Fourth Australia - New Zealand
Conference on Geomechanics,
Perth, 14-18 May 1984

Monitoring the Effect of Massive Sandstone Roof in a Longwall Operation at West Cliff Colliery, New South Wales

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SUMMARY Results of monitoring the first longwall at West Cliff Colliery are presented. The monitoring programme had the express aim of establishing the stability of the gate roadways and the intervening pillars, overlain by a massive sandstone roof, as input data for future longwall operations at the colliery. The parameters monitored included (i) pillar load close to the face (ii) convergence in gate and main roadways (iii) convergence in other roadways located around the longwall (iv) load on face support chocks (v) stress in other pillars ahead of the longwall face. The results of these investigations showed that the size of the pillars is adequate and the convergence of the roadways is within acceptable limits. Excessive loading occurred on four-way roadway intersections and in future these will be eliminated. Results of observations also show that roof behaviour in the goaf is greatly influenced by the joint system in the sandstone, leading to asymmetrical failure of the goaf behind the face.

1 INTRODUCTION

West Cliff Colliery is one of the three operating mines of the Coal Cliff Collieries Pty. Ltd. The colliery came into production in 1976 and since then it has mined 8.6 million tonnes of high quality coking coal. The colliery mines the Bulli Seam having an average thickness of 2.5m at an average depth of 475m. The seam is intersected by a series of faults (shear zones) striking at 300-310° (magnetic). The immediate roof rocks consist of shales and mudstones in the northern and western areas of the colliery holdings, while in the central region it consists mainly of sandstone. The main roof invariably consists of massive sandstone of very high strength. Figure 1 shows a stratigraphic section close to the centre of the Longwall 1 coal block, along with the material properties. While the strength of the Bulli coal varies between 12-22 MPa, the strength of certain sections of roof rock is as high as 130 MPa. The weighted average uniaxial compressive strength of the strata in the roof up to eight times the seam thickness (∿ 20m) is 110 MPa.

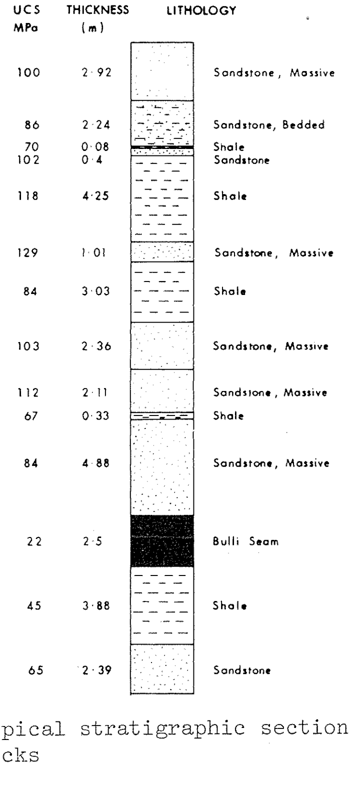

Figure 1 Typical stratigraphic section of roof and floor rocks

2 INITIAL INVESTIGATIONS PRIOR TO INTRODUCTION OF LONGWALL MINING

Earlier experiences in mining by the highly productive longwall operation at this depth under massive, high strength sandstone have been very limited. The earliest experiences on record in this area were the unsuccessful longwall mining attempts at Coal Cliff Colliery, Wollongong. In 1962 in the 6 North district a longwall face of 137m length was established. The height of the seam in that area was 1.5 - 2m overlain by a massive sandstone roof. The face support selected had a density of 100 tonnes per linear metre with individual legs having a capacity of 20 tonnes. This face experienced problems associated with the collapse of gate ends, falls of stone between the chocks and running in of goaf material, and excessive weighting and eventual iron binding of the face support system due to excessive convergence (> 300mm) and development of shear cracks on the face. The face was withdrawn after less than 12 months' operation. In late 1964 a second unit was installed in the 7 North area of the Coal Cliff Mine. It had a capacity of almost 2.5 times that of the most common and successful system then in use in Europe and was expected to be adequate to handle the massive sandstone roof. The support system had a capacity of 245 tonnes per linear metre. The length of the face was 165m. Problems developed which were very similar to those of the first installation. The failure of links between support and face conveyor due to floor heave resulted in stoppage of the face for a week. Roof weighting of the face occurred leading to collapse of roof between the face supports and the coal face on an ever-increasing scale. This face was also withdrawn after only eight months' operation.

