THE RESPONSE OF POWERED SUPPORTS AND PILLARS TO INITIAL LONGWALLING UNDER A STRONG MAIN ROOF

By

M.B. Wold1 and J. Pala2

ABSTRACT

The Ellalong Colliery has been the first to mine the Greta Seam using the longwall method. Analysis of data from geotechnical monitoring has pointed to the role of the strong, massive sandstones of the Branxton formation in the development of loads on the powered supports and pillars. Failure of the sandstone intermediate-roof has been related to first- and periodic weights, measured on the supports, using simple "voussior" analysis. Stress changes, including triaxial components measured in the barrier and chain pillars, have been compared with 2- and 3-dimensional numerical models. The applicable range of the models for the prediction of pillar loads under the strong roof is discussed.

INTRODUCTION

Australian experience in longwall mining has in many cases been gained under conditions of strong and massive roof. This has presented the miner with geotechnical problems involving large caving spans and heights, and has implied heavy loads on powered supports, pillars and abutments. The widespread use of longwalling was delayed following early experience with equipment insufficiently robust to cope with problems associated with the strong roof, but with the introduction of high capacity chock-shield-type supports the use of the method is growing rapidly.

At the Ellalong Colliery in the Hunter Valley of NSW, ground conditions unsuitable for bord and pillar mining led to the mine being planned and developed for retreat longwalling from the outset. Being the first to attempt this method of mining the Greta Seam meant that no direct experience was available to the mine planners for the purpose of support design and mine layout. Rock mechanics investigations were carried out for several years before the start of production in 1981, and the results of these investigations and their influence on mine planning and support selection has been summarised by Yeates, et al (1983) and McKandry and Yeates (1983).

Investigations have continued since then, with the aim of further optimizing mining operations. During production from the first longwall block, automatic monitoring of the powered-support pressures was undertaken by the CEIRQ (Wold and Pala, 1986) in parallel with a study of stresses and deformations in the longwall block, chain pillar and gateroads undertaken by ACIRT (Pala, et al, 1984).

It became apparent fairly early in the extraction of the first block that abutment loads were being distributed widely. Large gateroad closures and rib spall occurred ahead of the face and were associated with the sudden onset of heavy support loading. Rib spall was also observed in the development headings, which were separated from the block by an 75 m wide barrier pillar. The strong and massive sandstone roof, which lies above the relatively friable 10 m to 15 m thick lower roof of the Greta Seam, was thought to be largely responsible for this behaviour.

With mining of the first coal block well advanced, another instrumentation programme was begun, with stress changes and roadway closures being monitored in the barrier pillar and the chain pillar (Wold and Pala, 1986). This work included the measurement of pillar stress changes in three dimensions, which gave some insights into the way in which pillars tend to deform around the caving zone.

The failure of the first massive sandstone stratum in the roof of the Greta Seam was reflected in the response of the powered supports. This was seen as the "first weight". It was analysed in terms of a "voussior" plate failure, and the subsequent development of periodic weighting was also analysed in terms of failure of this stratum. With respect to the pillar responses to mining, the measured distributions of side abutment stress showed evidence of higher than expected stresses in the central region of the barrier pillar, and irregular stresses in the chain pillar which had regions of both high stress concentration and also stress relief in close proximity. This may have been related to failure of the strong roof soon after passage of the coal face. These measurements of the behaviour of the powered supports and pillars, and their analysis, are outlined in this paper.

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The AusIMM Illawarra Branch, Ground Movement and Control related to Coal Mining Symposium August 1986

194
MINE ENVIRONMENT

GEOLOGY AND LAYOUT

The Greta Seam at Ellalong is 3.0 m to 4.7 m thick and lies at depths from 320 m to 640 m. It is overlain by fairly massive sandstones and some shales. The seam dips at about 3.5° to the south-east, and longwall no.1 was mined at depths from 320 m to 640 m (Fig.1). The face width is 150 m and the length of block no.1 is about 1550 m. The block is separated from the five development headings by a barrier pillar 75 m wide and a single tailgate roadway. Chain pillars 75 m wide by 95 m long separate the maingate and trackgate roadways for longwall no.1, the latter becoming the tailgate for longwall block no.2.

