the type of the roadside packs
5. Calculation of the sidewall movement, the floor lift, and of the subsidence volume
6. Layout of the supports and wallings depending on the calculated convergence
7. Planning of the change-over coal face / mining road and its supports, calculation of the convergence
8. Monitoring of the road to increase prognosis safety

Similar as for the calculation of the convergence of rock drifts many factors have to be taken into consideration when calculating extraction roads. These are chiefly based on geometrical concerns. A very important detail for every other road is the derivation of the convergence from the

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other road is the derivation of the convergence from the basic case of an advance road with one-side extraction where the final convergence is reached approx. 300 m behind the face (Fig. 3).

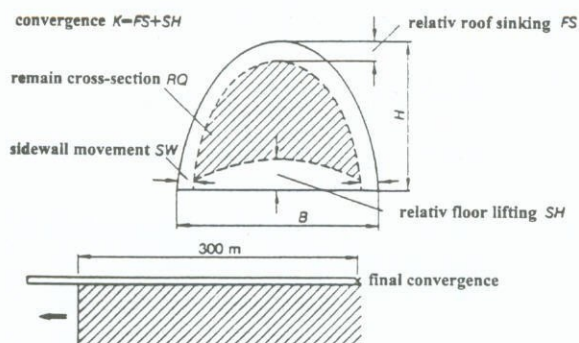


Fig. 3: Convergence of headings ([2])

Unlike that basic case, different variants are possible each one leading to another convergence behavior. This is true for both, the extraction geometry (one-side or two-side extraction with or without coal pillar) as well as for the routing of the roadway (retreat working or advance working with mining in the roadway either in advance, simultaneously or afterwards). The expected convergences are estimated or pre-calculated using the sketched planning aids. Therefore, the dimensioning of the roadway supports is also based on the prediction of the convergence. It has to be recognized, that here only the geometrical conditions, the depth pressure and the mining-induced additional pressure which is a function of the respective state of planning are taken into account. Dynamic parameters like for example the speed of advance are not taken into consideration. This is why we need very much more research and engineering work here.

Dimensioning of the roads supports

The necessary road supports are laid out in accordance with the results of the convergence pre-calculation, shortly described in the present chapter. Since the deformation of the common arch-type supports increases with a growing convergence the supports are dimensioned in accordance with the pre-calculated convergence.

For the usual slide arch-type supports the necessary amount of steel SA can be worked out as follows as a function of convergence K ([1]):

$$SA = 22 e^{0,008 K} \quad (1)$$

When the required amount of steel SA is figured out, the support unit's distance BA and the profile weight PG are

determined as a function of road cross-section Q as follows ([1]):

$$BA = (PG / SA) * ((6 + 0,36 Q) / Q) \quad (2)$$

and

$$PG = (BA * SA * Q) / (6 + 0,36 Q) \quad (3)$$

For a rigid arch-type support an amount of steel SA of 25 to 30 kg/m³ is recommended. Operational examinations showed that more steel did not lead to a significant improvement of the road conditions. If a rock drift is laid out with an amount of steel calculated in this way, arch-type support can be used up to the limits as presented in the following figure (Fig 4) without the medium deformation indicator exceeding value "2" (little deformation).

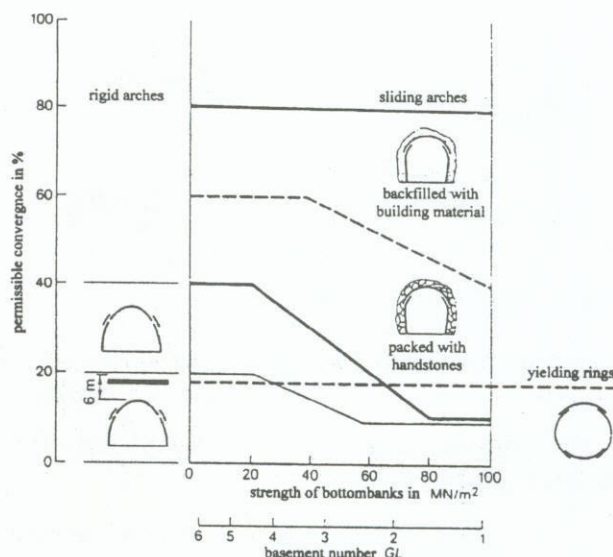


Fig. 4: Capacity limits of the road supports

If extreme convergences occur and therefore deformations are likely, the following reinforcements of the supports can avoid extensive damage in certain cases:

- Floor connection
- Center props
- Floor traverse bracings
- Floor bolts
- Guniting

MONITORING SURVEYINGS

Since the beginning of the eighties many scientists at home and abroad dealt with the pre-calculation of convergences in underground coal mines. Especially in Germany

there are numerous publications on this subject whereas the international trade press does not deal very much with it. The reason for that is on the one hand that other countries use other methods of extraction. In 1975 for example, only 4% of the production in the United States of America came out of longwall face mining, whereas until 1996 this share already climbed up to nearly the half of the underground coal production ([5]). On the other hand at least until now much more emphasis is put on the technical feasibility (machines) or manageability (ventilation) when the deposits structures and mining conditions are similar. International places of discovery for calibrating possible models for convergence calculation by means of the respective measurements are not known to the authors. However, such models will be more important in the future. In his contribution on high-performance longwall-face winning for the international colloquium in Aachen WALKER ([12]) indicates that also in the USA coal mines will have to cope with depths like in Europe in future. From that he deduces that these coal mines too will face the commonly known difficult circumstances. For this reason the following reflections are focussed to certain examinations which took place in Germany until now and were published in the relevant literature. What is precisely presented here, is the knowledge on the influence of the extraction dynamics. These examinations of main roads and pits from different coal mines were concerned with the influence of rock pressure, face stagnations and advance speed on the convergence in long-lived mining excavations.

Examination of cross cuts

The most important intention of an examination of three main roads in different shaft plants was to develop a procedure for determining the creep convergence for the planning case which could be easily controlled ([6]). In contrary to the usual characteristics procedure used until then, the new procedure should also take into account influences coming from face stagnations and different advance speeds. As an example, figure 5 presents the ground plan situation and the pressure distribution over the monitoring period for one of the examined roads.

As for the minimization of the creep convergence in a main road the most important conclusions for mining operations can be summarized up as follows according to the study ([6]):

- „The rate of advance should be as high as possible!“
- „Face standstills should be avoided!“
- „The mining activities should be layed out in a way that the main roads are located in areas with low rock pressure!“

However, these results can only partly be realized for mine workings. Especially the request for a very high face advance is in contradiction with reflections on how to protect the surface ([8]). It has to be presupposed that the concerning examination only served to look at the creep conver-

gence and it is true that the results then made sense. But a general view of the situation where also other factors are taken into account (protection of the surface, etc.) shows that it is necessary to check again the problem of the advance and to revise it.

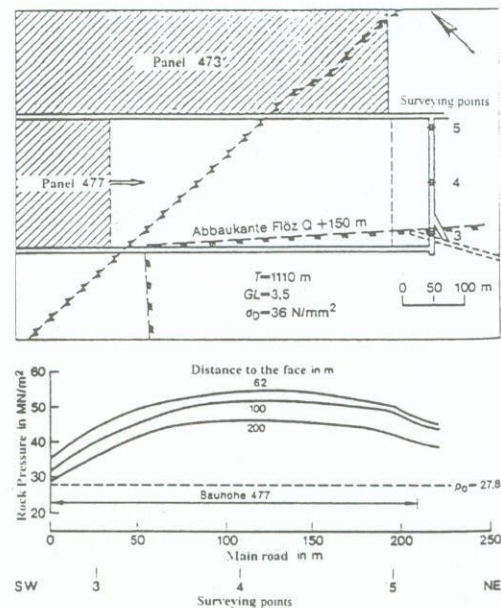


Fig. 5: Pressure distribution in the main road with approach of the face

HANSEL ([7]) reports in a publication about the influence of backfill on inner mining damages and he examines in that context also the parameters of face advance speed and rate of stress to strata due to mining. The result is shown in table 1.

Type of stowing	Speed of face advance	Rate of stress due to mining	Inner mining damages
Pneumatic packing	low	Low	little
Caving	low	Low	average
Pneumatic packing	low	High	average
Pneumatic packing	high	Low	average
Caving	high	Low	severe
Caving	low	High	severe
Pneumatic packing	high	High	severe
Caving	high	High	very severe

Table 1: Parameter for inner mining damages ([7])

Another important result of his examinations was the reduction of the face advance while undermining a cross cut from approx. 5 m/d to 3.2 m/d within a specified range

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in order to minimize the expected inner mining damages. Figure 6 presents the situation in the horizontal plan.

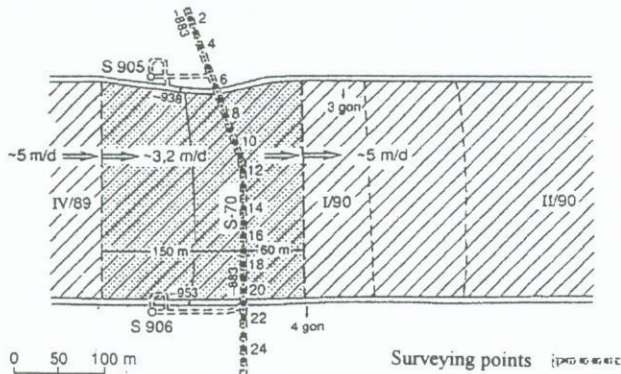


Fig. 6: Undermining of a cross cut, reduction of the face advance. ([7])

In a subsequent look at the subsidence he comes to the conclusion that a forward dynamic strata movement as a result of a high extraction speed could be avoided since there were no irregular subsidences. Also additional surveys of the cross section did not show particular strange results. If that partial examination had shown some, the face of advance would have been reduced further.

For that reason, when cross cuts are over- or undermined, it is required that all mining operations must be executed with a maximum advance of 3.2 m/d in order to keep the expenditure for the reconstruction of the affected cross cuts as low as possible. What is remarkable here is that the determination of 3.2 m/d in the examined example was a quite appropriate decision to minimize inner mining damages. But for an optimum utilization of the working places it must be examined if exactly that advance is indeed the best and if in certain cases also other (perhaps higher) advances would be suitable without much more damages and costs for reconstruction.

GOERKE-MALLET ([4]) examined the consequences of a similar mining configuration in another mine with a very specific geological background. As the most important conclusion he discovers a dependence of the convergence from the dynamic strata movements caused by extraction: „The advance speed is directly proportional to the reduction of the cross section in the undermined excavation: the slower a longwall face is being blocked up the smaller are the convergences.“

Examination of mine shafts

In the hard coal production industry mine shafts are the fundamental link between the underground mining activi-

ties and the processing of the raw materials at the surface. Therefore their protection is an imperative target of mining activities. On the other side few costs arise for the extraction of these reserves (preparatory work, transport of staff and material, mine ventilation) so that these parts of the reserve close to the mine shafts are more and more the subject of current extraction plannings. A secured subsidence prognosis of the effects of the mining activities to the shaft is a basic prerequisite for an inclusion of shaft-close reserves in order to be able to make out already in the layout planning phase which shaft sections in which state of the mining process are subject to vertical stress of a certain size. PREUSSE ([9]) reports on examinations during un-

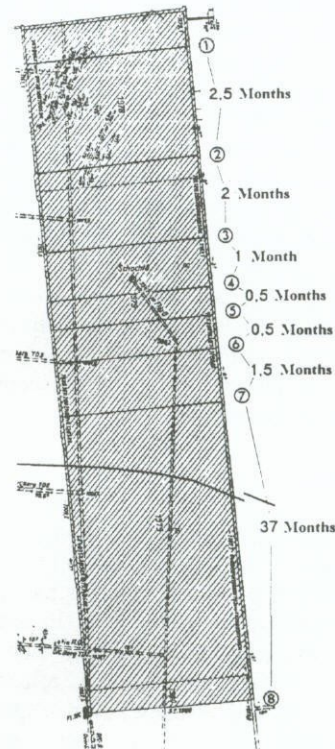


Fig. 7: Undermining of the shaft, states of mining at the time of shaft measurement ([10])

determining of a deep surface shaft in the Ruhr coalfield in a vertical distance of 180 m. Figure 7 presents the situation in the horizontal plan.

Based on an analysis of the available measuring data, model parameters have been derived which helped to determine the vertical stress load along the shaft pillar. Figure 8 shows the vertical arrangement of the measuring points in the shaft. Important results are presented in Figure 9.

