RATIONAL LONGWALL LAYOUT DESIGN BASED ON NUMERICAL STRESS ANALYSIS

By

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ABSTRACT

Rational design of layouts for longwall operations is greatly facilitated by use of an accurate and economical method for predicting stresses and displacements. The displacement discontinuity method (a form of boundary element method) is based on replacing actual excavation geometries by thin slits of the same plan area. This approach allows three-dimensional stress-analysis at a fraction of the cost of alternative methods. The analyses can be readily performed on economical microcomputers, putting the analysis capability within the reach of mine-site personnel.

This paper describes investigations associated with the German Creek Central Colliery, the first longwall operation in Queensland’s Bowen Basin.

The material properties used in the model, in particular the rock mass and goaf properties, have been ‘tuned’ to fit stress measurements obtained during extraction of the first and second longwall panels at the mine. The study shows that model predictions based on suitably scaled anisotropic rock strata properties are in good agreement with the stress measurements made to date. This model predicts average chain pillar loads significantly higher than predicted by models based on United Kingdom conditions.

A working chain pillar design method is proposed using average pillar loads predicted by the numerical model and ultimate strength characteristics given by Bieniawski’s formula.

INTRODUCTION

This paper describes geotechnical investigations associated with the German Creek Central Colliery. Being the first longwall operation in Queensland’s Bowen Basin it presented a number of unique design problems. Geotechnical experience from existing Australian longwall mines (in New South Wales) was of limited relevance because of regional geological differences. For this reason the mine operator, Capricorn Coal Management Pty. Ltd. (Capcoal), initiated a wide ranging programme of geotechnical investigations (Robertson and Ward, 1986). CSIRO has been involved in geomechanical investigations associated with the colliery from the very early planning stage.

Longwall production commenced in mid 1986 and record production levels have been achieved. Encouraged by this initial success, Capcoal is developing a second longwall mine, Southern Colliery, at the German Creek site. Construction started in late 1987, with the longwall scheduled to be operating by early 1990.

The chain pillars for the first longwall panel were dimensioned in the face of a high level of uncertainty. Accordingly, a number of design and analysis approaches were used (Robertson and Ward, 1986). This paper details refinements to the design methodology made possible by stress measurements obtained during extraction of the first and second panels (301 and 302).

A ‘working’ chain pillar design method is proposed based on average pillar loads predicted by the numerical model and ultimate strength characteristics given by Bieniawski’s formula.

GERMAN CREEK CENTRAL COLLIERY

The mine was planned at the outset as a retreat longwall operation (Figure 1). Panels were designed for strike retreat towards the main development headings with twin entry gate roads drives. The main headings are developed on 50 m centres with cut-throughs at 50 m centres. The longwall face length is 200 m. The chain pillars between the first two panels (301 and 302) are 25 m wide rib to rib with cut-throughs at 100 m centres. The gate roads are nominally 3.1 m wide.

The average depth of cover for the first inter-panel pillar is about 150 m. At the limit of the proposed mining plan the depth of cover will be about 350 m.

The German Creek seam thickness ranges from 1.8 m to 2.6 m. The extracted thickness is nominally 2.4 m. The coal is very weak by Australian standards, with an average laboratory uniaxial compressive strength of 9.6 MPa.

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ALTERNATIVE CHAIN PILLAR DESIGN APPROACHES

The design of longwall extraction layouts requires quantification of three interrelated factors:

- the abutment loads induced by extraction
- the response of the pillars to these loads
- the response of the gate roads to these loads

Clearly, the development of improved design procedures rests on our ability to quantify all three factors. The present paper focuses on investigations aimed at obtaining reliable measurements of pillar and abutment loading.

There are essentially two approaches to pillar design: stiff pillar or yielding pillar.

The stiff pillar approach is primarily used for 'permanent' pillars in room-and-pillar mines and for barrier pillars in total extraction mines. These pillars are designed with a high factor of safety (typically of the order 1.5 to 2) against total failure and assume that a pillar does not have any load carrying capacity after its strength has been exceeded.

Development pillars for longwall layouts should be designed using the stiff pillar approach. Optimisation of cut-through spacing and pillar width can be achieved by combining tributary and abutment loading in the pillar stress analysis. Considerable reduction in drivage distances can result from proper design of barrier and development pillars.