The lower roof of the Greta Seam consists of conglomerates, sandstones and shales, up to the Pelton Seam. This lower roof varies from about 5 m to 16 m, with a general tendency to increase in thickness down dip. Above the Pelton Seam, more massive sandstones extend to the surface. The floor of the Greta Seam in the initial longwall blocks consists of about 0.1 m of moderately soft mudstone with underlying siltstone, grading down to a siltstone/sandstone laminites.

STRENGTH OF ROOF, FLOOR AND COAL

Detailed lithological and mechanical properties of strata from about 3 m below to 27 m above the Greta Seam have been given by Hall (1978) and Wallman (1980). The intact specimens of roof strata tested had unconfined compressive strengths (UCS) in the range 60 MPa to 90 MPa, with the weaker materials tending to lie closer to the seam (Fig.2). The Young's moduli (E) tended to be directly proportional to the strength, ranging from about 4 GPa for the floor mudstone to 10 - 20 GPa for the roof sandstones. In the selection of the supports, the bearing capacity of the floor was an important factor. It was investigated using plate bearing tests (Enever, et al., 1979), and the siltstone/mudstone horizons chosen for the working floor had capacities of 15 MPa or greater, while dry.

Relatively little is known about the higher overlying strata. Structurally, the lower roof has little jointing, but parts of the upper roof are well jointed.

For the purposes of calibration of instruments and for numerical modelling (discussed later), triaxial compression tests, direct shear tests and dynamic resonance tests were carried out on 50 mm, 54 mm and 16 mm diameter specimens of Greta Seams coal (Wold, 1986). The results are summarised in Table 1.

PRE-MINING STRESSES

Stress measurements using hydraulic fracturing and overcoring methods (Wallman, 1980) indicated that the pre-mining vertical stress was a principal stress consistent with the depth of overburden, i.e. about 10 MPa at a
depth of 400 m. Because of practical difficulties, the measured values of horizontal stresses were subject to some doubt. The ratio of horizontal to vertical stress was within the range 1.0 to 2.0 with the former case more likely. No preferred direction of horizontal principal stress was identified from the tests, and the best estimate was that the pre-mining stress conditions were lithostatic.

**TABLE 1. Mechanical properties of Creta Seam coal from triaxial compression, direct shear and dynamic resonance tests**

<table>
<thead>
<tr>
<th>Core diameter</th>
<th>Mohr construction</th>
<th>Triaxial construction</th>
<th>Direct shear</th>
<th>Dynamic</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UCFS</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>K</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50, 56</td>
<td>4.7</td>
<td>3.9</td>
<td>36.5</td>
<td>23.5</td>
</tr>
<tr>
<td>147</td>
<td>2.2</td>
<td>4.3</td>
<td>38.6</td>
<td>6.7</td>
</tr>
<tr>
<td></td>
<td>1.8</td>
<td>0.34</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

C is cohesion, $\phi$ is angle of friction, K is triaxial stress factor, $\nu$ is Poisson's ratio.

**INSTRUMENTATION**

**POWERED SUPPORTS**

To record the leg pressure history, an intrinsically safe data logging system, designed and built at the CSIRO Division of Geomechanics, was installed on eight supports. The support hydraulic pressures were monitored in pairs at the quarter, centre and three-quarter face positions, and singly at the maingate and tailgate ends. On each support, one front and one rear leg pressure was monitored, selected diagonally on the support.

The system recorded data on cassette tape underground, resolving pressure to 0.4 bar. Generally data were recorded at 2 minute intervals, but continuous (5 sec.) scanning was used as required. Manual observations of support closure, and shearer and support advance sequences, were made in parallel with continuous pressure monitoring at various times.

**PILLARS**

Stress monitoring gauges were installed within the barrier pillar and the chain pillar at sites approximately 1260 m and 1460 m respectively from the starting roadway. The majority of the gauges were of the TRAD vibrating-wire type which measured uniaxial (in this case vertical) stress changes.