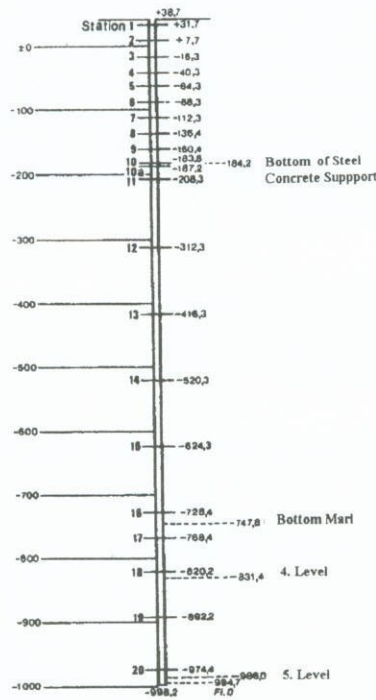


Fig. 8: Arrangement of the measuring points in the shaft

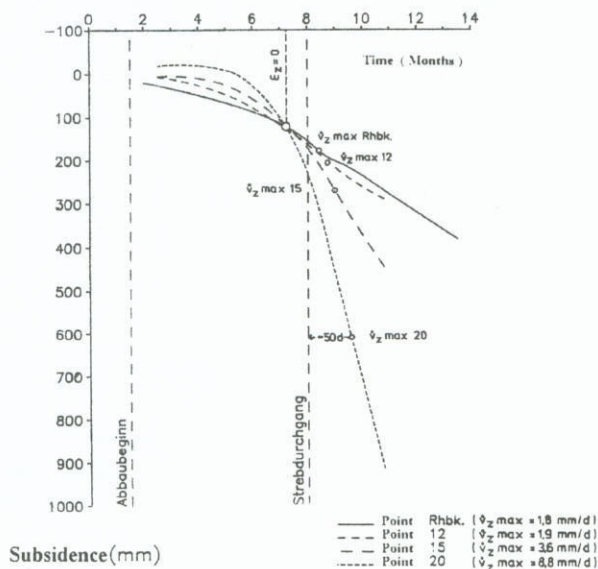


Fig. 9: Measured shaft point subsidences $v_z = s$ and maximum subsidence rate $v_{z,max} = s^*_{max}([9])$

It was recognized by means of the present example that there is a clear dependence of the depth from the shaft point subsidence, subsidence speed and subsidence duration in a sense that with a growing distance from the underlying workings the defined shaft points feature smaller subsidence dimensions and subsidence speeds; in parallel, the

duration of subsidence increases. Another important fact of that examination is the proved transition of the shaft pillar from an initial state of compression into a subsequent state of extension (Fig. 9). This makes the importance of a dynamic pre-calculation model for a correct description of the rock movement process very clear. This model is presented in the following chapter and its suitability is demonstrated exemplarily by a precalculation of the deformations of a rock drifts.

Dynamic pre-calculation method

As already notified, it can be concluded from the relevant literature that an face advance adapted to the local conditions is a necessary and very effective measure to keep convergences as small as possible and to maintain the functionality of the undermined roads in order to reduce mining damages. The purpose of the adaptation or reduction of the advances is to prolongate in time the mining-induced stress. Practical experiences showed that such a prolongation significantly reduces inner mining damages.

As per SROKA ([10-11]) the inner mining damages are dependent on the timing of the mining operations. That does not only concern the speed of face advance but also its changes and stagnations (mining interruptions). A homogenization of the effects of subsidence by means of a continuous extraction process (uninterrupted operation at a constant speed) has the same great influence on inner mining damages as the adaptation of the face advance at all. These parameters, e.g. the speed of face advance and the duration of an advance stagnation are elements of the so-called dynamic strata movement caused by extraction. The analysis of the in-situ measurements showed that the dimensions of the inner mining damages are depending to a large extent on the following ground movement factors:

- Subsidence rate: s^* [mm/day]
- Compression rate: ϵ^* [mm/m/day]
- Compression: ϵ [mm/m]

For a depth range from 800 to 1000 m of the Deutsche Steinkohle AG (DSK, German Hard Coal Co.) the derived critical values are as follows corresponding to SROKA:

$$\begin{aligned} s^*_{Gr} &= 25 \text{ mm/day} \\ \epsilon^*_{Gr} &= 0.25 \text{ mm/m/day} \\ \epsilon_{Gr} &= 5.0 \text{ mm/m} \end{aligned}$$

In general, bigger influences cause larger convergences for the underlying works and endanger, at least for a certain period of time, a further utilization of the roadways.

For a determination of the characteristic of the mining operations timing, an algorithm was drawn up on the base of the DSK pre-calculation method which allows to work out the characteristic of the speed of advance and the duration of the allowed stagnation. This is presented here using

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a practical example on the basis of the allowed subsidence rate.

The data needed for the calculation are as follows:

Mining data

- Average mining depth: $H = 960$ m
- Average total thickness: $M = 1.08$ m
- Face length: $d = 250$ m
- Face advance (planning): $v_{pl} = 10.5$ m/d

Object data (3rd level)

- Average undermining distance: $h = 103.5$ m
- Permitted subsidence rate: $s_{Gr}^* = 25$ mm/d
- Safety factor: $SF = 1.5$

Characteristics of the RAG method

- Subsidence factor (self-filling): $a = 0.9$
- Limit angle (surface): $\gamma = 55$ gon
- Bounding plane characteristic: $n = 0.5$

The characteristic of the speed of face advance is calculated according to the following formulas:

$$v_{max}(h) = \frac{1}{SF} \sqrt{\frac{\pi}{k}} \frac{\dot{s}_{Gr}}{s_{max}(h)} \cdot R(h) \quad (4)$$

where:

$$h = \ln 0.01 = 4.6052, \text{ and furthermore:}$$

$$R(h) = R \cdot \left(\frac{h}{H}\right)^n, \quad R = H \cdot \cot \gamma$$

$$s_{max}(h) = aM[1 - \exp(-0.9 \cdot D(h)(D(h) + 1.3))]$$

$$D(h) = \frac{d}{R(h)}$$

$$v(x, h) = v_{max}(h) \cdot \exp\left(k \frac{x^2}{R^2(h)}\right) \quad (5)$$

The worked out characteristic is presented graphically in Figure 10. It can be recognized that with the existing conditions a reduction of the advance speed has to begin approx. 115 m before the axis of the 3rd level. The zone with a reduced advance speed then extends between the extraction points +115 m and -115 m. A possible distribution of the

advance speed within this zone is shown as a stepped curve (v_{sol} for $SF=1.5$) in Figure 10. The real extraction characteristic lies between the calculation results for $SF = 1$ and $SF = 1.5$ and did not lead to any appreciable troubles during mining in the 3rd level. In the zone with the reduced advance speed there were 6 days of mining work, e.g. the extraction stagnations were 1.5 days max. They are calculated with the following formula:

$$\Delta t_{max}(h) = 0,44 \sqrt{\frac{\Delta s_{Gr}}{s_{max}(h)}} \cdot \frac{R(h)}{v_{max}(h)} \quad (6)$$

where Δs_{Gr} is the permitted homogenization value of the subsidence curve.

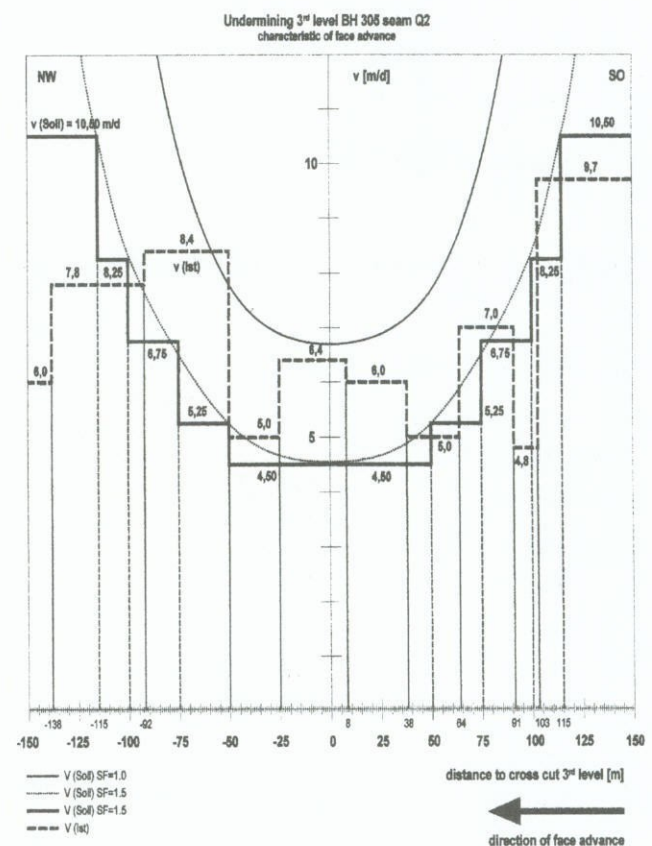


Fig.10: Undermining of a cross cut (3rd level), situation in the horizontal plan

CONCLUSIONS AND RECOMMENDATIONS

As explained in the present lecture, the convergence calculation for roads in underground mining companies in the Federal Republic of Germany is still based on the supposition of a static basic case because of the pressure of the



overlying strata in conjunction with a pressure increase due to mining activities. Furthermore, there are many monitoring surveys on selected objects of the individual mining companies which had the intention to verify the quality and expressiveness of the convergence prognosis. In addition to that there are first promising approaches taking also the dynamic parameters into account with especially looking at the influence of the face advance. The sometimes contradictory statements (see [3] and [4]) occurring in that context must however urgently be examined.

To achieve a higher stability of long-lived mining plants it would be advisable to examine the measurement documentation from the different mines taking especially into consideration the dynamic aspects of the parameters influencing the rock movement (e.g. face advance) and thus causing inner mining damages. With a correspondingly higher prognosis safety the support and wallings of the roads could be better adapted to the requirements and at the same time negative effects for the underground openings could be better mastered and unnecessary costs resulting from an over- or under-dimensioning of the supports be avoided. Finally that would lead to a better use of the capacities of the individual pit and increase its economic result for a long time. Another attempt of optimization includes the quality control in the coal mines. Here we have the possibility, based on the idea of a real quality management, to separate the technical reasons for occurring convergences (work control on site) from extraction-induced differences from the pre-calculation (face advance, mining layout and planning). For these reasons it is not only of scientific interest but also a question of economics to continue looking at the dynamic parameters which influence the convergence in underground plants and to include already existing as well as future measurements into these examinations.

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Ground Deformation in the Case of Underground Mining of Thick and Dip Coal Seams in the Jiu Valley Basin

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ABSTRACT

The phenomena developed on the influence of thick coal seam no.3 mining, by horizontal slices and rocks caving roof control, in the Jiu Valley coal basin, are very complex. By analyzing the influence volume of underground faces and defining the stress and strain zones, around the faces, it is possible to evaluate the suitable powered support performances. Also, Jiu Valley thick coal seams involve about 50% of reserve balance in the main safety pillars of social, industrial and natural objectives, situated on the mining surface perimeters. For a correct design and drawing of main safety pillars complex research was done with the aim of establishing the subsidence parameters. Upper face rocks medium could be assimilated as a granular medium, which is in a permanent movement. For describing the rocks movement, respectively the ground surface displacement, was taken into account the stochastic mechanics principles. The theoretical solutions are in accordance within situ measurements.

GENERALITIES

The Jiu Valley is one of the main basins in Romania due to the reserve balance value, which is estimated to be more than two thousand million tons, and the special coking qualities of the bituminous coal in the west of the basin.

This deposit contains about 21 coal seams, having a variable distribution within the basin; of these 12 are workable. The most important seams are 3 and 5, holding 60% of total coal reserve, seam 3 having the greater share. In these conditions, seam 3 ensures 90% of the output yielded by the entire basin.

Seam 3 thickness, on the normal, is variable. In certain cases where the seam dip is high, thickness measured on the horizontal can be up to 100 m. Seam dip is also variable, from horizontal to very steep dip.

The complicated deposit tectonics, determines delimiting geological blocks of reduced extent (most of them varying between 200 m and 300 m) and an equally complicated configuration, which gives rise to special technical difficulties in winning. Moreover, there occur methane gas emission (of over 10 to 15 methane m³/ coal tone) and there is a marked tendency of coal self- ignition.

Working level depth generally varies between 300 and 500 m, sometimes reaching 1000m.

The most used underground mining method of coal seam no.3 is by horizontal slices and roof control by caving (fig.1).The mining technologies involve coal cutting by blasting or by means of shearers and roof support with powered support or articulated caps and hydraulic props.

