ROADWAY AND PILLAR DESIGN CONSIDERATIONS FOR DEVELOPMENT AND LONGWALL EXTRACTION AT CENTRAL COLLIERY, QUEENSLAND

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ABSTRACT

Central Collie was developed to exploit large underground reserves in the German Creek Lease. It was planned from the outset as a retreating longwall operation and is the first longwall mine in Queensland. Longwall mining is not new to Australia but all previous experience has been confined to the coalfields of New South Wales. There was no precedent for longwall extraction in the markedly different geological setting of the Bowen Basin in Central Queensland. Various investigations were thus carried out during the initial development of Central Collie, prior to the installation of the longwall, to ascertain the geotechnical characteristics of the coal seam and the adjacent strata and to predict the performance of the heading and pillar design as mining proceeds to depth.

Laboratory tests showed the coal to be weak with an unconfined compressive strength of 9.6 MPa. Site inspection and geophysical correlation revealed a shear zone and extensive intrashearing shear planes in the seam. The latter were found to exert a controlling influence on rib reaction in the headings. This behaviour was predicted by finite element modelling of the development headings which indicated that mining induced stresses in the coal would be relieved by block movement of the ribs. A chain pillar width versus depth curve was established for yielding pillar design based on empirical and computational methods. This gave pillar widths from 25m to 40m over the proposed depth range of 150m to 300m. Finite element modelling of the 25m pillars chosen for the first gate roads confirmed the overall stability of the design but suggested that rib spall could be significant in the adjacent tailgate.

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INTRODUCTION

The German Creek Project in Central Queensland is managed by Capricorn Coal Management Pty. Ltd. and produces export coking coal from both open cut and underground mining operations. An underground mine, Central Collie, was commenced in January 1984 to exploit reserves in the German Creek seam. Longwall mining equipment was installed and a retreat face commenced operation in mid 1986. Figure 1 shows the location of the Collie.
The German Creek seam is weak in comparison to other Australian coals, such that design comparisons with other mines are not necessarily valid, especially in relation to heading and pillar stability. Furthermore, because the seam dips to the east at a grade of 1 in 10, mining conditions are expected to deteriorate as mining progresses down dip.

There had been no experience of working this seam by underground methods apart from a small underground operation conducted on the outcrop in 1978 to collect a bulk sample for marketing purposes. Although longwall mining has been practised in Australia for over 20 years, this experience is confined to coalfields in New South Wales. Central Colliery is therefore, the first Queensland mine to utilise the longwall method of underground mining. As this technology is critical to the long term future of the German Creek project and holds much promise for the future of the Queensland coal industry, it was clearly important to ensure that pillars and headings were designed adequately from the outset.

This paper outlines the features of the design of Central Colliery with particular regard to the design of mine openings using empirical methods, as well as computer modelling techniques.

**BACKGROUND SETTING**

The German Creek Lease contains some 100 million tonnes of open cut reserves (in situ) and in excess of 1,000 million tonnes of underground reserves (in situ).

Figure 2 illustrates the mine plan exhibiting the strip mining pits and the location of underground mining areas. Three principal seams are mined by open cut, these being in descending order:-

1. the Aquila seam (average 1.5m)
2. the Tieri seam (banded 1.0 to 2.2m seam)
3. the German Creek seam (average 2.5m)

The disposition of these and other minor seams in the Central Colliery area is shown in the cross section in Figure 3.

Central Colliery was located in the centre of the lease to mine the German Creek seam where the thickness ranged from 1.8 to 2.6m. In this area the Aquila seam is too thin for economic underground mining and the Tieri seam has been coked by an igneous sill.

A detailed rationale for the development of longwall mining at Central Colliery is presented in Galt & Jones (1985). Whilst initial feasibility studies were based on small-scale bord and pillar mines, a decision was made in 1982 to adopt longwall mining technology, due largely to economic reasons and the promising results achieved by new longwall installations in New South Wales and the USA.