Although it is general practice to measure only the vertical component of coal pillar stress changes, a more complete appreciation of pillar behaviour requires knowledge of the transverse response of the pillar to changing load conditions. In the design of pillars, assumptions of horizontal stresses and stress changes are important in the prediction of yield zones at the pillar ribs. The depths of these zones are often measured using multi-point extensometers, but the horizontal components of stress change generally remain unknown.

In response to this need, several CSIRO triaxial stress gauges were also installed adjacent to the TRAD gauges. These enabled measurement, by each instrument, of the complete triaxial stress change. Their use as a stress monitoring device in coal is discussed further by Wold and Fals (1986).

All gauges were placed in horizontal boreholes at the mid-height of the pillars. In the barrier pillar, they were installed from a pillar cut-through roadway. The IRAD gauges were placed 6 m deep and generally spaced at 10 m intervals, 20 m to 100 m from the tailgate roadway. Three CSIRO gauges were placed at depths ranging from 3.4 m to 11.0 m, about 30 m from the tailgate roadway. Measurements of roof, floor and coal rib displacements were also made at intervals along the cut-through, adjacent to the IRAD gauges.

In the chain pillar, and at adjacent sites in longwall block no.2, IRAD gauges were installed from the crackgate roadway at depths ranging from 6 m to 16 m. They were also installed in longwall block no.1, 6 m deep from the maingate rib. In the same region, CSIRO gauges were installed on the central line of the chain pillar, at a depth of 10 m from a pillar cut-through.

All instruments were installed at least one face-width in advance of the longwall face passing.

**RESPONSE OF SUPPORTS TO MINING**

The powered supports which were supplied by Dowty Mining Equipment are of the 4-leg check shield type, with a capacity at yield of 600 tonnes at 386 bar pressure, and a nominal setting load of 480 tonnes at 310 bar pressure. The operating height range is 1.85 m to 6.1 m, with a nominal mining height of 3.5 m. The setting density before cut, one web back, is nominally 70 tonnes/m². The system comprises 98 supports at 1.52 m centres, for a face width of approximately 150 m. Coal shearing was uni-directional using a double ended ranging drum shearer, with the supports one web back. Top coal was cut towards the maingate from about support no.72.

The force generated by a lemniscate shield type of support is a function of the leg.
pressures (hence forces) and also their geometry and that of the lemniscate linkage. An analysis, based on that of Wilson (1983), showed that for the Ellalong supports, the resultant canopy force may be assumed to be approximately equal to the sum of the leg forces. The line of action of the force vector could also be readily calculated. Knowledge of the leg pressure history would enable estimation of the strata loading on the supports during the caving process.

The general features of the support-leg pressure curves in response to the mining cycle have been well exemplified, for instance, by Bates (1978).

FIRST WEIGHT

Although no direct experience of longwalling the Crater Seam was available, the presence of competent and extensive sandstone beds above the Pelton Seam suggested that rather large spans might develop during the course of mining. This could be particularly expected during the early stages, before the caving zone had developed to its full extent. Heavy support loading, often known as first weight, would be associated with the failure of a large initial span. At the same time, heavy loading on pillars and abutments, with associated rib spall and the possibility of floor heave, could be anticipated.

First roof falls occurred at a span of 20 m. As mining progressed, caving was observed to extend up to the Pelton Seam, as expected. The support monitoring showed that leg pressures seldom approached yield, for spans up to about 110 m (Table 2, from manual readings of all support legs across the face). At this stage, pillar rib spall about 20 m in advance of the face indicated that transfer of load from the main roof to the abutments was intensifying.

Continuous monitoring records showed that very small load changes were apparent on the supports during successive mining cycles (Fig. 3). As the mining span approached 127 m, several cycles produced yielding, followed again by lower loads. Shortly after production ceased, at 127 m, very large roof fall were felt right across the face and all monitored supports except at the tailgate went into continuous yield, front and rear legs (Fig. 4). Roof fracture appeared to originate near the maingate end (support no. 23) and propagate both ways along the face. The heavy support loading was accompanied by severe face spall, extensive spall on the coal block side of the maingate, and rib spall in the trackgate.