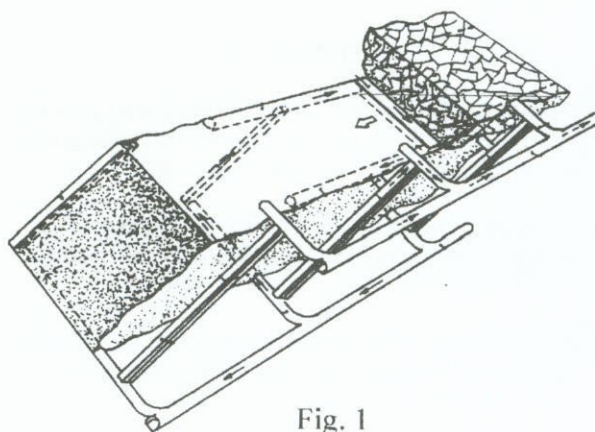


Fig. 1

As the deposit genesis is sedimentary, the most frequent rocks in the basin are: limestones, marls, argillaceous or marly sandstones, conglomerates, etc., their strength ranging between 15-60 MPa up 50-60 MPa, sometimes even more. Mainly, they are rocks of relatively low stability.

STRAIN AND STRESS STATE DEVELOPED AROUND THE MINING FACES

Pressure Acting on the Faces Supports

The face zone, shown in the fig.2, is PA zone. As a consequence of underground measurements we conclude that average pressure acting on the faces supports range within very large limits: from 0.05 MPa, for the case of face no.74, block VII, Lonea mine, to 0.25 MPa, in the case of face no.2 East, block IV, Lupeni mine conditions[3].

After a data analysis, by multiple regression method,

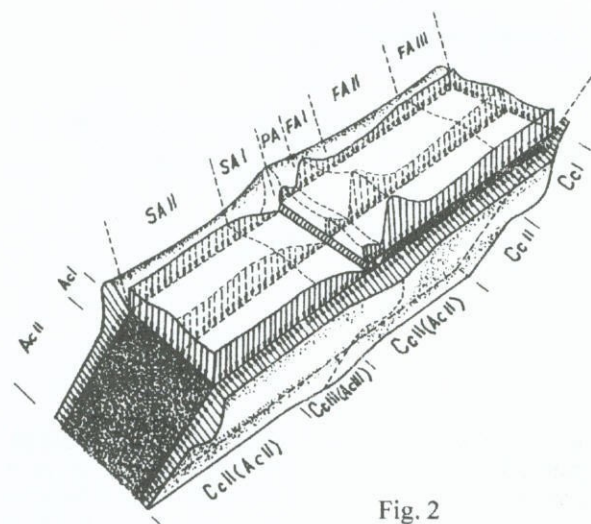


Fig. 2

of faces measurements and observations it was obtained a dependence between the support average pressure acting on the middle face and certain independent variables:

$$P = \frac{6.789 \cdot 10^{-10} \cdot l_{ab}^{3.7} \cdot h_f^{43.1} \cdot H^{8.2} \cdot R_c^{13.9}}{h_{ex}^3 \cdot \alpha^{4.4} \cdot R_a^{21.7} \cdot C^{5.5}}, [10^{-2} \text{ MPa}] \quad (1)$$

where: l_{ab} is face length or horizontal seam thickness, in m; h_f - slice height or face height, in m; h_{ex} - coal seam vertical height already mined, measured from starting level, in m; α - dip seam, in degree; H - deep level of the faces, measured from surface, in m; R_a and R_c - average roof and floor rock strength, in MPa; C - maximum admissible face convergence, in 10^{-3} m/m face mining advancement.

Applied conditions of previous relation are the followings: l_{ab} = 20-110 m; h_f = 2.5-3 m; h_{ex} = 10-380 m; α = 30-80 m; H = 65-580 m.

The model sensibility to the mining height h_{ex} and slice height h_f are presented in figures 3 and 4. It is shown that if the mining height increases and face height decreases, the necessary face support performance is more reduced. In the most difficult Jiu Valley conditions the indigenous powered support used, ensure a maximum convergence of 125 mm/m.

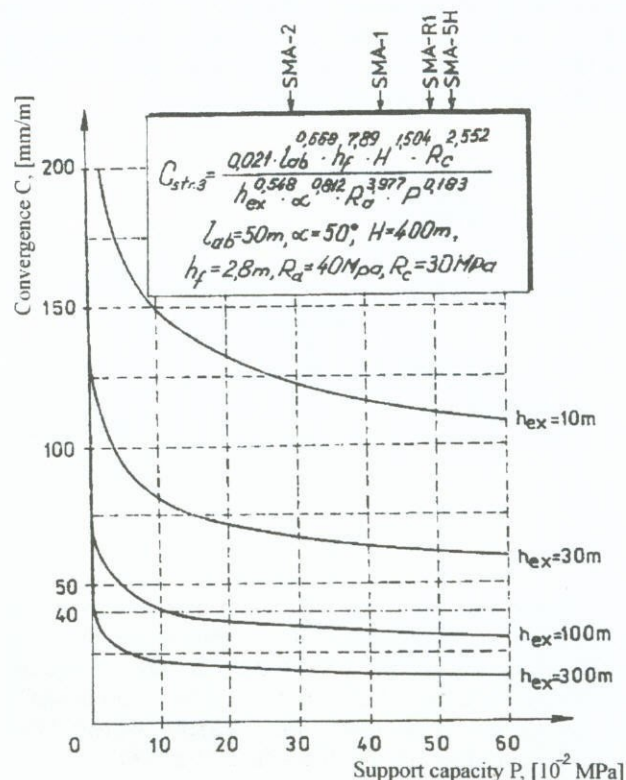


Fig. 3

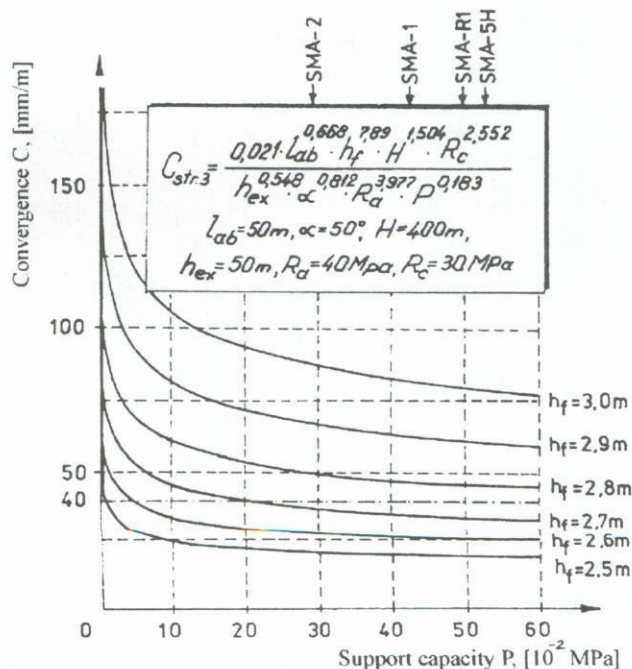


Fig. 4

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Concerning the load distribution on support canopy, it is remarked an approximately parabolical distribution, with the maximum value toward the gob. In the case of face no.69, bock III, Lonea mine, for a support load density of 0.40 MPa, the measured load at the face was about $9 \cdot 10^4$ N/m and toward the gob $38 \cdot 10^4$ N/m (figure 5).

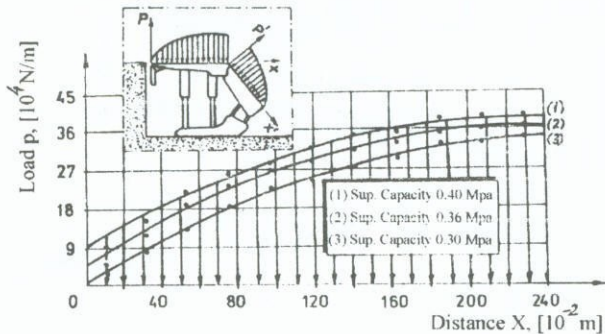


Fig. 5

Along of the face we find a maximum load concentration on the support in the middle length of the face (figure 6). At the floor zone the loads have the values reduced by 30% and to the roof by 50%. In the crossing work zones with the faces, the pressure is two times greater by comparison with the adjacent zones.

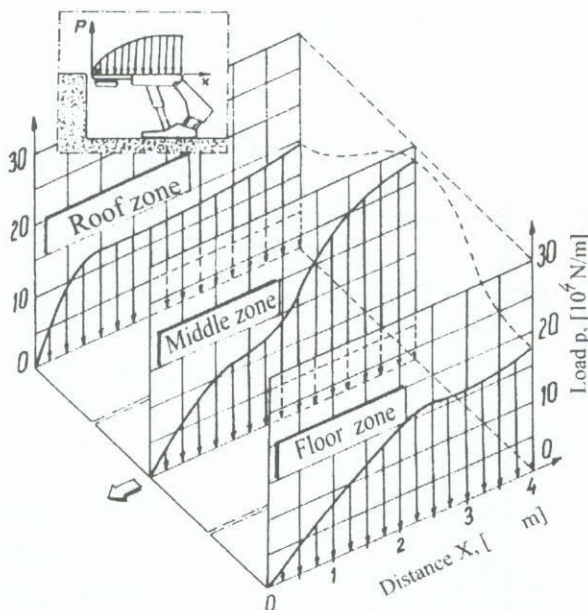


Fig. 6

Front Abutment Stress

Abutment stress acting, immediately in front, on coal face slice, in FA I zone, represented in figure 2, is a consequence of stress distribution around the mining coal area.

In these conditions the normal stress concentration on the coal mass could be over 2 times the initial state of stress[3].

Equivalent materials studies carried out for our conditions, have had as result, the different values for the concentration factor k_n , depending on the depth, measured from coal slice surface and the distance d , from the coal face (figure 7).

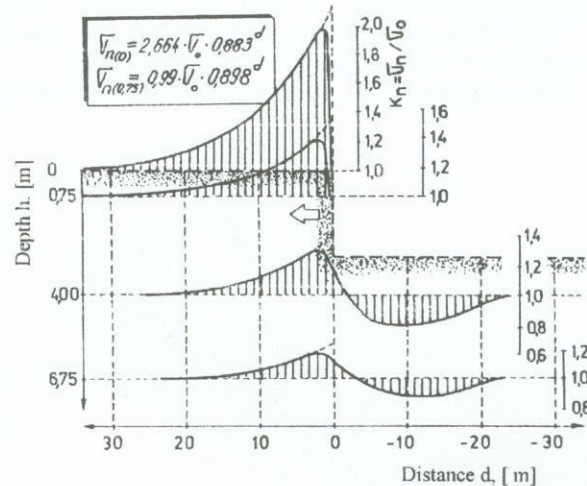


Fig. 7

In the case of coal seam no.3, mined by horizontal slices methods, the front abutment pressure is a result of the difference between the density of intact rock in front of the face and the caved rocks within gob area. The last ones, being in continuous motion, cannot assume the whole dimension of the stress.

In the area FA I, the abutment stress diminishes exponentially, depending on the front face distance, and it is remarked 3 zones are noticed, where the coal mass has different behaviour:

a) on the zone 1, the coal have a visco - plastic behaviour. The coal is fissured because the stress concentration on the front face, immediately after the coal cutting operation. The stress is to the limits of coal residual strength. The extension of that zone is between 0.5 and 1-1.5 m on the front of coal face.

b) inside the zone no.2, the coal has a prevalent plastic behaviour, on about 1-2 m of distance. The state of stress increases deeply, until it reaches the maximum value of the abutment stress development.

c) on the zone 3, the coal behaviour is elastic, the stress has a gradual reduction, over a distance of 20-30 m. In the area of preparatory working - face joints, the stress concentration exceed 4 times the initial state of stress.

Because of the weak coal strength, in the eastern side of the basin, the abutment pressure has as a result the coal failure, followed by a slide of coal face.

Also, in figure no.8 we can observe that the pressures acting along the preparatory work, situated on the roof limit (of maximum 0.05 MPa) are lower than in the case of work situated on the floor limit (of maximum

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0.3MPa). These ones have an exponential distribution, diminishing with the increasing of distance. The lower value of pressure acting on the roof work support is explained by the bridge roof rocks effect. Therefore, the work is situated under the protection of a bridge, generated by non caved roof rock plates. That situation goes on until the rock plates cave, and the pressure increases very much.