The yield pillar approach draws upon evidence that after coal falls it can have significant residual load carrying capacity. Pillars can yield progressively without jeopardizing overall mine stability. Because yield pillars are intended to 'fail', they can have a factor of safety of the order of 1.0. For a retreat operation a stable chain pillar is necessary for roadway protection during extraction of the first panel and until the second face is adjacent. As the inbye section of the tailgate is no longer required, with further retreat of the face the pillar can yield fully with a factor of safety less than 1.0.

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ULTIMATE PILLAR STRENGTH

The ultimate strength of a pillar is a function of three major factors: (i) its size or volume, (ii) its shape (i.e., width to height ratio), and (iii) the properties of the coal. Numerous equations have been proposed to calculate ultimate strength of pillars. Most are of the form:

\[ \sigma_p = \sigma_0 (A + B \frac{W}{H}) \] (1)

or

\[ \sigma_p = K \frac{W}{H} \] (2)

where \( \sigma_p \) is the pillar strength, \( \sigma_0 \) is the strength of a cubical pillar at the critical specimen size, \( K \) is a constant characteristic of a coal seam, \( A, B, \alpha \) and \( \beta \) are constants expressing the shape effect, \( w \) is the pillar width and \( h \) is the pillar height.

Bieniawski (1982) recommended equation (1) for United States conditions, with \( A = 0.64 \) and \( B = 0.36 \), i.e.,

\[ \sigma_p = \sigma_0 (0.64 + 0.36 \frac{W}{H}) \] (3)

The strength of the cubical pillar at critical size, \( \sigma_c \), is estimated from the laboratory uniaxial compressive strength, \( \sigma_0 \), using:

\[ \sigma_c = \frac{\sigma_0 D}{D_c}^{0.5} \] (4)

where \( D \) is the diameter or side dimension of the laboratory specimen.

and \( D_c \) is the critical size (36 inches or 0.9 m).

The most successful application of equation (2) has been in South Africa, where values of \( K, \alpha \) and \( \beta \) have been obtained from statistical analysis of pillar failures in room-and-pillar mines (Salomon and Munro, 1967). However, caution should be exercised in using this data for other geological environments.

GATERoad STABILITY

Wilson (1983) has developed a method for design of longwall inter-pillars that is essentially based on the concept of protecting the roadway from the stress increase resulting from extraction of the adjacent panel. The pillar itself would have a high factor of safety against ultimate failure. For design purposes Wilson assumes that the in situ UCS is one-fifth of the UCS derived from small specimens. This scaling factor is consistent with equation (4).

A set of input parameters has evolved from extensive experience with United Kingdom conditions. It should be emphasized that Wilson's method is not based on rigorous application of continuum mechanics theory.

NUMERICAL MODEL

DISPLACEMENT DISCONTINUITY METHOD

The stress distribution near the gate ends of the longwall face cannot be adequately modelled by two-dimensional methods. Although in principle three-dimensional finite element or general boundary element models can be used for such geometries, the high cost and effort required for such analyses is rarely justified. The displacement discontinuity method is a special version of the boundary element method for 'tabular' excavations, i.e. excavations that are characterized by their negligible thickness in comparison to their plan dimensions. The method assumes the excavations are replaced by thin slits of the same plan area. The relative movement between the roof and the floor is treated as a displacement discontinuity. This allows three-dimensional stress analysis at a fraction of the cost of alternative methods and allows analyses to be run on economical microcomputers.

The present work is based on the QSTIRO program MINLAY which analyses a rock mass consisting of parallel layers with distinct anisotropic elastic constants (Wardle, 1984). The unloaded upper surface of the system takes account of ground surface effects.

The extraction layout is represented by a mesh of equal-sized rectangular 'elements'. The mesh used in the current study consists of 14400 elements. The element size (3 m by 3 m) was chosen on the basis of modelling all roadways by one element width.
MATERIAL PROPERTIES

Substantial effort has been devoted to determination of meaningful input data for the numerical model. In general, anisotropic rock properties represent actual in situ behaviour more closely than isotropic models (Crouch, 1976; Wardle and Enever, 1983). A detailed study of the CSR Laelah No. 1 Colliery at South Blackwater (also in the Bowen Basin) by Wardle and McSabb (1985) found that anisotropic properties gave both stresses on the excavation plane and surface subsidence in good agreement with observations.