These experiences of mining by longwall were very disappointing and they left a general impression that longwall mining under the massive sandstone roof conditions existing in the area was not possible

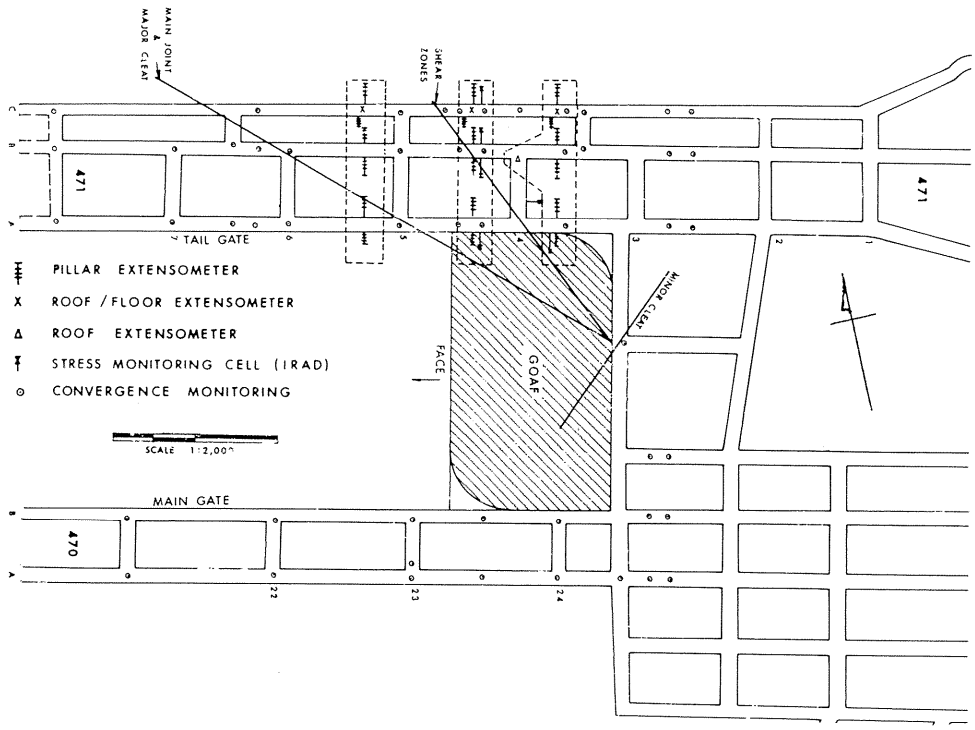

Figure 2 Layout of Longwall 1 showing location of instrumentation

with the then-existing technology.

Developments both within Australia and overseas, particularly the advent of the shield support for thick seams in the early 1970's, forced the Company to reconsider the introduction of the longwall system for mining at West Cliff Colliery. Extensive investigations were undertaken to correctly define the roof and floor conditions and design a face to meet the geological and mining environment of a new high production longwall installation.

The major cause of the failure of the longwalls between 1962 and 1965 was the inadequate capacity of the support system to cope with dead and live loads of the massive, high strength sandstone. Thus it was essential to undertake a full investigation of the processes governing the behaviour of such massive and strong roof rocks.

Physical model studies were undertaken by the Australian Coal Industry Research Laboratories Ltd. laboratory at Bellambi, NSW. A model was constructed to a scale of 22:1 to represent not only the massive sandstone roof and its fracture behaviour but also the effect of support load setting on convergence, fracture propagation between the face and the support canopy, effect of variation of the exposed span, support load density, setting and yield load etc. The results of model studies suggested a face support of 866 tonnes capacity.

Mathematical modelling using the boundary element method was undertaken to define the size of the gate roadway pillars. Based upon a three-heading panel layout and operational experience the size of the chain pillars was selected to be 35m x 55m and 20m x 80m (Figure 2). On the basis of mathematical and physical model studies, underground geological data analysis and observations in other mines working in the region, a 900 tonne four-leg chock-shield support system was selected. A 1250 tonne/hour conveyor system and 350 kW shearer-loader formed the backbone of the longwall operation. The face support equipment is the largest ever manufactured in the world for this purpose. Figure 3 shows the face installation.

Besides the face support system, other aspects of design of a longwall at West Cliff included ventilation and gas control, dust control and economic factors. Because of the high gas content of the Bulli Seam coal and the high rates of coal production envisaged due to high capital costs, very high gas emissions were expected. Intensive investigations were undertaken to control gas emission at the face and a full-scale gas drainage system was established. Investigations were undertaken to optimize the length of the longwall face and layout of the longwall both from face stability and gas emission point of view.