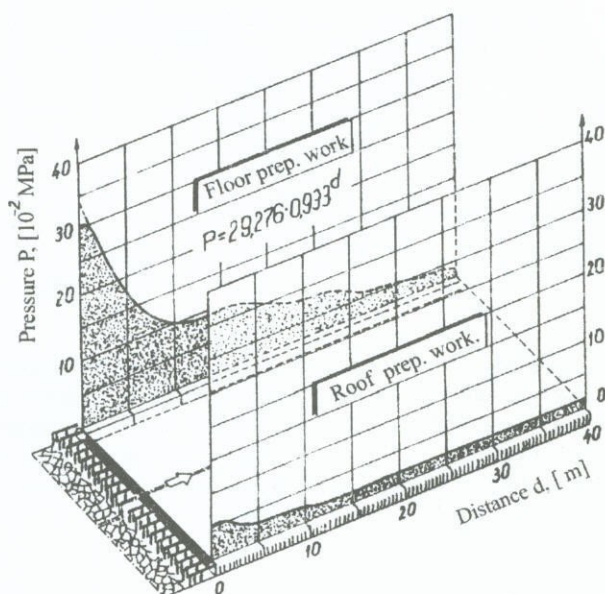


Fig. 8

Taking into account the observations obtained in the caving moment, we can conclude that the pressure has a cyclical development, with an increasing of the pressure acting on the face and preparatory working in the roof area, over 50-60%.

Therefore, there is a steep roof caving following the coal dip, of 5-8 m, and a steep caving following the face advancement direction with the values, ranging between 15 and 20 m.

Stress Distribution on Coal Slice Plane of the Gob

The stresses propagated through the caved rocks in the gob area, respectively on the zones SA I and SA II are generally more important toward the floor coal stratum. These increase in time, as the caved rocks settling increases, having a maximum in the middle of stratum.

This phenomenon was appraised according to the pressure data analysis, measurements being made with pressure transducers, abandoned in the gob area[3].

In the face no.1 East, block IV, Lupeni mine, the measurements showed that maximum stress value is of 1 MPa at roof zone and 1.7 MPa at the floor zone, at 10-12 m distance from the front coal face. In the face no.11, block I, level VIII, Aninoasa mine it was remarked an emphasized increasing of maximum load toward the floor, respectively 2 MPa by comparison with 0.5 MPa, at the roof. In that

case it is marked the stress distribution in the roof caving moment (figure 9).

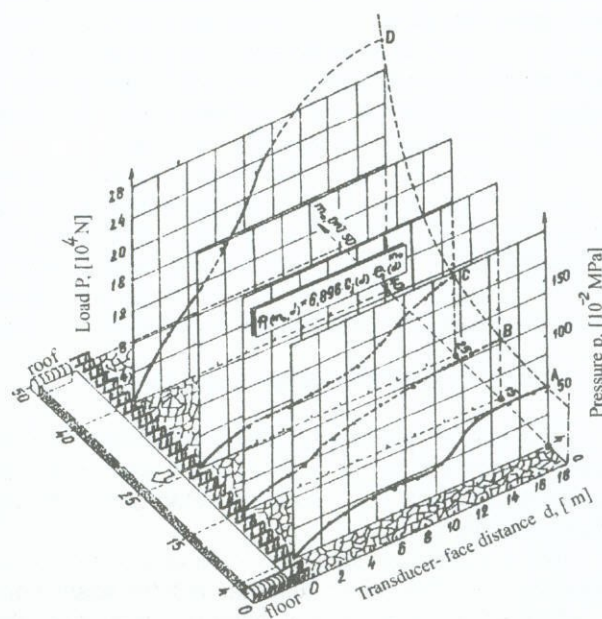


Fig. 9

Normal Stress Distribution on the Coal Stratum Roof and Floor Planes

We take into account the Ac I, Ac II zones at roof plane and Cc I, Cc II, Cc III zones at floor plane[3].

As a result of data obtained by equivalent materials modelling and with finite element method it was established that, because of the mining space influence, under the slice level the normal stress acting on the roof and floor planes, increase up to two times the initial stress along over 20-30 m. Because of the face influence, on the Ac III and Cc III zones, the abutment stress has a supplementary concentration about 30-50%.

Also, it was observed that the floor rocks do not have a slide tendency because of the lateral pressure, generated by the caved rocks in the gob space.

The influence volume of the mined space is very important for finding the preparatory and opening working places that ensure their good stability. Also, it is necessary for designing the working and face supports.

Rock Displacement Mechanism as a Consequence of Coal Seam No.3 Mining

It is very difficult to observe directly by in situ observations the rocks displacements and phenomena developed in the gob space.

Being very complex, that mechanism is not possible to be modelled by numerical methods. Therefore, the most efficient method, in the previous conditions, for describing the qualitative phenomena, is the modelling by equivalent materials. The rocks behaviour simulation of thick coal

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seam no.3 mining was studied in detail by Al.Popescu[4]. After the researches on the equivalent materials models, correlated with the in situ measurements data and observations (for medium and great dip zones of the coal seam no.3) have been identified 3 deformation zones: 1)non regularly caving zone; 2) regularly caving zone 3) bending and slabbing zone (for these zone the rocks deformation mechanism is described in detail).

Concerning the rocks stability of the floor, for great dip zones of coal seam no.3, the researches performed both by Roman[4] and by Popescu[4]showed the fact that the displacements are not important and these cannot to reach the values which determine the rocks floor slide, because the lateral pressure generated by the caved rocks.

GROUND SURFACE MINING SUBSIDENCE

The JiuValley thick coal seams involve about 50% of reserve balance in the main safety pillars of social, industrial and natural objectives, situated on the mining surface perimeters.

For a correct design and drawing of main safety pillars were made complex researches with the special aim of establishing subsidence angles value and the determination of surface ground displacement and deformation mechanism, on the underground mining influence.

Because the mining system of thick coal seam, upper face rocks medium could be assimilated as a granular medium, which is in a permanent movement, following the vertical slices mining[1].

The study of the surface ground displacement and deformation under the underground mining had aimed at solving, two main problems:

- correct determination of subsidence angles with a view to drawing the main safety pillars;
- establishment of surface ground deformation character for finding the prognosis relations of displacement parameters.

For that goal, on the surface of mining perimeters were placed the monitoring stations, for surface ground displacement and deformation measurements (perpendicular and parallel with slice face advancement direction).

The values of subsidence angles calculated by the topographical measurements, used in the main safety pillars design, are the following:

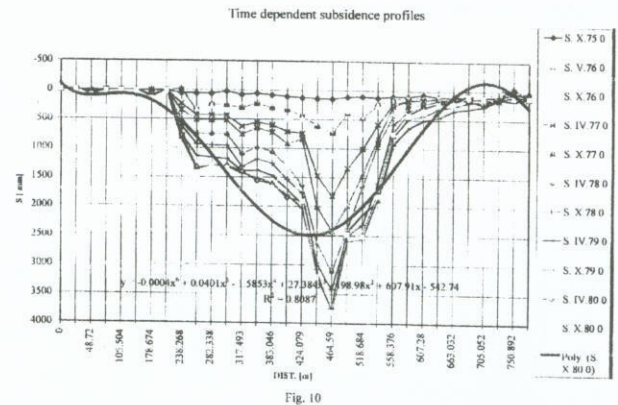
Table no 1.

Subsidence angle	Value
β - on the floor of seam	47° - 52°
γ - on the roof of seam	64°
δ - on the strike of seam	64° - 67°

In situ measurements established the fact that the subsidence profile arose on the mine surface depending on the horizontal thickness of the coal seam[2].

For the horizontal thickness up to 30 m, the subsidence profile, transversal on the coal seam, could be assimilated with an undercritical subsidence profile, and on the strike with an overcritical one.

The subsidence profile, time dependent, is presented in figure no.10.



Taking into account horizontal slice mining system, with caving ground control, above the face is a granular rock medium. The intact roof and floor rock displacement toward the gob, included the ground surface, could be described by a differential equation, using the stochastic mechanics, of pulverulent medium:

$$\frac{\partial^2 w}{\partial x^2} - \frac{\partial^2 w}{\partial y^2} + 2C \frac{\partial^2 w}{\partial x \partial y} = 0 \quad (2)$$

$$\frac{\partial^2 u}{\partial x^2} - \frac{\partial^2 u}{\partial y^2} + 2C \frac{\partial^2 u}{\partial x \partial y} = 0$$

where: W and u are displacement vector of a surface or underground material point of the rock mass, following the vertical and horizontal direction.

The integration of the above equation system, by introducing exponential function of probability, with a limit condition of subsidence extent, lead to relation for prognosis of subsidence profile:

$$W_x = \frac{W_{\max}}{2} \left[\Phi \left(\frac{L-x}{\sqrt{H \cdot h}} \right) + \Phi \left(\frac{L+x}{\sqrt{H \cdot h}} \right) \right] \quad (3)$$

where: (FI) is Laplace function; x-point abscissa on the adopted reference system; L- horizontal distance, between maximum subsidence point and the inflexion point of subsidence profile; H-mining depth; h-distance measured from the ground surface to the rock mass where the rock strata are in a continuum deformation; Wmax=a.g-vertical maximum subsidence; a-subsidence factor, depending on the ground control system and the size of mined coal seam; g- slice thickness.

The value of a factor was determinated by statistical data analysis, respectively by multiple correlation of in situ measurements, depending on the time and l/H and L/H;

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where: l is length of mined horizontal slice; L -strike extent of mined coal seam.

The others parameters of ground surface displacement and deformation as: dip of subsidence profile; curvature, horizontal displacement and deformation are determinate by relation (3), on the basis of mathematical correlation between these parameters.

The theoretical results are proved by the in situ measurements.

These research results have enabled us to correctly design main safety pillars, with important benefits in coal reserve recovery.

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Prevention of Time-dependent Subsidence by Elimination of Ground Movement over an Abandoned Mine

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ABSTRACT

Abandoned coal mines cause subsidence and structural damages. Shallow abandoned mines induce the most severe and harmful damages. In addition, they produce time-dependent subsidence which is difficult to predict. Backfilling of underground cavities with waste materials have been used to prevent the time-dependent subsidence or to mitigate its effect. However, this procedure is expensive.

A new drill and blast technique has recently been proposed to prevent surface damage from time-dependent subsidence over abandoned mines. It utilizes the principle that blasting in the caved rock and roof in the vicinity of a cavity will dilate the ground and create a self-supporting effect owing to increase in the volume.

Physical modeling has demonstrated that artificial dilation of caved rock using the drill & blast technique creates positive effects due to the following sequence of events: the dilated rock creates a supporting effect for the suspended rock strata and reduces its span which decelerates 3.4 times the process of caving. The roof span is reduced by the next caved layer, which causes the hole boundaries to incline to each other. This effect converts the process of caving from a developing mode to a reduction mode. Finally, the sinkhole zone assumes a triangular shape with a negative angle of draw $64-70^\circ$ in contrast to a rectangular shape of the chimney zone for natural caving. Vertical dimension of the triangular zone is 3.87 times of the room height.

Final surface subsidence decreases at least 2.09 times; displacement 1.31 times; slope 2.27 times; curvature 3.42 times; horizontal strain 1.21 times in comparison to natural caving of the ground over an abandoned mine.

INTRODUCTION

Abandoned coal mines cause subsidence and structural damages long after mining. Millions of acres of undermined land have a high potential to subside (Peng, 1992). Shallow abandoned mines induce the most severe and harmful damages. In addition, they produce time-dependent subsidence which is difficult to predict. Backfilling of underground cavities with waste materials has been used to prevent the time-dependent subsidence or to mitigate its effects. However this procedure is expensive.

A new drill and blast technique has recently been proposed (Nazimko, 1996) to prevent surface damage from the time-dependent subsidence over abandoned mines. It was suggested that a special blasting in the caved rock and roof in the vicinity of a cavity may dilate the ground and create a self-supporting effect owing to increase in the rock volume. In this paper, preliminary results of a detailed investigation of the blasting effect have been described.

DESCRIPTION OF MODELING TECHNIQUE

It is impossible to choose the optimum parameters for the new method without proper knowledge of the ground behavior. There are highly irreversible processes involved in the vicinity of an underground opening when a forced caving occurs, e.g. strata caving, cracking, downwarping, creeping, etc. It is impossible to simulate such a behavior by analytical or numerical methods. This is why physical modeling has been employed to investigate ground displacement distribution when the drill & blast technique is used to control the surface over an abandoned mine.