The relative values of the shear modulus \( G \) and the Young's moduli in the horizontal and vertical directions \( (E_h, E_v) \) have the greatest influence on subsidence profile shape whereas variations in the Poisson's ratios have a negligible influence. Steep subsidence profiles are obtained by using highly anisotropic properties, i.e. values of the shear modulus \( G \) that are much lower than the Young's moduli \((E_h, E_v)\). The ratio of Young's modulus \((E_h/E_v)\) is taken as 1.0 and \(G/E_v\) is 0.05. These values were used to model British subsidence data (Crouch, 1976) and fit subsidence and stress data from the Laelah No. 1 Colliery.

The in situ coal modulus based on stressmeter and total deformation rod extensometer results is 1.70 GPa.

Data from surface drilling into the roof strata above 301 panel at Central Colliery indicates a goaf bulking factor of 1.28 and modulus of 26 MPa. These values are consistent with data from Laelah No. 1 Colliery, for which the bulking factor is 1.25 and modulus is about 20 MPa (Wardle and Enever, 1983).

For simplicity of analysis, the rock mass is assumed to be homogeneous. Assuming that the parameters defined above are kept fixed (i.e. \(E_h/E_v\), \(G/E_v\) and goaf modulus), the material parameter that has the greatest influence on the stress distribution is the magnitude of the Young's modulus of the rock, conveniently specified by \(E_v\) (-E). Three values of \(E_v\) were used here, namely \(E_v = 0.44\) GPa, 1.2 GPa and 7.4 GPa. The intermediate value approaches the rock mass properties used for Laelah No. 1 Colliery (distinct properties were used for the major stratigraphic units). The lower value was determined by matching the predicted goaf load deficiency to that assumed by Wilson as representative of British conditions. Elastic moduli of this order of magnitude, although representative of gross in situ rock mass behaviour, are significantly less than obtained from laboratory tests.

COMPARISON WITH STRESSMETER RESULTS

Seven vibrating-wire stressmeters were installed in the coal seam and were orientated to monitor changes in vertical stress. Figure 2 shows the stressmeter locations. Five stressmeters (Nos.1-5) were installed in the chain pillar bounded by 301 panel, 302 panel and cut-throughs 11 and 12. Stressmeter No. 6 was installed in the 302 panel main block about 10 m from the 302 panel tailgate. Stressmeter No. 7 was installed in the 301 panel main block about 5 m from the 301 panel mingsate.

Fig. 2 - Locations of stressmeters.

The stressmeters were successfully monitored during extraction of 301 and 302 panels until the 302 panel face was adjacent. Further readings could not be obtained due to technical problems.

For simplicity of analysis the results from the 5 stressmeters in the chain pillar are combined to give the average pillar stress, calculated by assuming linear variation between stressmeter sites and taking ribside values equal to values for the nearest stressmeter. Total stresses are calculated by adding the average prior to commencement of panel extraction, estimated as 4.5 MPa based on tributary area theory and the nominal primitive vertical stress (3.6 MPa).

As one stressmeter (No. 2) consistently gave lower stresses than its neighbours during extraction of the first panel and fluctuated substantially during extraction of the second panel, a second set of average pillar stresses was calculated with this stressmeter excluded (given in brackets below). Using all stressmeters the average stress increase during first panel extraction was 4.8 MPa (5.4 MPa). When the face of the second panel was adjacent the stress increase was 8.1 MPa (7.7 MPa).

Figure 3 shows the average pillar stress as a function of face position. The numerical model results are for three values of rock modulus \((E_v = 0.44\) GPa, 1.2 GPa and 7.4 GPa). The model with \(E_v = 2.4\) GPa gives the best overall fit to the measurements.

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Figure 4 shows results for the stress increase in the panel main blocks as a function of face position. Again the numerical model results are for three values of rock modulus ($E_r = 0.44$ GPa, 1.2 GPa and 2.4 GPa). Although results from stresimeter No. 7 (in the 301 panel main block) are close to the model with $E_r = 0.44$ GPa, the results from stresimeter No. 6 (in the 302 panel main block) are substantially lower than given by any of the models.