Based on the span at first weight, 127 m, and the material properties of the sandstone beds lying immediately above the Pelton Seam, the approximate thickness of a sandstone roof "plate" whose failure could lead to the sudden heavy support loading, was calculated. This calculation assumed the development of a "voûte" plate, in which the sandstone roof bed, having separated under its own weight from

<table>
<thead>
<tr>
<th>Span</th>
<th>Leg pressure (per cent of legs)</th>
<th>m</th>
<th>above 380</th>
<th>350-380</th>
<th>below 350</th>
</tr>
</thead>
<tbody>
<tr>
<td>110</td>
<td>1.7</td>
<td></td>
<td>1.7</td>
<td>89.0</td>
<td></td>
</tr>
<tr>
<td>127</td>
<td>52.8</td>
<td></td>
<td>30.4</td>
<td>16.8</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 3. Leg pressures for several mining cycles at span of approx. 110 m.

Fig. 4. Leg pressures at span of 127 m, showing onset of "first weight".

The AutIMI Illawarra Branch, Ground Movement and Control related to Coal Mining Symposium August 1986

197
small span/thickness ratios and low shear strength. Snap-through failure tends to occur at larger span/thickness ratios, as a result of either plate buckling or compressive crushing at the centre of abrupt hinge points. Initial failure of the immediate roof at the 20 m span may have been as a result of shear, or progressive snap-through of successive thin beds immediately above the Orata Seam.

Assuming a span of 127 m at failure, the design curves of Reer and Mall were used to derive the plate thickness as a function of plate span/width ratio, 0.5 and UCS (Fig. 5). For an estimated in situ E of about 6 GPa for the sandstone (about 1/2 times the values from the small test specimen, Fig. 2), and taking into account the increased possibility of buckling due to the pre-mining horizontal stress, the estimated plate thickness would be 6 m – 8 m.

It is considered that failure of the 8 m thick sandstone unit close above the Pelton Seam led to the development of first weight.

PERIODIC WEIGHTING

During subsequent mining two types of support response were distinguished, and these responses tended to recur on a cyclic basis. The more prominent response comprised a relatively slow build-up of pressure over a major portion of a mining cycle, then a rapid increase during the advancement of the neighbouring supports. Leg closures followed the same pattern, and were generally 4 mm – 6 mm. Pressures and closures tended to be increasingly concentrated towards the rear of the supports. The second type of response comprised a rapid build-up of pressure to yield, soon after setting, with closures of the order of 25 mm, and similar behaviour of front and rear legs. This response tended to follow after a number of cycles of the first type. Fig. 6 demonstrates the pattern.

Examination of the records for all the supports at this time showed that strata failure propagated from the region of the tailgate towards the minate, encompassing at least supports nos. 75 – 25.

A mechanism of roof deformation and failure which might lead to the observed support pressures and closures, and their periodic nature, assumes the existence of lower and intermediate roof zones below the main roof, following the general concept of Wilson (1983).

At Ellalong the lower roof has been observed to cave readily behind the supports at an angle of about 10°, with a generally small overhang. This lower roof is bounded approximately by the Pelton Seam, about three extraction heights above. This implies a coal bulking factor of 1.3 or less, which is reasonable for the rather disordered caving of the sandstones, conglomerate and lamine strata of the lower roof. The intermediate roof is considered to comprise the 6 m – 8 m sandstone bed discussed in relation to the development of first weight. Thus a general section of the coal face region can be depicted as in Fig. 7.

From this 2-dimensional geometry and the support setting and yield leads a cantilevering mechanism of the intermediate roof would lead to periodic weighting over about 6 – 8 mining cycles (Wold and Pala, 1986).

3-dimensional stress and geometry near the gateroads, and cantilevering parallel to the...
faceline associated with the support advance sequence, complicate the simple 2-dimensional analysis. These factors and the variations in roof structure encountered over the length of a longwall block would be expected to alter the frequency and intensity of periodic weighting as mining progressed.

**UPPER ROOF BEHAVIOUR**

At the completion of mining longwall block no.1, surface subsidence was negligible (about 80 mm). This implies that the competent upper roof sandstone was bridging across the caving zone at some unknown horizon, with the goaf remaining partly unconsolidated and the adjacent barrier and chain pillars carrying higher loads. To estimate the minimum thickness of the sandstone bed required to bridge the 150 m span, a vousoir plate analysis was applied assuming plane strain conditions (i.e. span/width ratio large, = 150/150 for longwall block no.1).