Plaster-sand equivalent material has been chosen to simulate the rock strata. Chalk was added to simulate

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ductile (plastic) properties and plaster for brittle behavior of the equivalent (synthetic) material. Scale factors have been selected as recommended by (Kuznetsov et al., 1968)

$$l_m/L_p = 1:33; \sigma_m = \sigma_p \rho_m / \rho_p (l_m/L_p) \quad (1)$$

where l_m and L_p are linear dimensions of a model and of a corresponding prototype: σ_m and σ_p are any parameters of the model and prototype having stress dimension, while; ρ_m and ρ_p are the densities of the model and prototype, respectively. Sand has been selected from Avdeevsky quartz quarry. It consisted of (by weight) 47% of grain with dimension 1-0.55 mm and the remaining 53% smaller than 0.5 mm. Typical sedimentary rock mass has been modeled. The layer thickness was 0.165 m. Uniaxial compressive strength was 10-30 MPa. Adjacent layers were separated by mica (100g/m²) to reduce cohesion. According to Eq. (1) the uniaxial compressive strength of the synthetic material should be 180-540 kPa. Considering the weathering effect, this value should be reduced by a factor of 0.1. Therefore the final uniaxial compressive strength of the synthetic material is 18-54 kPa. Such a level of strength has been achieved by using the following mixture: 2.8% of plaster; 1.4% of chalk; 3.6% of water; and the remainder is sand. Consolidation of a layer was made by applying mechanical compaction with a specific load of 40 kPa.

The mechanical properties of the synthetic material have been tested in a special equipment (Fig. 1). The conventional testing apparatus is depicted in Fig. 1a. A special device has been designed to test synthetic specimens under rigid load conditions (Fig. 1b). This device consists of a micrometer gauge, 1, with a micro sensing element, 2, for pressure recording. The specimen, 3, is loaded by rotating the micrometer's head (screw), 4. The load applied to the specimen was measured by element 2. Deformation of the specimen is recorded by the head, 4. Fig. 2 shows two typical stress-strain curves. Implementation of the special micro-equipment for specimens testing has established confidence in the mechanical properties of the synthetic materials.

The models consisting of layers of the synthetic material were placed in a special steel testing frame (Fig. 3), the dimension of which was 980 mm wide by 1370 mm high by 19 mm thick. Two 7-mm thick plates of glasses were used one in the front and the other in the back sides of the model. Two identical underground rooms with dimensions 65 mm high by 150 mm wide (2 m by 5 m in-situ) have been excavated at a depth of 440 mm (14.52 m in-situ). Therefore the overburden thickness was 7.26 times of the room height. Distance between the outer side of either one of the two rooms and the lateral boundary of the model was 226 mm (7.46 m in-situ). Distance between the rooms was 230 mm

(7.6 m in-situ). This was sufficient to eliminate the effects of lateral boundaries on strain distribution.

To simulate the drill & blast technique, three anchors were placed in the vicinity of the right-side room: the upper anchor was 2 m in the roof, the middle anchor was on the floor, while the lowest anchor was in the floor layer 1 m below the room floor. After the rock layers had caved, the anchors were pulled by cords in descending order. Such a procedure simulates consecutive blasting of explosive to generate artificial dilation of the ground.

The most difficult problem was to comply with the boundary conditions at the top of the model, i.e. the top should be free because it was a free surface. However subsidence in the model should occur in a reasonable period of time, say, in dozens of minutes or in hours at the least. The only widely accepted method that meets this requirement is centrifugal modeling. Unfortunately, this technique is expensive and labor-consuming. Therefore a new technique was developed. Equivalent time in the model is accelerated by vibration (Fig. 3). A vibrator was mounted in the center at the bottom of the steel frame. Its power P was 45W. Its magnitude of electromagnetic core oscillation was 6 mm, and the frequency of oscillation was 50 cycles per second. Testing of the technique on a small light (120 N) frame has demonstrated that a reasonable rate of acceleration of the time can be achieved: the immediate roof caves after 7-8 seconds and caving of strata propagates to the surface after 10 seconds.

It is not easy to find a scale factor for time. According to Kuznetsov et al., (1968), the most reliable approach is to compare duration in a model and in a natural prototype for a well-known process. This approach was adopted in this research. In addition, the scale factor was theoretically and qualitatively estimated. To this end, the thermodynamic concept of durability T_p for the rock was used (Nazimko and Grayson, 1995) by considering rock collapse as a phase transition. Note the area S under a curve in Fig. 2 is proportional to the energy that should be spent for the transition. This energy accumulates in a real rock mass naturally due to irreversible processes under gravity action and can be supplied by a vibrator to a physical model in a period of time, t_m , i.e.

$$P \cdot t_m \cdot a = S \quad (2)$$

where a is a proportionality coefficient which can be derived from a special experiment. Briefly, the more the power of a vibrator, the less time required to collapse a model. Finally, the time scale factor f_t has been derived as a ratio of the duration of the prototype to model for a well-known process: $f_t = T_p/t_m$. Caving of a roof may be adopted as such a process.



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Ground movement was recorded by special technique. 144 monuments were laid out on the front side of the model. At least four base monuments had to be pasted on the front glass. To increase accuracy, eight base monuments have been pasted. Successive states of the model have been recorded on a photo film. These photographs were then scanned by a scanner. A special computer code had been developed before to read the coordinates of the monuments on a picture. This program records monument coordinates in the scanner system (namely in pixels). The program is founded on affin transformations and treats the raw data and corrects them considering that the base monuments are motionless. It had been established that a scanner produces stochastic mistakes. These mistakes induces errors of readings up to ± 4 pixels. In addition, mistakes accumulate because of replacement photo camera, deformation of the film during its development, human imperfection, etc. To determine the total accuracy of ground movement measurement, preliminary readings were made from the same state of the model taking on two pictures. A total of 68 readings were made. Figure 4 depicts the mistake distribution (in meters of natural rock mass). It can be seen that it resembles a normal distribution with a standard deviation of 0.016 m. Therefore the accuracy of ground deformation measurement will be no worse than ± 0.032 m at 95% confidence level.

After having determined the ground displacements in the horizontal and vertical direction, their distributions were treated by SURFER. A specially made finite element computer code was then used to determine the linear, tangential and volumetric strains in a vertical section of the model and the conventional characteristics of surface deformation, namely vertical subsidence, horizontal displacement, slope, curvature and horizontal strain.

ANALYSIS OF SUBSIDENCE CHARACTERISTICS IN VERTICAL SECTION

The initial state of the model is depicted in Fig. 5 (Photo 2). For comparison purpose they were colored with green neutral paint. Figure 6 (Photo 3) shows the next stage of the model's treatment when the immediate roof had started to cave. Such a state occurred 210 minutes after vibration. It can be seen that the process of caving occurred simultaneously in both rooms. It should be noted that the rock layers caved in good order, occupying approximately the same orientation as they had in the virgin rock mass before caving. Figure 7 (Photo 4) shows the state of the model after 6 hours of vibration and the roof caved to a height of 2.5 m. The strain characteristics were calculated from this moment and forward.

Experience has shown that vibration causes the sand to crumble down and consequently the model body becomes thinner and subsides as a whole. This made it impossible to use the pasted monuments as the base monuments. To solve this problem, individual virgin sectors at the top of the model were selected as the top base monuments. Such an approach excluded relative consolidation of rock mass and avoided or at least dramatically reduced distortion of the data.

Although there were 144 monuments on the model body, it was not sufficient to determine all strain components with relevant accuracy throughout the whole model body. However strain distributions obtained so far can discern main peculiarities of ground subsidence process in all stages. Apparent difference between the rooms in stage #5 can be seen (Fig. 8, Photo 5). At this stage, the preliminarily placed charges of explosive have been blasted. From this moment and forward, the subsidence process over the right-side room lagged behind 2.8 m.

Another drawback of the experiment was identified. The vibrator power was not sufficient to accelerate the caving process in a big heavy model (760 N). The wave energy dissipated in the vicinity of the bottom of the model and could not initiate strata caving far from the vibrator. Hence additional dynamic flow of energy was needed for ground control. To correct the deficiency, the experimentalists reactivated ground caving by periodically knocking the frame with two wooden blocks. Such an action broke the time scale, but it did not affect the final results essentially.

Continuing ground deformation caused the propagation of strata caving up to the surface over the left-side room (Fig. 9, Photo 6). It should be noted that the strata fell abruptly to the whole height and subsided in proper order. The closer to the surface the better the caved layers maintained its original order. One may see that a sinkhole or a chimney mode of subsidence propagated up to monument #31, namely 4 times of room height over the room. This parameter falls into those predicted by Peng, (1992). In addition, it should be noted that the boundaries of the sinkhole are vertical.

On the contrary, the hole boundaries are inclined toward each other over the right-side room where the artificial dilation of the caved rock was made. Caving height over the right-side room at stage 5 was 2.1 m while that over the left-side room was 7.1 m. Thus the process of caving subsided over the left-side room was at least $7.1/2.1 = 3.4$ times over the right-side room. It occurred because the self-supporting effect reduced the roof span over the right-side room. Immediately after the blasting, the span reduced from 5 m (initial width of the room) to 3.5 m (see Fig. 8). Roof span reduction continued and reduced to 2.5 m at stage 6 (Fig. 9). Roof span disappeared completely at stage 7 (Fig. 10). This

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stage of subsidence was actually the final stage because neither vibration nor additional striking on the frame altered the condition in the model. It is important to note that the process of progressive reduction of sinkhole subsidence finished at monument #103 or at the 3.87 times of room height over the right-side room. A minor ground disturbance was detected over this point by electronic means. The hole boundaries of the final sinkhole were 64-70° from the vertical. Hence the positive effects of artificial dilation of caved rock consists of the following sequence of events: the dilated rock creates a supporting effect for the suspended rock strata and reduces its span resulting in 3.4 times of reduction in roof caving. Every subsequently caved layer reduces the roof span, which causes the hole boundaries to incline toward each other. This effect converts the process of caving from a developing mode to a declining mode. Finally, the sinkhole zones assumes a triangular shape with a negative angle of draw 64-70° in contrast to a rectangular shape of the chimney zone in natural caving. Its vertical dimension of the triangular zone is 3.87 times of the room height.

ANALYSIS OF DEFORMATION AT THE TOP SURFACE OF THE MODEL

Figures 11 and 12 depict the vertical subsidence and horizontal displacement characteristics, respectively. Considering the above-mentioned threshold of accuracy ± 0.032 m, one should conclude that surface subsidence up to stage 4 may be neglected. Furthermore, even state 5 is an untypical subsidence. Maximum vertical depression occurs over the pillar between the rooms (Fig. 13). This was probably due to the interaction and consolidation of the whole model body mentioned above. Horizontal displacement distribution seems to prove this point (Fig. 14). It may be neglected as well.

Surface subsidence became natural at stage 6 (Figures 15 and 16). Maximum subsidence over the left-side room occurs approximately over its center and is 0.546 m or 27% of the room height. Maximum surface depression over the right-side room is 0.228 m or 11% of the room height and 2.39 times less than that over the left-side room. Horizontal displacement may still be neglected at this stage. Slope reaches 0.0028 over the left-side room whereas it is 0.0012 over the right-side room which is 2.33 times less. The same is true for the curvature: $8.9 \cdot 10^{-5}$ over the left-side room against $2.3 \cdot 10^{-5}$ over the right-side one, which is 3.87 times less. The final stage of subsidence has mitigated these differences (Fig. 17 and 18). However large differences remain: subsidence 2.09 times; displacement 1.31 times; slope 2.27 times; curvature 3.42 times, and horizontal strain 1.21 times (see Table 1).

CONCLUSION

The positive effects of artificial dilation of caved rock using the drill & blast technique result from the following sequence of events: the dilated rock creates a supporting effect for the suspended rock strata and reduces its span which decelerates the process of caving by 3.4 times. Every subsequently caved layer reduces the roof span which causes the hole boundaries to incline toward each other. This effect converts the process of caving from a developing mode to a reduction mode. Finally, the sinkhole zone assumes a triangular shape with a negative angle of draw 64-70° in contrast to a rectangular shape of the chimney zone in natural caving. Vertical dimension of the triangular zone is 3.87 times of the room height.

The final surface subsidence decreases at least 2.09 times; displacement 1.31 times; slope 2.27 times; curvature 3.42 times; and horizontal strain 1.21 times in comparison to natural caving of the ground over an abandoned mine.

New findings will help develop optimal parameters for ground control and further reduce subsidence values.