3 GATE ROADWAY SUPPORT SYSTEM AND ROADWAY CLASSIFICATION

Layout of the longwall face and stability of the face and gate roadways is very much influenced by the effect of major and minor joints and faults. Geological studies indicated that the shear zones which have been the cause of major outbursts at the mine could not be avoided in the area set aside for longwall operations. These shear zones strike at 300° - 310° (magnetic) and their width varies from a few centimetres to almost 100cm.

The dominant joint set in the area strikes 330°. The coal cleat directions are about 40° and 300°. Based upon general studies of roadway behaviour the best development direction was established as east-west (magnetic), the longwall face therefore being north-south. This meant that the face made an obtuse angle with the major joint set and also with the main cleat and shear zone directions; these factors have induced stability at the face.

The gate roads were driven and logged taking into consideration their behaviour after development. The roadway sections were classifed depending upon their condition, using an allocated point system considering the following:

i Presence of open or closed joints (2 sub-classes)
ii Presence or absence of shear zone
iii Presence or absence of roof collapse and extent of damage (4 sub-classes)

iv Presence or absence of ribside guttering (4 sub-classes)
v Roof rock lithology
vi Support density
vii Loading of supports as indicated by visual assessment
viii Extent of rib crush or collapse

Depending upon this system, gate roads were classified into four groups;

Group I	:	0 - 1	good
Group II	:	2 - 3	satisfactory
Group III	:	4 - 7	poor
Group IV	:	8 - 16	very poor

Gate roadways belonging to Groups III or IV were specially supported in advance of the face.

4 MONITORING PROGRAMMES

The purpose of the monitoring programme was to establish the accuracy of the design parameters and to collect data which may be useful for future longwall operations. It was required to establish the following :

i Is the support system adequate or if not, is it over- or under-designed?
ii Are the sizes of the chain pillars adequate?
iii Can the gate roadways withstand the loads generated by the longwall extraction and is the support system in the roadways adequate?

The longwall monitoring programme comprised

i Convergence monitoring in gate roadways
ii Monitoring load on face supports
iii Convergence monitoring of face supports
iv Monitoring load on chain pillars and on the longwall block ahead of the face using Irad stress gauges and CSIRO hollow inclusion (H.I.) stress cells
v Limited but intensive observations of absolute roof and floor movement in the first 250m of the retreat of the longwall face
vi Limited but intensive observations of displacements in roof, floor and pillars in the first 250m of face retreat using extensometers

Besides instrumentation, an extensive programme was undertaken to monitor the conditions of the roadways and roadway supports, fracturing behaviour of the goaf and roof behaviour at the longwall face.

5 INSTRUMENTATION

5.1 Roadway Convergence Measurement, 470 and 471 Panels

Total vertical roof-to-floor convergence was monitored on a routine basis in the gate roads at 186 locations in 470 and 471 Panels, as shown in Figure 2. The distances between fixed pins in the roof and floor rock were measured by steel tape or telescopic measuring stick to an accuracy of ± 1mm or better.

5.2 Relative Displacement Measurement, 471 Panel only

Vertical displacements in roof and floor rocks and lateral displacements in coal pillars and in the Longwall 1 coal block in 471 Panel were obtained using four-anchor wire extensometers installed normal to the ribside to depths up to 15m. Groups of measuring stations were located between cutthroughs 3 to 6 (that is, near the beginning of the longwall retreat) and cutthroughs 20 and 21, about two-thirds along the longwall block. Each comprised several lateral extensometers and a smaller number of vertical installations. Their locations are shown in Figure 2.

5.3 Measurement of Hydraulic Pressure in Longwall Face Supports

Each of the 89 chock-shield longwall face supports (Figure 3) was equipped with two pressure gauges, enabling the hydraulic pressures in the rear leg pair and front leg pair to be measured separately. Pressure was also monitored using pressure transducers and the information conveyed to the surface by telephone cable. Computer processing of this information presented some early problems.