Development mining commenced in January 1984 to install pit bottom facilities and initiate main entry development. Development entry design was based on the formation of square pillars with intersections at 50m centres, this design representing a fairly conservative approach for an unknown seam as far as mining conditions were concerned. In late 1984 an order was placed for equipment to operate a 200m longwall face of design capacity 1500 TPH (continuous). This face was installed in May 1986.

The face equipment comprised Doocy 4 leg 800 tonne chock shields and an all electric Eickhoff EDM 230-2L-2M D.E.R.D. shearer.

Figure 2 - German Creek Mine plan
Support loading conditions were anticipated by a consensus of current Australian practice and an appraisal of a physical model study (Richmond et al. 1986). Although the latter indicated that 600t supports should prove satisfactory, it was decided, in view of the lack of caving experience, to select a conservative capacity of 600t.

Figure 4 shows the basic panel layout planned for Central Colliery, with the 200 and 300 series longwall panels being laid out for strike retreat off the main dip development headings. The location of the main development headings, 2 East, was governed by the presence of a rider seam, the German Creek Upper seam, which split away from the main seam over the northern part of the area. As it was expected that roof conditions would be potentially less stable where the two seams coalesced, it was decided to locate the main development headings along this alignment, with allowance for roof cutting where necessary. This was considered preferable to locating the 300 district take-off lines in such uncertain conditions.

GEOLOGY

Central Colliery is located approximately in the central west of the Bowen Basin. The German Creek seam is the basal seam of the German Creek formation, which is of deltaic origin with fluviatile and marine phases present; it was deposited in the Late Permian period.

Typically, the overburden comprises moderately strong to strong thickly bedded fine to medium grained lithic sandstone, moderately strong thinly bedded siltstones and mudstones and occasional thin layers of very strong quartzose sandstone.

The immediate roof is usually an interbedded sequence of lithic sandstone, siltstone and mudstone, usually fine grained at the roof contact. The coal typically parts easily from the roof, although the contact is occasionally undulatory. Where the German Creek Upper seam split occurs, the interseam sediments are a thinly bedded sequence of moderately strong (20 to 35 MPa/m²) sandstones, siltstones and mudstones.

The floor is invariably lithic sandstone with up to 600mm of mudstone below the coal in places; it is moderately strong (30 to 45 MPa/m²) and resistant to weathering.

Downhole sonic logging was correlated with laboratory strength testing to develop profiles of geophysically-estimated rock strength (Miller, 1983). These profiles demonstrated the uniformity of the strength of the overburden and coal over the Central Colliery reserve area.

The coal in the German Creek seam is not only very weak by Australian standards but is also affected by the presence of large scale horizontal shearing. Regional tectonism has formed a zone of horizontal shearing some 100 to 150mm thick at about mid-seam height, occasionally dissipating over a wider zone of reduced coal strength.

Within the shear zone, the coal is either pulverised or contains an extremely high density of slickensided surfaces. The upper and lower contacts of the zone are slickensided or smooth intraformational shear planes, associated with thin bands of clay and mylonitised coal.

A number of claystone or mudstone bands are present in the seam, varying in thickness up to 50mm, but usually 15 to 20mm thick. Significantly, one or two of these bands appear both near the roof and floor and these often show evidence of horizontal shearing where they have been transformed to a "puggy" clay. These
bands provide planes of weakness and allow differential strain to take place in the rib section on exposure. In some cases polished shear surfaces persist in the coal even where the claytona band is absent.

The existing principal stress regime at Central Colliery is believed to be regionally oriented to the North East and at least partly of tectonic origin. A compressional horizontal stress from the NNE direction and estimated at approximately 2.5 times the vertical stress, was deduced using the hydraulic fracture technique in a surface drill hole down to 135m depth. (Enever & Woolorton, 1985).