ACKNOWLEDGMENT

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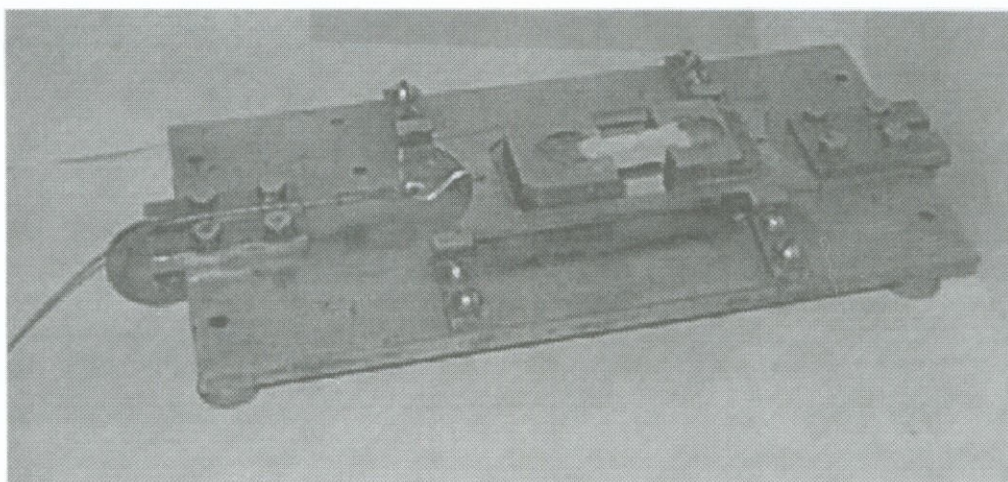
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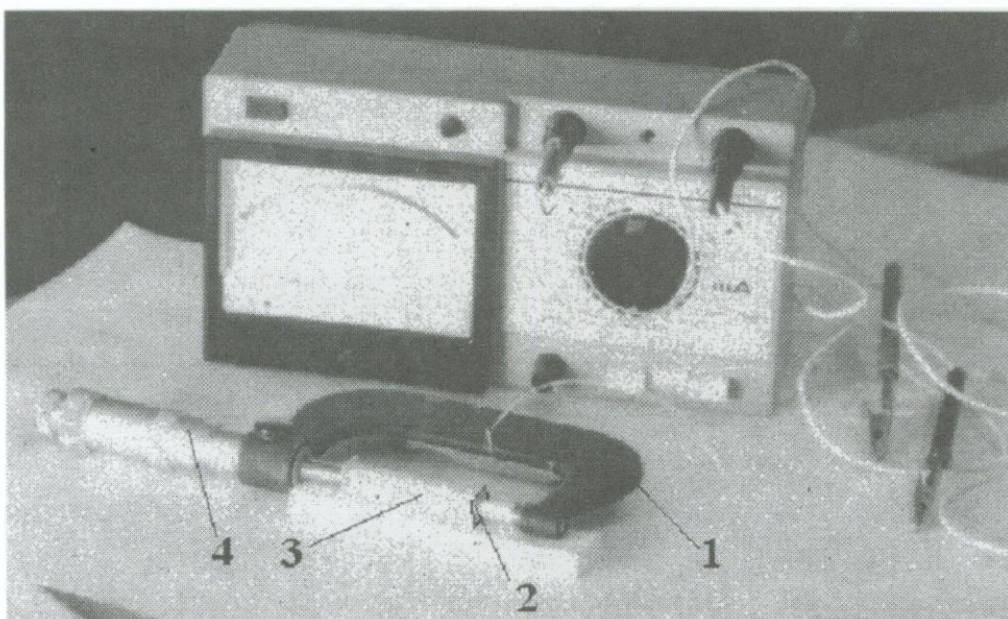


Table 1. Subsidence characteristics

Stage	Vertical subsidence, S, m			Horizontal displacement, U, m			Slope, i			Curvature, K, m^{-1}			Horizontal strain, ε		
	Left-side room	Right-side room	S/S_r	Left-side room	Right-side room	U/U_r	Left-side room	Right-side room	i/i_r	Left-side room	Right-side room	k/k_r	Left room	Right room	$\varepsilon/\varepsilon_r$
6	-0.546	-0.228	2.39				0.0028	0.0012	2.33	$8.9 \cdot 10^{-5}$	$2.3 \cdot 10^{-5}$	3.87			
7	-0.692	-0.331	2.09	-0.042	0.032	-1.31	$3.67 \cdot 10^{-3}$	$1.62 \cdot 10^{-3}$	2.27	$1.13 \cdot 10^{-4}$	$3.3 \cdot 10^{-5}$	3.42	$9.7 \cdot 10^{-4}$	$-8 \cdot 10^{-4}$	-1.21



a . Conventional device



b . New device

Fig. 1. Testing equipment for synthetic specimen

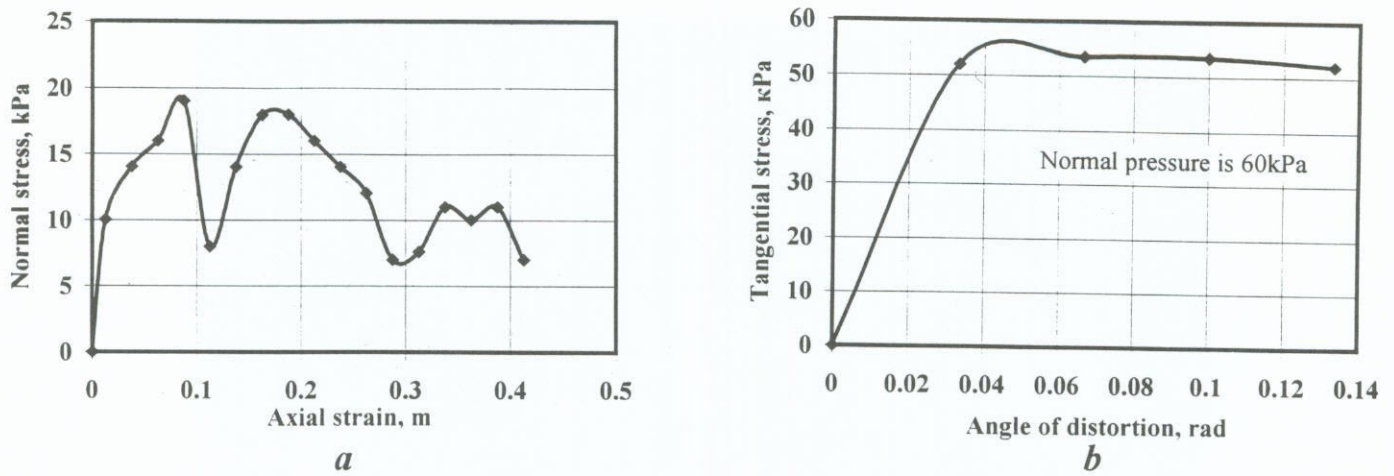


Fig. 2. Test results of synthetic specimens:
 a - stress-strain relation for uniaxial compressive test;
 b - stress-strain relation for shear test