5.4 Stress Change Measurement

Two methods of stress change monitoring were adopted: the Irad vibrating wire stress gauge and the CSIRO

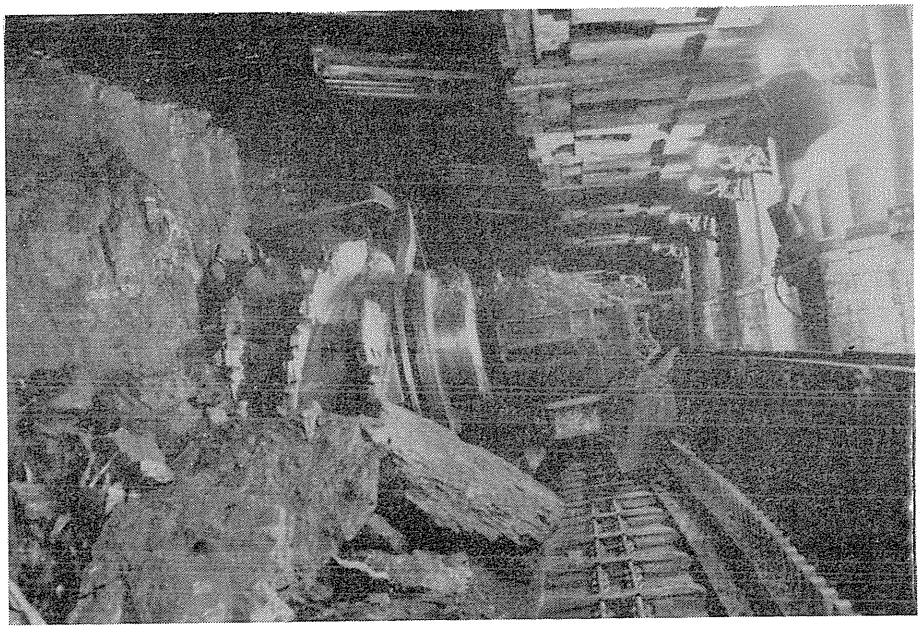

Figure 3 Longwall face installations

hollow inclusion (HI) stress cell, both of which are extensively described in the literature. These were located in the same sections as the extensometer installations.

The Irad gauge, being of relatively stiff construction (that is, incorporating a hollow steel body), would not be expected to suffer creep over the necessary time periods and consequently it is commonly used to monitor stress change. The Irad gauge enables only unidirectional stress changes to be monitored. To overcome this potential disadvantage the HI cell was chosen as a possible technique for monitoring three-dimensional stress changes. Twelve instruments were installed in the coal adjacent to Longwall 1.

6 RESULTS OF MONITORING PROGRAMME

6.1 Roadway Convergence

In 470 Panel, for A heading a typical plot of total convergence against distance between measuring point and face line is given in Figure 4. Convergence ranged from zero to 7mm ahead of the face line then increased to 35mm after the face line passed. An increase in convergence rate was evident when the face approached within 50m of the measuring station and began to decrease when the face was 100 - 170m behind the station. In B heading, Figure 4 shows an increase in convergence rate when the face line was 55m ahead of the measuring point; that is, of the same order as in A heading. Total convergence at the face line varied between 20mm and 76mm. This roadway caved 5 to 15m behind the face in general.

In 471 Panel A heading, an increase in the rate of convergence was observed when the face line approached within 70m of the measuring point. Caving generally occurred 2.5m behind the face line. In B heading, increased convergence was apparent 80m ahead of the face line, beginning to decrease about 20m after the face pass. Total convergence was very variable at the face pass, ranging between 3 and 281mm, depending upon the presence of shear zone or shale roof, but usually did not exceed 140mm. In C heading the convergence rate increased some 80m ahead of the face line and decreased 30m behind the face pass. The magnitude at the face pass varied from 1 to 138mm but was usually limited to 25mm.

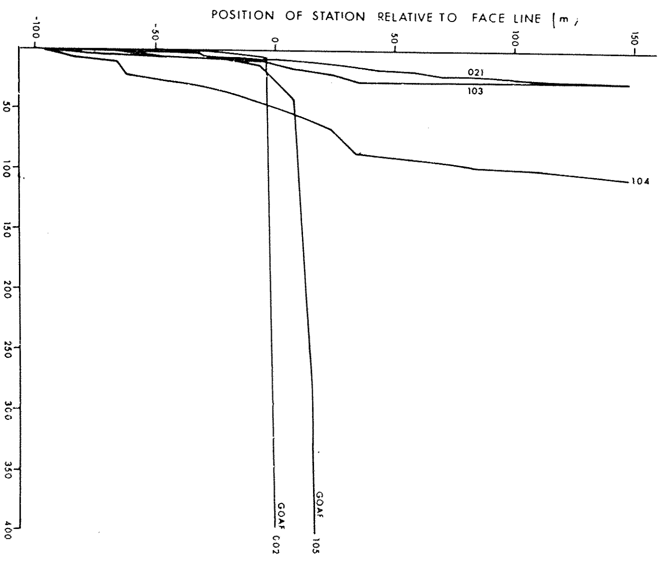

Figure 4 Convergence measurements in gate roadways

These observations showed that total convergence was least in the roadways furthest from the longwall block.