Overall, the best estimate for $E_r$ is 2.4 GPa as this gives the best fit to the average of the results from the 5 stresimeters in the chain pillar.

The goaf load deficiency is the simplest qualitative measure of the load distribution and can also be used as a basis for detailed modelling of yield zone development etc. (Duncan, Fama and Wardle, 1987). The goaf load deficiency is calculated from the stress distribution at the mid-length of the panel. The predicted (on the basis of $E_r = 2.4$ GPa) goaf load deficiency is 150% greater than assumed by Wilson as representing British conditions. The corresponding increase (compared to Wilson's method) in average pillar stress after extraction of one panel is 52% and after extraction of panels on both sides is 98%. Clearly, the goaf load assumed by Wilson is not valid for German Creek.

**Working Chain Pillar Design Method**

In the absence of a more rigorous method of estimation, ultimate pillar strength is obtained using Bieniawski’s formula. The average UCS for 100 mm diameter specimens is 9.6 MPa, which by Equation (4) gives the cubical pillar strength parameter, $c_1$, as 3.2 MPa.

Equation (3) and the measured average pillar loads quoted above give a safety factor of 1.51 after first panel extraction and 1.11 when the face of the second panel was adjacent. Using the best modulus estimate ($E_r = 2.4$ GPa), the predicted safety factor approaches 0.64 with further retreat of the face.

Figure 5 synthesizes the data for pillar loading and ultimate pillar strength as a...
function of pillar width. The pillar loading is from displacement discontinuity analyses with $E_y = 2.4$ GPa for 150 m cover. The predictions of average pillar loads from the numerical model should be reliable because of the correspondence with the stress measurements obtained during extraction of 301 and 302 panels.

The satisfactory performance of the 301/302 panel gate roads suggests that the pillar strength characteristic, $c_1 = 3.2$, is a lower bound for ultimate pillar strength. Although additional rib spalling to 1.0 m occurred in the tailgate during second panel extraction, no secondary support was required, suggesting there is scope to increase $c_1$ in the light of experience gained during extraction of subsequent panels.

Using the concepts discussed above, pillar widths for subsequent panels at greater depths are readily determined. Figure 6 shows pillar width as a function of cover depth. For comparison, curves based on Wilson's design method are also shown. These are calculated using UCS = 3.2 MPa, friction angle = 30°, ribside support pressure = 0.1 MPa, however the design widths are primarily dependent on the assumed goaf loading. Two cases are considered: (i) goaf load for U.K. conditions and (ii) goaf load given by the numerical model that gives the best fit to the stressmeter data ($E_y = 2.4$ GPa).

The conventional Wilson (i.e., U.K. goaf conditions) and proposed designs are comparable for 150 m cover, but the pillar width for the proposed design method does not increase with cover depth as rapidly as for Wilson's design. The Wilson design method as modified to match the stressmeter data gives unrealistically wide pillars.

![Graph of pillar loading vs. pillar width](image)

**Fig. 5 - Pillar loading (150 m cover) and ultimate strength versus pillar width.**

**Fig. 4 - Measured and predicted stress ahead of face.**

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Fig. 6 - Proposed pillar design method: Width as function of cover depth (200 m face)

DISCUSSION AND CONCLUSIONS

Stress monitoring during extraction of the first and second longwall panels at Central Colliery has allowed the material parameters used in the numerical model to be refined. Model predictions with suitably scaled anisotropic rock strata properties are in good agreement with the stress measurements made to date. Predicted average chain pillar loads are significantly higher than for models based on United Kingdom conditions.

The 'working' chain pillar design method based on a combination of load predictions from the numerical model (with tuned material properties) and ultimate strengths from Bieniawski's formula can be used for the design of pillars for subsequent panels down dip.

The MINLAV displacement discontinuity program can aid the design of chain, barrier and development pillars for longwall mines in different geological conditions. Stress predictions obtained from MINLAV can also be used as boundary conditions in more detailed two-dimensional numerical models to study important aspects such as cgrack loading and convergence, and the behaviour of pre-driven recovery roadways during longwall approach.

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