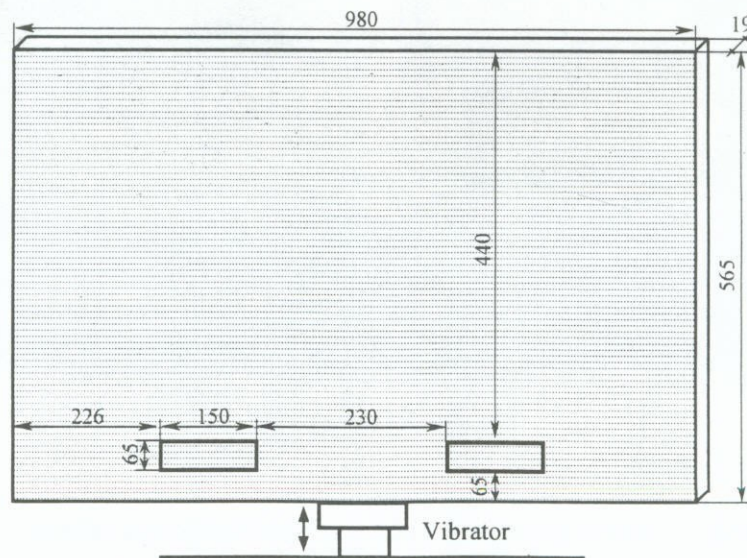


Fig. 3. Setup of physical model

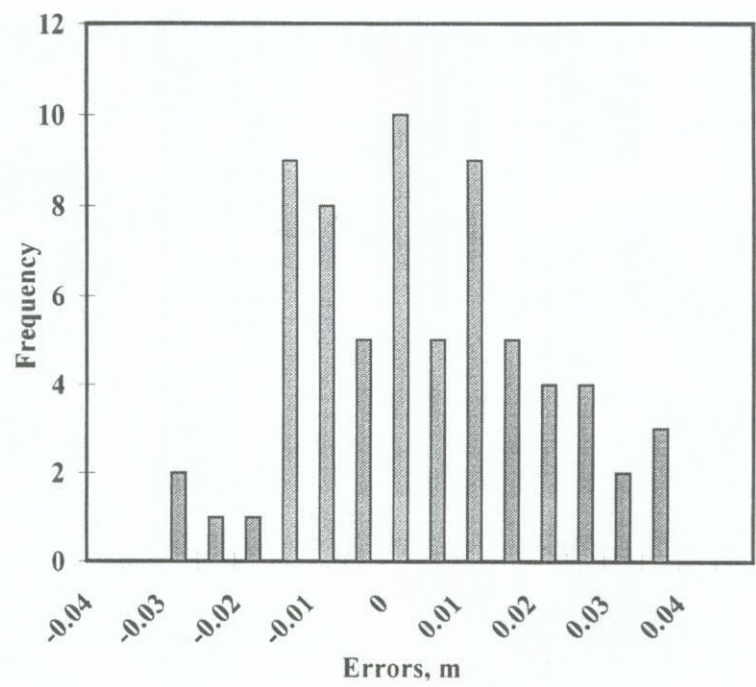


Fig. 4. Histogram of distribution of measurement errors

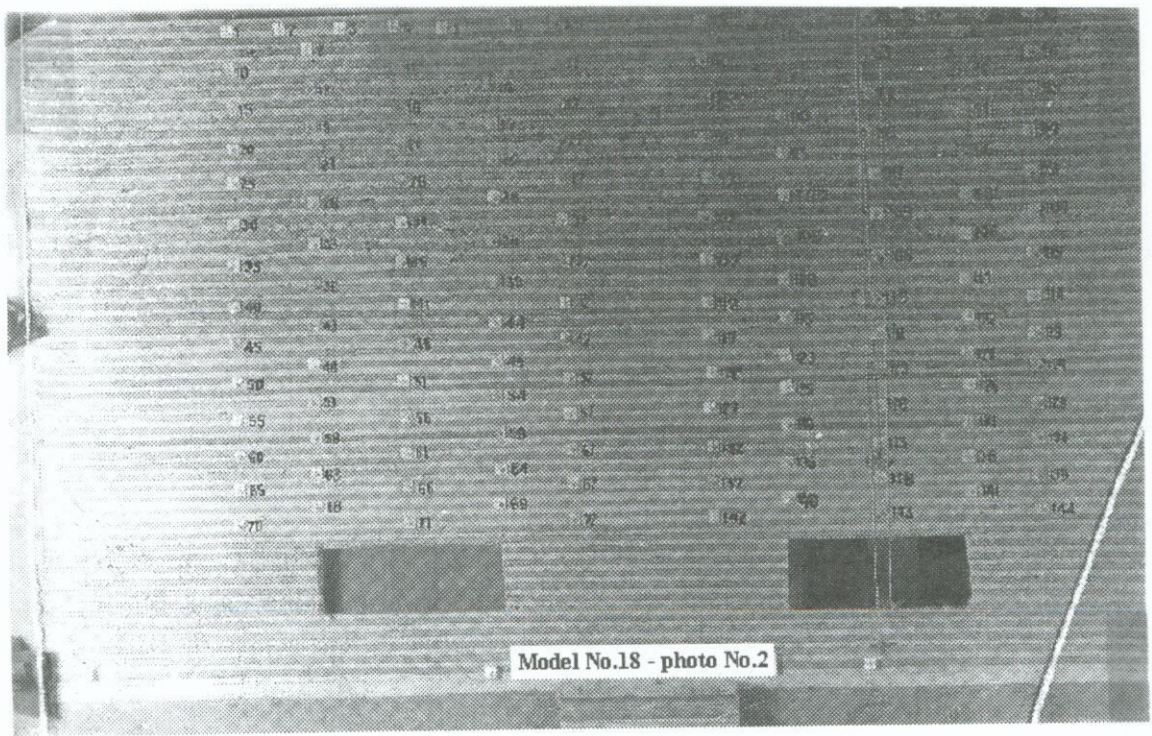


Fig.5. State of the model before testing

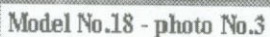


Fig.6. State of the model at stage 3

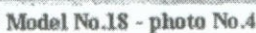


Fig.7. State of the model at stage 4

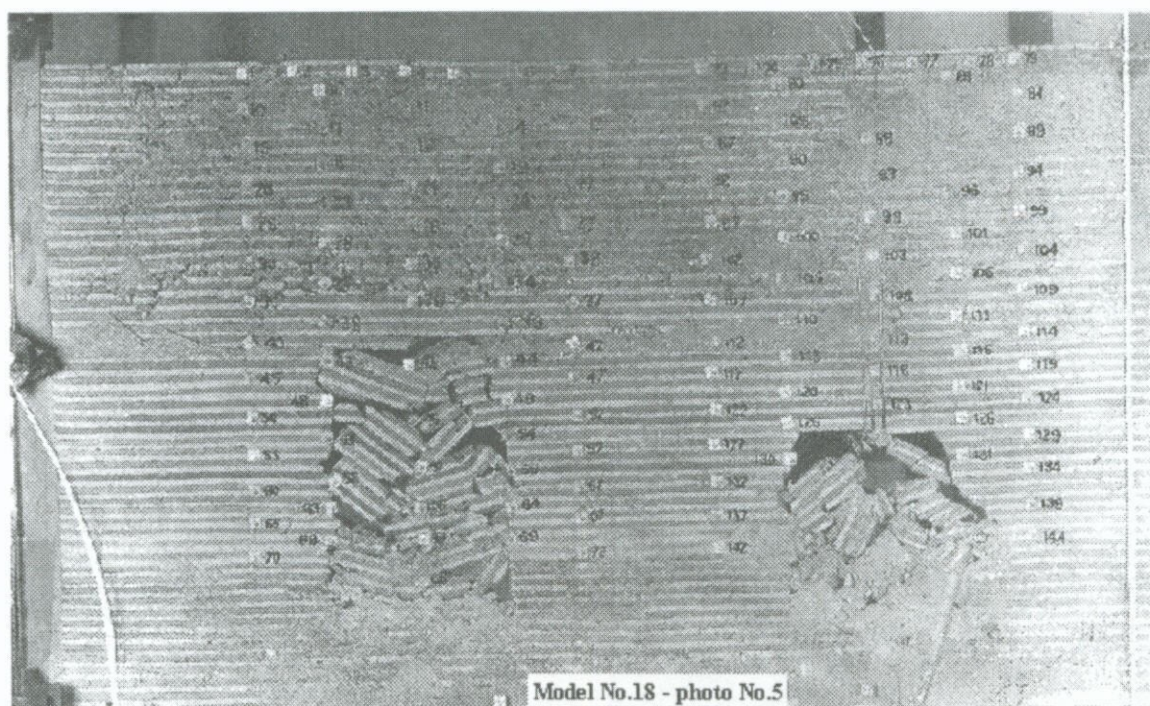


Fig.8. State of the model at stage 5

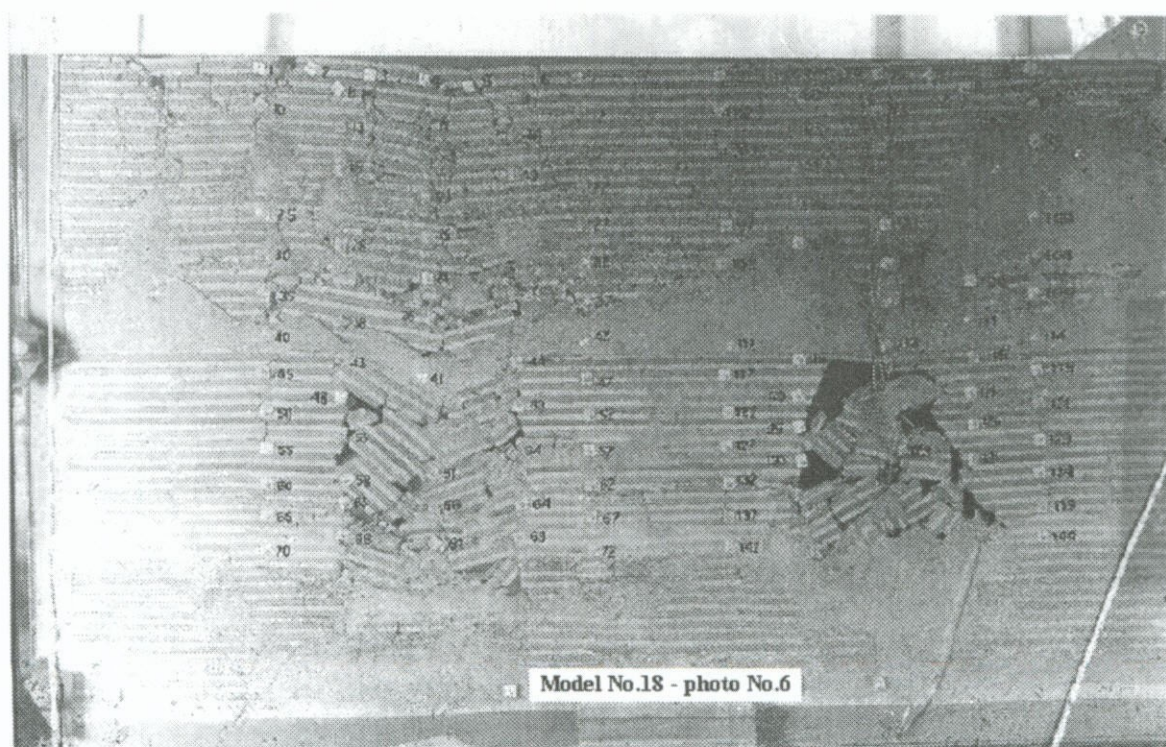


Fig.9. State of the model immediately after propagation of subsidence to the surface (stage 6)

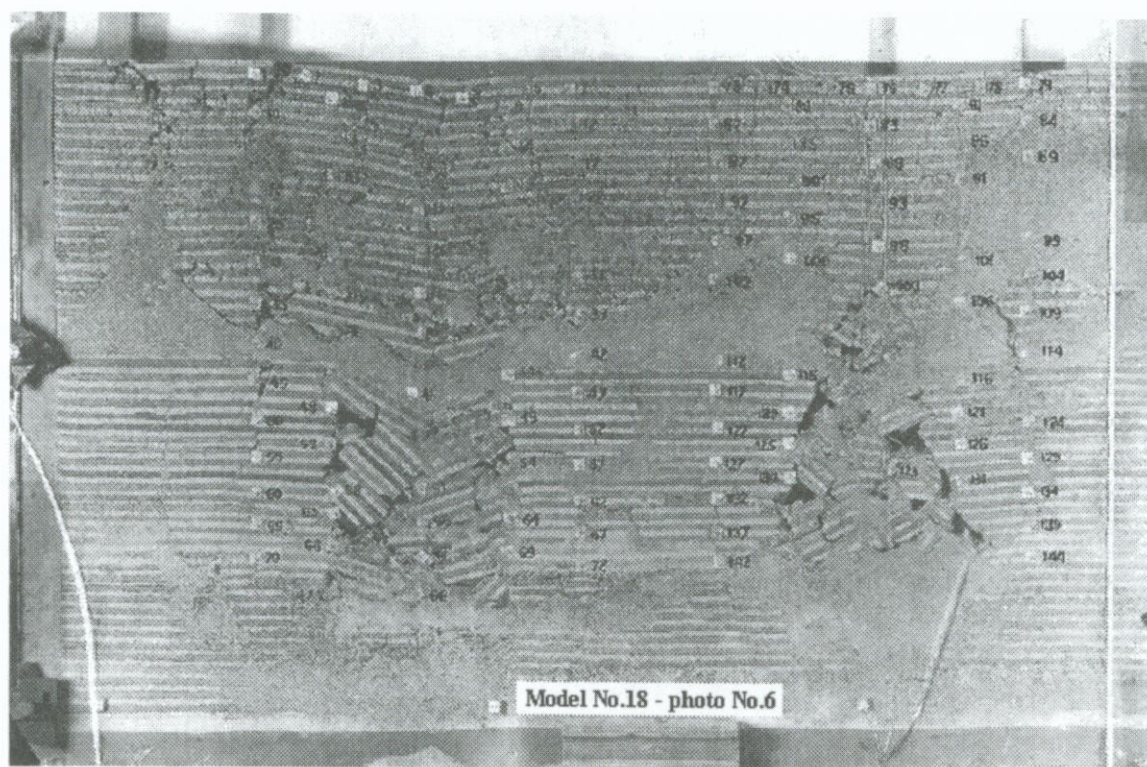


Fig.10. Final state of the model (stage 7)