6.2 Relative Displacement and Strain

6.2.1 Lateral displacement and strain in coal

Typical total bay displacements and strains are given in Figure 5 which summarises results for two of the extensometer stations located between cutthroughs 4 and 5 in 471 Panel. From the figure it is seen that bay strains in the smaller pillar (80 x 20m) remote from the longwall block were consistently about one-third less than for the corresponding bays in the larger pillar (55 x 35m), adjacent to the longwall block, after the face had retreated 500m and therefore was no longer directly influencing pillar behaviour.

Most fracturing in the coal occurred in the first 2.5m. The strains beyond 2.5m are in the elastic range of the coal.

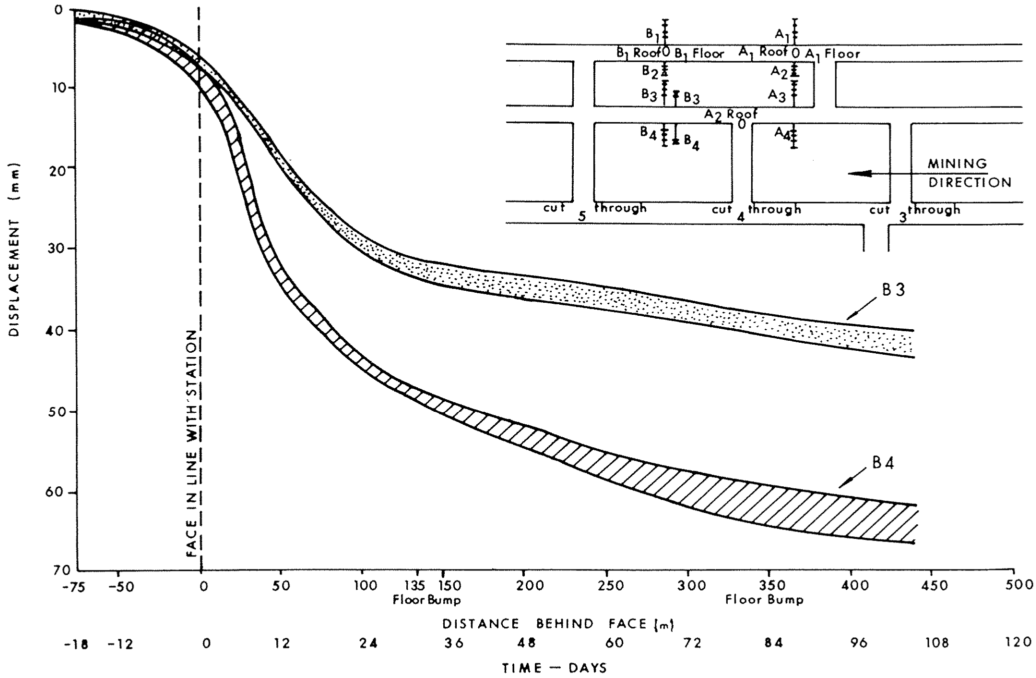

Figure 5 Extensometer measurements in chain pillars

6.2.2 Vertical displacement and strain

Total strain in the massive sandstone roof was generally of the order of 0.7×10^{-3}, while in the largely shale floor 7 to 10×10^{-3} were recorded at the same station. Strain of this magnitude was associated in many instances with floor heave in the roadways.

6.3 Face Support Hydraulic Pressure

Figure 6 shows face retreat distance up to 380m plotted against the range of hydraulic leg pressures measured along the line of chock supports. Peak pressure values occurred at intervals of face retreat ranging between 70 and 105m. Peaks were accompanied by a floor bump in the goaf immediately behind the line of chocks. This suggests that periodic failure occurred in the overlying strata as part of the normal caving mechanism of the sandstone roof followed by rebound and unloading.