Jointing in the overburden comprises a high angle conjugate set with a NE and NW trend. The coal cleat exhibits a similar trend but medium angle shear joints dipping in various directions are also present in the coal.

**COAL STRENGTH**

It was known from the open cut pit exposures that the Germantown seam was weak but only a limited amount of strength testing had been carried out prior to the construction of the portal. Once the true nature of the seam had been revealed in the headings, however, it became apparent that quantitative values would be required for use in pillar stability calculations and for mathematical modelling.

Confirmation of the strength subsequently came from unconfined compression tests on 63mm bore cores and on specimens cut from the rib by chain saw. The latter were tested either as 100mm cubes or cored to give cylindrical specimens. Obtaining suitably undisturbed samples from underground was made difficult by the closely spaced cleat and incipient omnidirectional shear surfaces that caused the blocks to fail apart as they were withdrawn from the rib. This was largely overcome by excising the cut faces in wire-reinforced plaster prior to moving the block.

The resulting values of unconfined compressive strength (UCS) ranged from 6.8 to 13.9 MN/m² with one anomalously low result of 2.7 MN/m². The distribution of UCS values is shown in Figure 5. There was no significant difference between the core core specimens or the cubes. Ignoring the one low value, the mean UCS was 9.6 ± 1.9 MN/m². This compares with other mean coal strengths from the Bowen Basin of 11.7 MN/m² for Toolebuck No.1 Colliery at South Blackwater (Wardle & Enever, 1983) and 14.5 MN/m² for Narromine Trial Colliery (O'Keefe, 1985). The mean values of Young's modulus and Poisson's ratio were 2.06 GN/m² and 0.47 respectively for Toolebuck and South Blackwater.

![Figure 5 - Unconfined compressive strength of Germantown seam](image)

A suite of triaxial tests was commissioned to provide parameters for finite element modelling. These were performed on both 63mm and 150mm diameter specimens, the former being cored from block samples cut from the rib, the latter from boreholes drilled on top of the exposed coal seam in the nearest open pit.

The results of the triaxial testing are shown in Figure 6. Least squares analysis of the three sets of data gave the following strength parameters:

- **50mm peak** $c = 3.88$ MN/m² $\phi = 36.9^\circ$
- **50mm residual** $c = 1.67$ MN/m² $\phi = 37.6^\circ$
- **150mm peak** $c = 0.42$ MN/m² $\phi = 35.5^\circ$

The angle of internal shearing resistance remained more or less the same, at 36° to 37°, but the cohesion, as would be expected, showed a major reduction with increasing sample size. The 150mm specimens tended to exhibit a small peak on the stress/strain plots, sufficient to give a 50% increase in cohesion when compared to the residual strength. This was not the case for the 150mm specimens which generally did not show any significant peaking.

The mean values for Young's modulus, calculated from the triaxial stress/strain plots, were 2.07 GN/m² for the 50mm diameter specimens (which compares with 2.05 GN/m² for the UCS tests) and 0.58 GN/m² for the 150mm diameter samples. The magnitude of this size-related reduction, plus the aforementioned lack of peak stress development suggests that the 150mm sample results are probably approaching a reasonable approximation of the in situ strength.

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STABILITY OF DEVELOPMENT HEADINGS

As the nature of the German Creek seam became apparent in the pre-production development headings concern arose as to the stability of the headings at depth, particularly with respect to the possible need for rib reinforcement. In addition there had always been concern about the potentially destabilizing effect of the German Creek Upper seam where it occurred in the immediate roof. The inter-seam distance varied from 0.5 to 4.0m, but was generally 1.0 to 2.0m, over the area of the proposed main development headings. The potential for roof instability was highlighted by roof falls in the early part of 1984 which were believed to have been exacerbated by delamination of the friable German Creek Upper seam at a height of 1.3 to 1.6m above the roof of the German Creek seam.