Fig. 11. Subsidence, slope & curvature profiles at stage 4

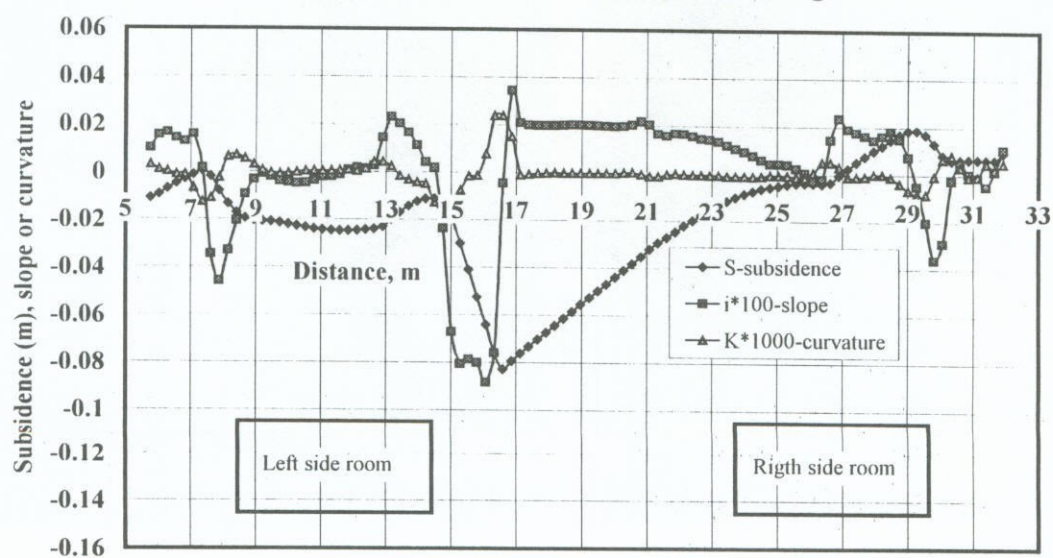




Fig. 12. Horizontal movement and strain profiles at stage 4

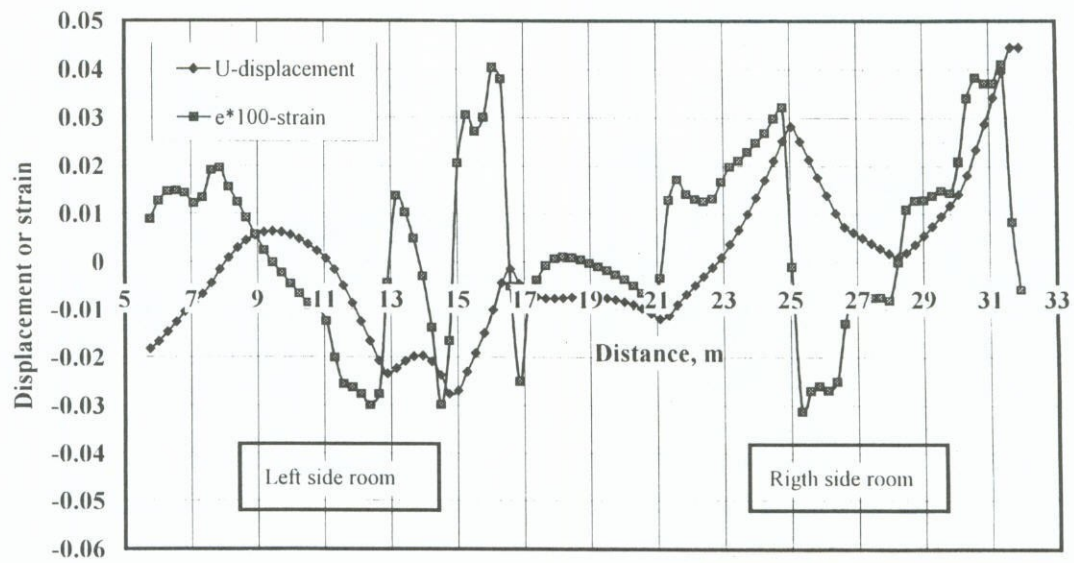


Fig. 13. Subsidence, slope & curvature profiles at stage 5

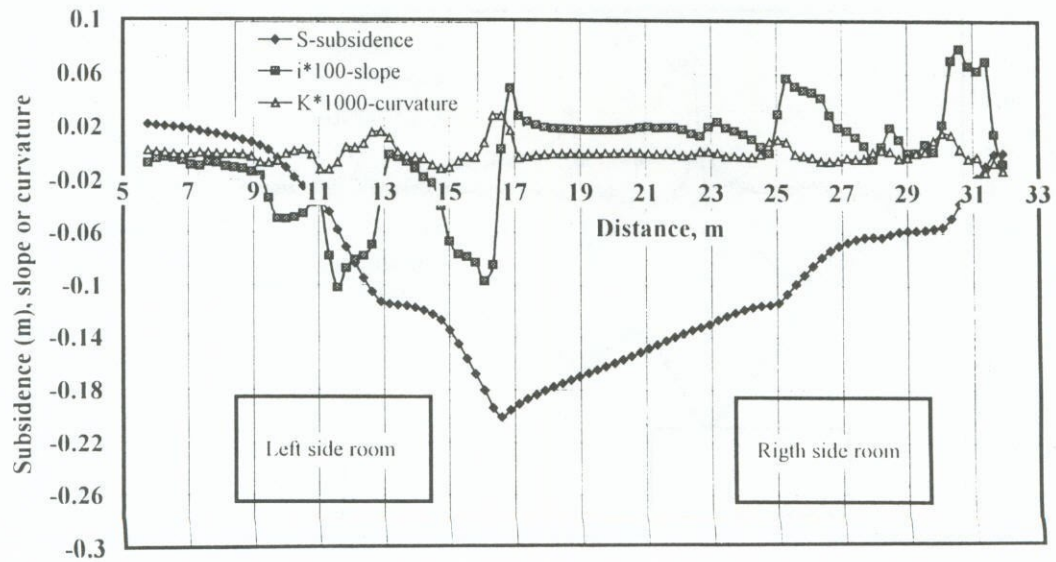




Fig. 14. Horizontal movement and strain profiles at stage 5

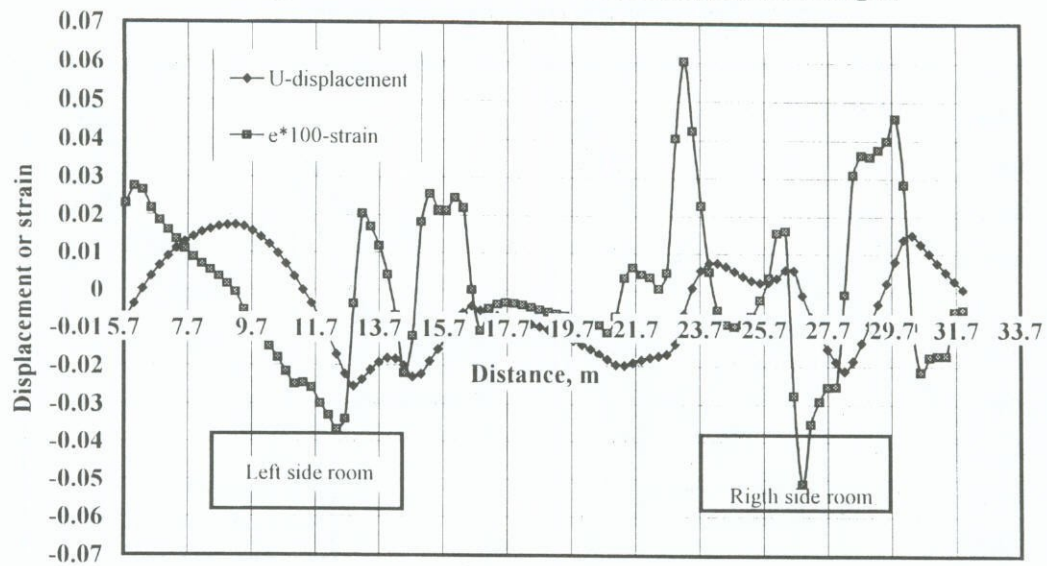


Fig. 15. Subsidence, slope & curvature profiles at stage 6

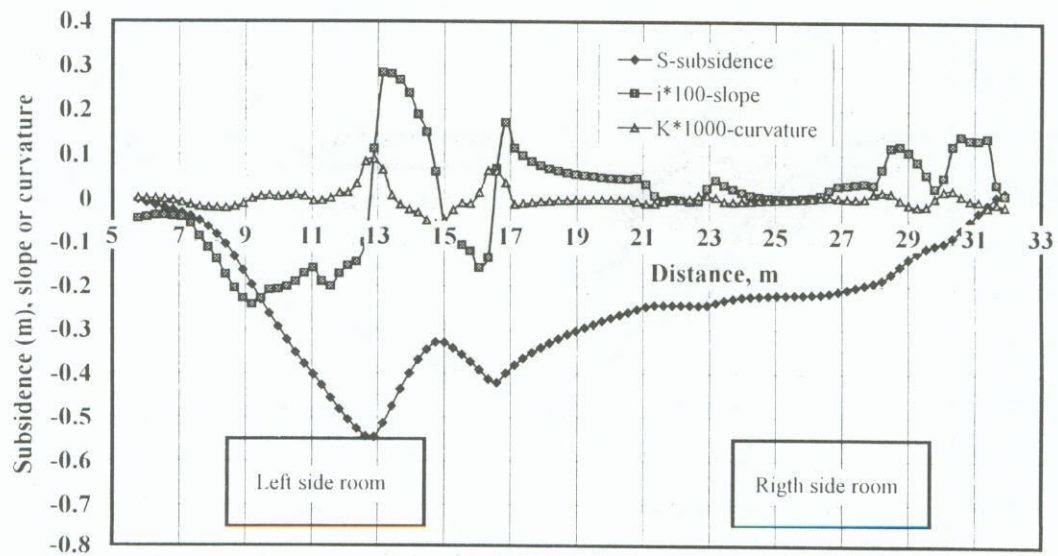




Fig. 16. Horizontal movement and strain profiles at stage 6

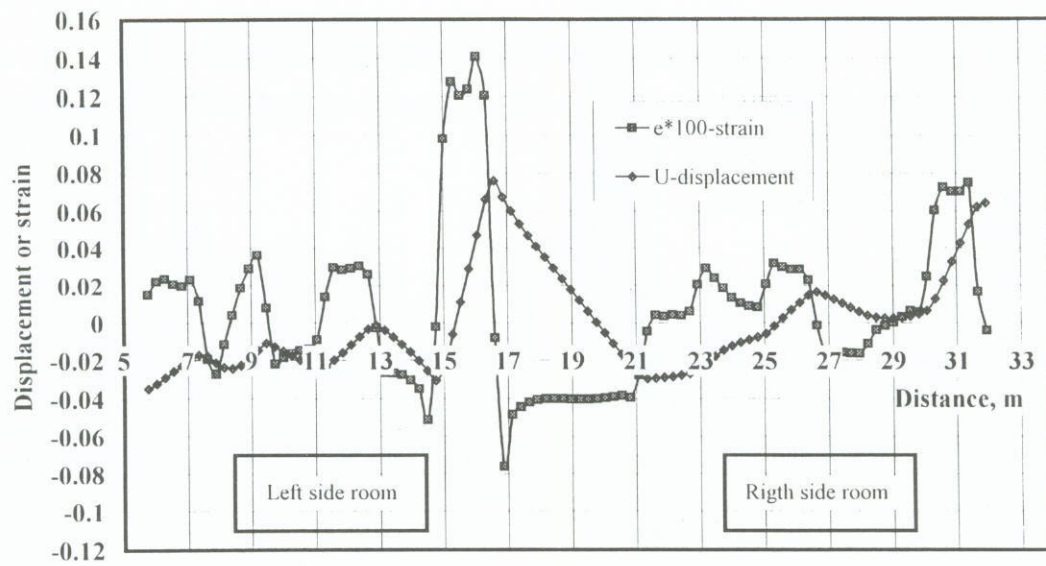


Fig. 17. Final subsidence, slope & curvature profiles

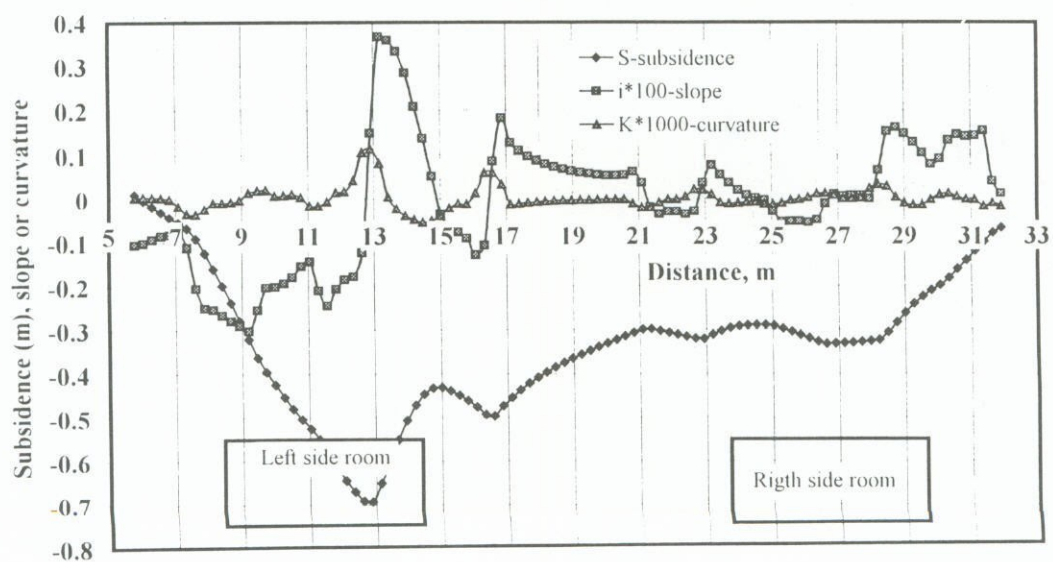
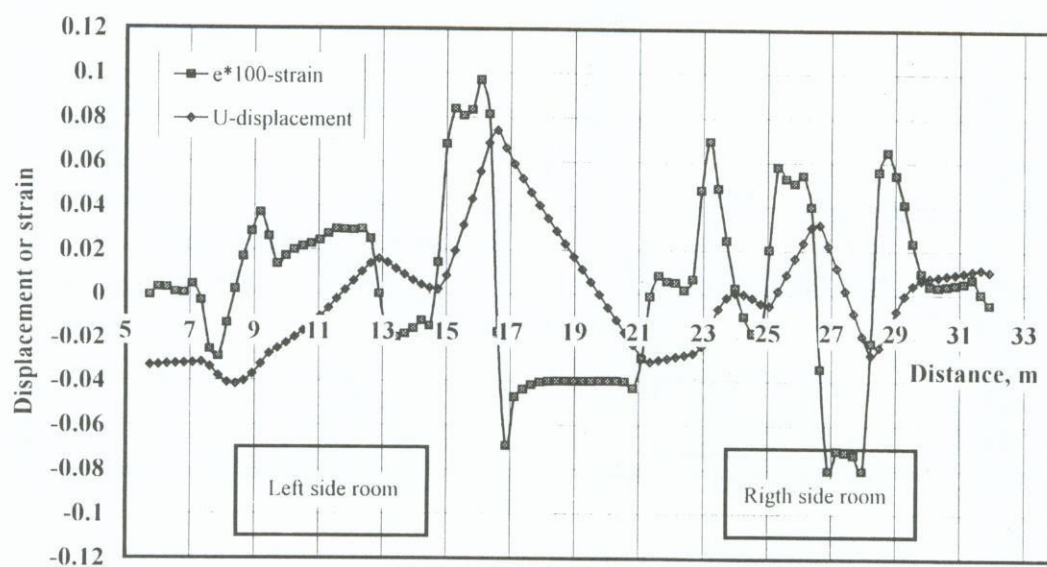




Fig. 18. Final horizontal movement and strain profiles





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