6.4 Stress Changes

6.4.1 Vertical stress changes

The changes in vertical stress monitored with Irad gauges during the initial face retreat distance of 500m are shown in Figure 7 for the gauges located

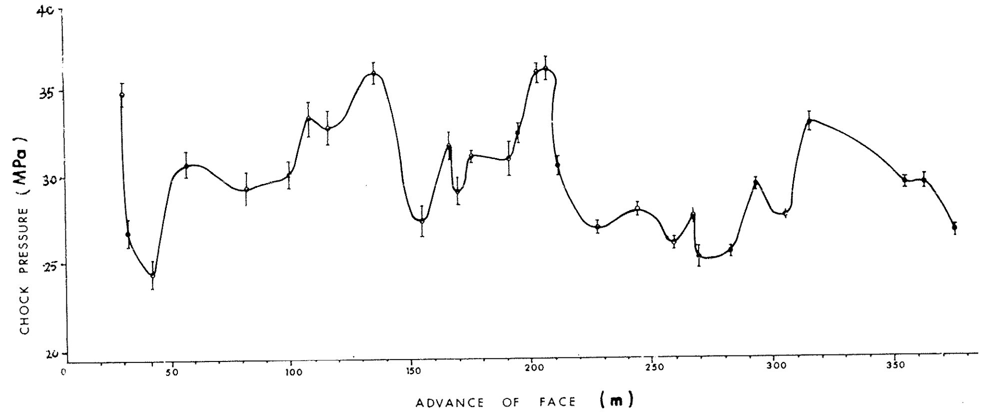

Figure 6 Development of loads on face supports

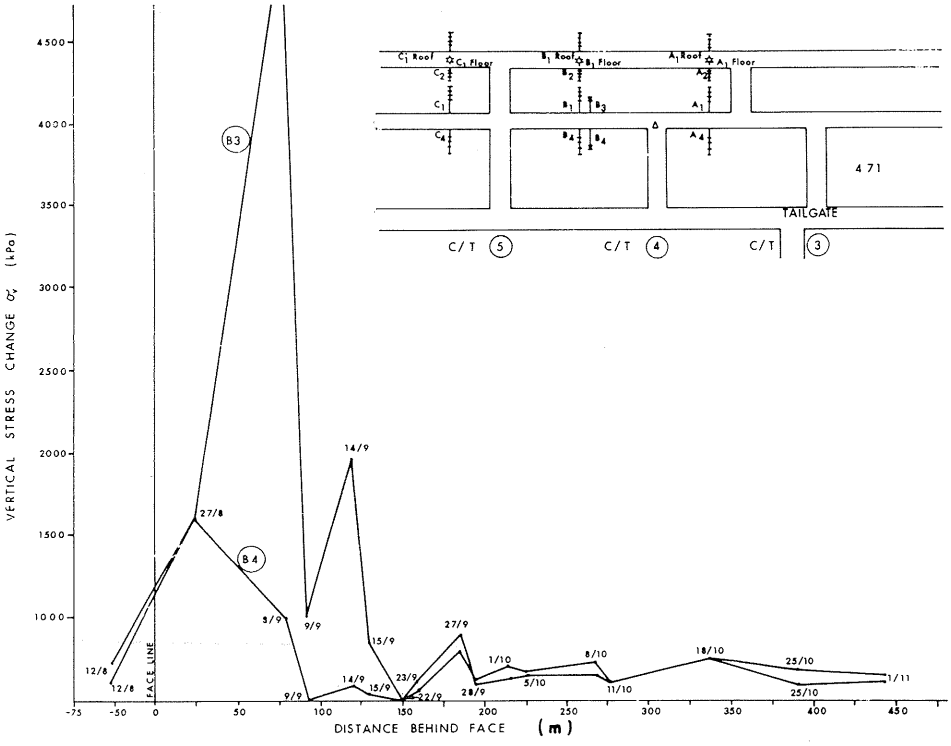

Figure 7 Vertical stress changes in chain pillars from Irad gauge masurements

between 4 and 5 cutthrough. A maximum vertical stress increase of approximately 2 MPa (compressive) was recorded in the 80 x 20m pillar 25m after the face pass. In the 55 x 35m pillar adjacent to the longwall block a peak vertical compressive stress increase of 5 MPa was recorded 80m after the face pass. It is notable from Figure 7 that the stress change vs. distance curve is irregular compared to those reported elsewhere and also in comparison with the displacement vs. distance curves at the same station (Figure 5). This may reflect low strength and low elastic modulus of the Bulli seam coal. Their effect on Irad gauge performance is uncertain.

An investigation of this question is in hand, together with the cross-axis sensitivity and temperature response of the Irad gauge.

6.4.2 Three-dimensional stress change

The stress change measured by the H.I. cells has proved difficult to assess. A typical strain change graph for a CSIRO H.I. cell is given in Figure 8.