As before, stability is strongly dependent on the Young’s modulus of the plate, but for these plate dimensions the crushing strength may be the controlling factor. The analysis showed that the fine grained sandstone bed, 14 m thick and 23 m above the top of the Greta Seam (Fig 2) may be stable at this span unless the mass modulus and strength were both less than half those of the test specimens. Thus at the completion of mining the first block, significant subsidence may not have extended beyond 20 m - 25 m above the top of the Greta Seam.

Extensometer measurements in a vertical borehole from the surface above longwall no.2 (Holba, 1986) showed immediate caving to a height of 12 m (lower roof), followed by caving to 36 m (intermediate roof + next massive sandstone unit - see Fig.2) when the face was 3 m - 16 m past the borehole. These results...
Support the predictions of the vousoir analysis, independently made by Pala and Wold (1986), concerning expected roof behaviour during mining of the second longwall block.

RESPONSE OF PILLARS TO MINING

BARRIER PILLAR

Observational evidence of heavy abutment loads being distributed about the longwall block more broadly than might have been expected on theoretical grounds tended to be supported by the field measurements. The distribution of vertical stress across the barrier pillar, measured by the IRAD gauges, is shown in Fig. 8 for various positions of the longwall face relative to the measurement site. A 3-dimensional displacement discontinuity model analysis was carried out for comparison, using the program MINLAY (Wardle, 1984). Some instruments indicated vertical stress levels 2-3 times higher than expected in the centre of the pillar. It is problematical as to what degree these high stresses represented concentrations caused by local variations in coal or roof, rather than to the bridging action of the strong sandstone of the intermediate and upper roofs. However, roadway closures at this site, and at an earlier measurement site in the barrier pillar, were also concentrated near the centre of the pillar (Fig. 9).

The general trend of stress gradient across the pillar, as indicated by the remainder of the instruments, was similar to the model. However, although the elastic model stresses stabilized about one panel width from the face, the measured stresses continued to increase until the face had progressed at least 200 m (Fig. 10). This indicated that time-dependent stress redistribution accounted for up to 25% of the total barrier pillar load (pro-mining + mining-induced). In contrast, the front abutment stresses, measured in the block being mined and thus having little time to creep, agreed well with the model.

The front abutment stress was the same at span 1466 m as previously measured at span 310 m, suggesting that the roof caving horizon was unchanged over most of the length of the extracted block.

CHAIN PILLAR

Stresses in the chain pillar, while initially increasing smoothly as the face approached, became strongly concentrated towards the maingate side of the pillar and destressed towards the trackgate rib as the face passed. (Fig. 11). A similar destressing was measured in the adjacent rib of the second longwall block, across the trackgate. A sharp reversal of stress when the face was 20 m past was detected by the CSIRO gauge in the centre of the pillar, followed by a rapid rise to a very high level (Fig. 12). At the same time the central IRAD gauge showed a large increase then became unserviceable.

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It is thought that high loads were transferred to the pillar during failure of the sandstone intermediate roof cantilevering from over the powered supports and the chain pillar. Pillar cracking may have led to the formation of the stress-relieved and stress-intensified zones within the pillar. Since the instruments were red manually, higher stress peaks are likely to have gone undetected.

Extrusion of the soft floor, caused by pillar punching and evident as floor heave, may have also contributed to stress relief in the coal ribs.

**Triaxial Stress Changes**

The triaxial stress changes measured by the CSIRO gauges indicated a strong shear stress component \( \tau_{V-V} \), parallel to the bedding, and complementary shear stress \( \tau_{N-S} \), sub-parallel to the cleat (seen on Fig. 12). The complete triaxial changes, shown schematically in Fig. 13, were symmetrical about the caving zone. The tops of the pillars tended to shear inwards towards the goaf, and there was considerable relief of horizontal stress in this direction. However, horizontal stress increased in the direction of pillar length.