The initial design for development headings was 6.0 x 2.1m rectangular openings with 1.5m spot bolts at approximately 2.0m centres, with short cartridge resin anchorage, installed by machine-mounted bolting rigs. Following the roof falls mentioned above the support density was progressively increased to four 2.4m bolts with straps at 1.2m centres and long resin cartridges which gave virtual full encapsulation. This required 0.5m of roof strata to be cut to allow for installation of the longer bolts and to provide additional anchorage above the level of the German Creek Upper seam. To avoid delamination, bolts were installed at the face using hand held bolters (Gophers).

It was decided to use the finite element method (FEM) to predict ground response and heading performance of main developments headings under conditions of increasing depth of cover and, following the changes to roof support described above, to model roof and rib support and the effect of cutting the roof.

The first FEM study (Mikula, 1984) was unsuccessful, mainly because the program was unable to handle post failure conditions. Failed elements rounded, mathematically, to become voids, that is to offer no further resistance to continued strain. This meant that as computations were reiterated the system failed to converge, enlarging failed areas out of proportion. Final ground reaction, for practical purposes, had to be picked virtually as an arbitrary point in the reiteration sequence, which naturally negated any quantitative value in the exercise.

In addition, no guidance on roof and rib support levels was forthcoming from the study since, as it turned out, the program was incapable of modelling the effect of bolting.
A second FEM parametric study (McNabb & Wardle, 1986) provided a more plausible representation of ground response, the program in this case being suitably modified to reproduce post failure behavior such that convergence to equilibrium conditions resulted. Figure 7 shows the basic configuration; it was originally specified as 6.0 x 2.1m with the German Creek Upper seam at a height of 1.0m but a later decision was made to include 0.5m of roof cutting as this had been adopted as standard design for the development headings by that time. It is important that the full rib had to be considered as coal in order to avoid unacceptable delays in reconstructing the finite element mesh. The positions of all the elements were adjusted to ensure that the principal stress path was still orientated approximately to the axis of symmetry of the stress field and is inconsequential unless the bottom side of the rib is allowed to move.

The reference case was for 200m depth of cover with top and bottom shear planes and shear zone and the options modelled included variations to depth, stress ratio, presence and stiffness of shear planes, strength of coal and degree of support.

The conclusions of the study were as follows:

1. The presence of the upper and lower shear planes controls the behaviour of the rib while the shear zone only becomes significant if one or other of the shear planes is absent.
2. There is a marked increase in the amount of rib failure between 200m and 300m depth.
3. Variation in pre-mining horizontal to vertical stress ratio does not alter the amount of rib failure significantly.
4. The system is not particularly sensitive to variation in the residual strength of the coal, but reductions in modulus or strength on the shear planes produce a marked increase in rib to rib convergence.
5. Application of a uniform support pressure representing roof and rib bolts does not significantly influence the overall roadway stability.

Considering the conclusions of the FEM parametric study in more detail, the effect of the shear planes is shown in Figure 8. Without the shear planes the mining induced stresses are passed into the coal, overstressing the corners, causing the rib to bulge and create a yield zone to a depth of 1.1m. The presence of the shear planes, however, allows stress relief by horizontal slippage such that corner stresses concentrations are not developed in the coal and the rib moves as an intact block. The shear zone lies more or less at the axis of symmetry of the stress field and is inconsequential unless the bottom side of the rib is allowed to move.

Figure 9 illustrates the effect of increasing depth of cover for the reference case. The German Creek Upper seam remains intact at 100m but begins yielding between 100m and 200m depth. There is no rib yield in the German Creek seam at 100m, about 0.2m of yield at 200m, but a dramatic increase to about 1.5m is apparent at 300m.