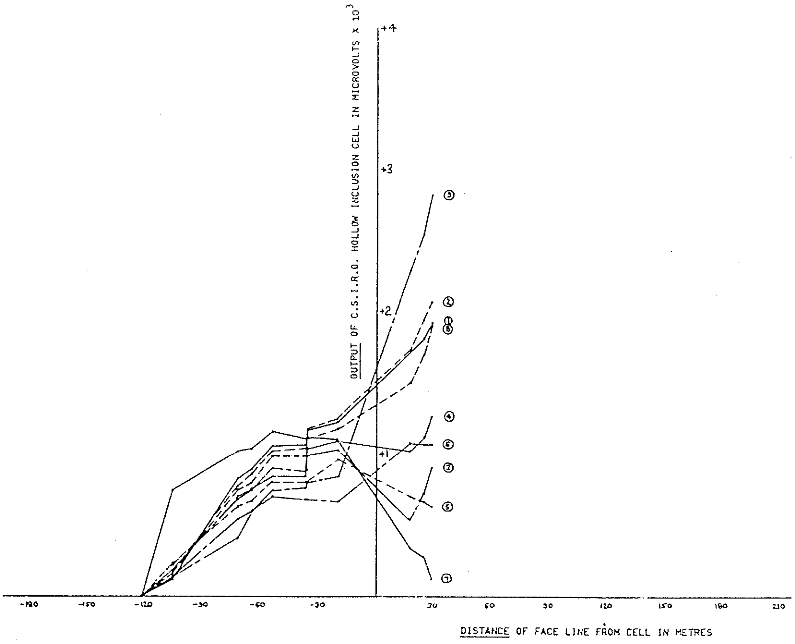

Figure 8 Typical strain change output from CSIRO H.I. cell. Stresses not determined

The results are ambiguous; current laboratory investigations of the creep characteristics of the epoxy material of the cell body and of the installation cement, the effect of heat annealing and temperature response generally indicate that the apparently irregular responses of the cell may be caused by one or all of these variables. It appears necessary to determine certain fundamental characteristics of the cell before the results can be used with confidence.

6.5 Behaviour of Gate Roadways

Observation of both the main gate and tail gate showed that the presence of jointing played a dominant role in the behaviour of the gate roadways. The main joint system, which made an angle of 30° to the face (Figure 2), protected the main gate. No floor heave occurred in the main gate ahead of the face. This joint system also controlled goaf behaviour such that the main gate road remained open behind the face, sometimes up to 20m but more usually 10-15m when mining under sandstone roof. The tail gate on the other hand experienced heavy loading conditions (Figure 9). Excessive floor heave occurred at the tail gate end of the face resulting at times in collapse of the tail gate end. Floor heave was seen up to 15m (usually 6-8m) ahead of the face in the tail gate roadway.

When under sandstone roof the transport roadway (A heading) on the main gate side of the longwall, located at 28m centres from the face end, underwent smaller convergence and deformation and suffered less damage generally than the tail gate B heading located 35m from the face end. Also the T-intersections on the tail gate side experienced heavier loading than on the main gate side. When shale was present in the roof, however, the behaviour of the roadways both on the tail and main gate sides did not differ very much; extensive guttering occurred in the roof associated with roadway collapse.

Four-way intersections were the worst affected. Pillar crush was heavier in the tail gate roadways then in the main gate. In general, the roadways in 471 Panel were more affected than in 470 Panel.

7 CONCLUSIONS

Monitoring of the longwall face and surrounding gate roadways showed that the behaviour of the goaf and condition of the roadways were greatly influenced by the orientation of the joint system in the roof strata. The obtuse angle which the main joint set in the massive sandstone made with the face resulted in forward loading of the tail gate roadways. The goaf break angle was asymmetrical, with the higher angle on the tail gate side. The main gate roadways were protected by virtue of the orientation of the joint system.

The sizes of pillars as determined by the mathematical models seems to have been adequate. Most of the fracturing in the B and C headings in the tail gate was limited to about 2.5m depth into the rib. Additional load on the C heading (which has ultimately become the main gate for the second longwall) was limited to 2 MPa. The support system, using W-straps with 5 point-anchor high-strength steel roof bolts, worked satisfactorily under these conditions when mining under a sandstone roof. However this