The development of horizontal shear stress components implied the rotation of the principal stresses from the vertical-horizontal directions. This rotation, expected a priori, was confirmed by a 2-dimensional analysis of the pillar stress-changes with mining. The analysis was carried out using the boundary element program BEMR (Crotty, 1983). The model stresses were lower in magnitude than those measured, and significantly, the horizontal stress increased with caving, in contrast to the in situ decrease. This decrease would lead to a lower ultimate pillar strength than predicted by numerical models.

The shear stresses parallel to the bedding and cleat were considered in terms of possible modes of pillar rib failure in the model. Two criteria were used:

1. assuming the presence of a ubiquitous set of parallel joints within the model pillar, and using frictional and cohesive properties from triaxial and direct shear tests (Table 1), the possibility of shear failure long the joints was computed for a range of joint orientations.
2. assuming homogeneous and isotropic properties within the pillar, the possibility of shear failure through intact coal was computed.

With these assumptions, shear failure through intact coal to a depth of about 2 m was predicted, compared to values of 1.8 m obtained in situ using multi-point extensometers. Shear along the horizontal bedding or vertical cleat was not predicted, but the higher shear stresses measured in situ suggest that it could still be possible. The higher measured shear component may have been related to a lower shear modulus along the bedding of the coal, which was suggested by the dynamic resonance tests on the coal.

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CONCLUSIONS

1. The strong intermediate roof of the Greta Seam at Ellalong led to heavy first support conditions on the powered supports, which were observed at a span of 127 m. This roof comprised the 6 m - 8 m thick sandstone stratum based about 10 m above the roof of the Greta Seam.

2. Periodic loading of the supports, with a period of 4 - 6 mining cycles, was related to the cantilever failure of this same sandstone bed.

3. Failure and well-developed subsidence may not have extended more than 100 m - 250 m above the roof of the Greta Seam during the life of longwall no. 1. The thick and massive main roof strata above this horizon are likely to have bridged from longwall block no. 2 to the barrier pillar.

4. The visual evidence of heavy abutment loads being distributed about the longwall block more broadly than might have been expected on theoretical grounds tended to be supported by the field measurements. Some instruments indicated vertical stress levels 2 - 3 times higher than expected in the centre of the barrier pillar and roadway closures were concentrated in this region. However, the general trend of stress gradient across the pillar, as indicated by the remainder of the instruments, was similar to that from a 3-dimensional elastic numerical analysis.

5. Barrier pillar stresses stabilised with the face 1.0 - 1.3 face-widths past the measurement site. Time-dependent stress increases in the barrier pillar accounted for 25% of the total pillar stress (initial + induced). This was not evident in the front abutment and could not be predicted by the elastic model.

6. Chain pillar stresses were high adjacent to the mainege roadway but the rockgate side of the pillar was destressed. It is thought that failure of the lower roof as the face passed and subsequent failure of intermediate roof above it may have led to the stress relieved zones in the rockgate ribs. Extrusion of the soft floor, evident as floor heave, may have contributed to destressing of the rib.

7. The triaxial stress measurements indicated a tendency for the top of the pillars to shear inwards towards the caving zone as the face passed. Shear stresses components developed parallel to the bedding and cleat; a 2-dimensional numerical analysis produced similar shear stresses but of lower magnitude. Computation showed that shear failure in the ribs would be through intact coal rather than along these planes.

8. As the face passed, the horizontal components of normal stress increased in the direction of mining, but were relieved in the direction parallel to the face, i.e. towards the caving zone. This stress relief would tend to lessen pillar stability. The 2-dimensional model showed the opposite effect, with an increasing horizontal stress in the pillar core. This model did, however, predict rib yield to a depth of 2 m, compared to measured values of 1.8 m.

9. The 3-dimensional displacement discontinuity program MINLAV provided a good estimate of the regional stress distribution around the longwall workings, but tended to underestimate the stress magnitudes, which increased as the rock crept. The 2-dimensional boundary element program BITEMJ provided an economical first-pass estimate of pillar stress magnitudes. The chain pillar monitoring showed that agreement broke down when pillar stresses were affected by adjacent roof failure. Prediction of pillar strength at higher stress levels, for instance after mining subsequent longwall blocks, requires post-yield strain softening material properties.

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