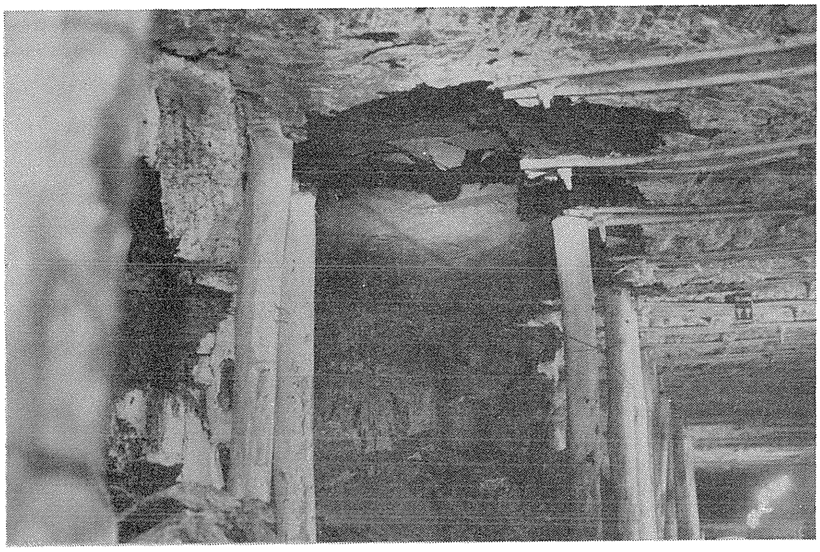

Figure 9 Conditions of roadway in B heading, 471 Panel, in sandstone area showing effect of high pressures

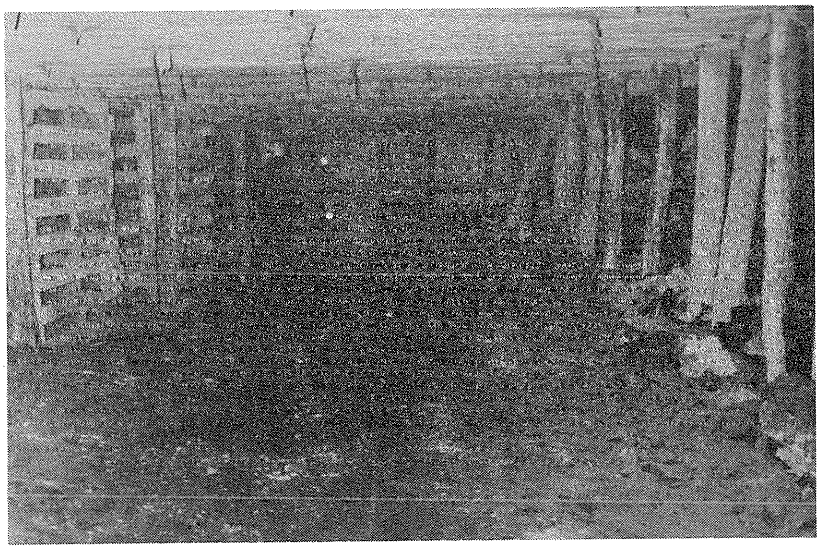

Figure 10 Conditions of roadway in A heading, 470 Panel, showing little deterioration

method of support was not adequate when shale was present in the roof.

The design of the face support system successfully met the static and dynamic load requirements imposed upon it by the massive sandstone roof. A face support system of lower capacity could have resulted in collapse of the face.

Convergence of gate roadways was mostly due to floor heave. Absolute vertical displacement of the sandstone roof was usually limited to about 10mm, at which point the roof rebounded upon collapse of the goaf and unloaded itself.

The failure of the massive sandstone in the goaf occurred cyclically, with the maximum cantilever length varying between 70 - 105m.

Stress change values measured using Irad stress gauges seemed to be lower than expected. The CSIRO H.I. cell in its present form does not seem to be suitable for monitoring changes in stress in coal.

8 ACKNOWLEDGEMENTS

The authors wish to thank the management of Kembla Coal & Coke Pty. Limited for providing the facilities to carry out these investigations and financial support to The University of New South Wales School of Mining Engineering. Thanks are due to the staff of Australian Coal Industry Research Laboratory, for instrumentation and measurements carried out in the project under a NERDDC grant. Also the support provided by Mr. R.J. Miller, Manager, West Cliff Colliery; Mr. P. Lamb, Senior Geologist, Coal Development Services Pty. Ltd.; Mr. P. Moxon, Colliery Surveyor, West Cliff Colliery; Mr. L. Griffiths, Consultant and Mr. A.M. Nizankowski, Research Student, The University of New South Wales, are all gratefully acknowledged.

The opinions expressed in this paper are those of the authors and are not necessarily those of the Company.