The influence of the pre-mining horizontal to vertical stress ratio is not particularly significant. The 2:1 ratio used in the model was presumed to represent typical Australian coalfield conditions. Reducing this to 1:1 extends the depth of yield in the rib from 0.2m to 0.4m at 200m and from 1.5m to about 1.9m at 300m. The improved stability under horizontal stress conditions is presumably due to a rectangular opening with a horizontal long axis being marginally better disposed towards a horizontal major principal stress than a vertical one. Hydraulic fracturing investigations (Snever & Woltorton, 1965) subsequently showed the stress ratio to be about 2.5:1 with the major principal stress alignment between 0° and 065°. Most of the development headings are oblique to this direction. Development to date has not revealed any significant variation in stress effects with direction of driveage.

The influence of the residual strength was likewise shown to have an insignificant effect. For strain softening behaviour the coal strength (OCS) used for the reference case was 9 MN/m² peak and 4 MN/m² residual with a modulus of 2 GN/m², these values being derived from the UCS and small diameter triaxial tests. Reducing the residual strength by 50% gave only a marginal increase in the depth of yield in the rib and in the German Creek Upper seam. Reducing the modulus to 1 GN/m², on the other hand, reduced the rib yield to negligible proportions at 200m, but, almost doubled the amount of rib to rib convergence. This is highly significant as the
Figure 8 - Effect of shear planes (after McNabb & Wardle, 1986)

Figure 9 - Effect of increasing depth (after McNabb & Wardle, 1986)

larger diameter triaxial tests, which were not available at the time of the study, suggest that the modulus could be as low as 0.6 GN/m².

Roof and rib support was simulated by applying a uniform pressure to the opening. Overall, the results were marginal implying that bolts or dowels would have little effect in reducing deformation. In practice, however, they would be beneficial in maintaining the integrity of roof or rib under such conditions.

Underground observations made at the time of the study, when the headings were approaching a depth of 100m tended to confirm the predictions. Yielding of the rib was not apparent and there was not even any spalling except in localised areas where adverse joint surfaces were present. Moreover the upper rib at least was seen to be displaced by block movement on the upper shear plane and the shear zone. Thus the urgency of providing rib support in development headings was removed, at least until the depth of mining passed the 200m mark. On the other hand, the results confirmed the need to continue to ensure a good anchorage above the level of the German Creek Upper seam, cutting the roof where necessary.

The stability of development pillars

The main development entries were designed on a 50m grid, this being a fairly conservative approach for an unknown seam as far as underground mining conditions were concerned. Again, as the nature of the German Creek seam was revealed in the pre-development headings it was thought necessary to seek confirmation of the suitability of the pillar dimensions for mining to greater depths.

First calculations of pillar stability were made using the empirical formula of Obert & Duvall (1967), where:

Pillar Strength = \( k_c (0.7b + 0.22w') \), where

\( w' \) = reduced pillar width (41m)
\( b \) = pillar height (2.4m)
\( k_c \) = in situ coal strength

Pillar dimensions, for computational purposes, were taken as 45m square less 2m depth of spalling, that is 43m square. The value of \( k_c \) was determined from the UCS using the relationship given by Hustrulid & Swanson (1977).
The factor of safety against failure was taken as the ratio of pillar strength to pillar load, the latter being the gross tributary load.

Tributary Load = \((w+\delta b)^2\) \(1\), where

- **W** = pillar width (45m)
- **\delta b** = heading width (5.0m)
- **\gamma** = overburden density (2.50 t/m³)
- **h** = depth of cover

The mean value of \(k\), calculated from the UCS tests was 1.7, and using this as the in situ strength value gave the results shown on Figure 10, that is, indicated the factor of safety was less than two for depths in excess of 200m. This was clearly not acceptable.

Similar stability calculations were made using an alternative empirical formula (Salamon & Munro, 1967) where:

\[
Pillar\ \text{Strength} = k_0^0.046\ \text{UCS}^{0.66}
\]

However, this gave even less satisfactory factors of safety, as shown on Figure 10.

It is known that the Salamon & Munro formula is highly conservative for squat pillars, that is, pillars with a high width/height ratio, as is the case at Central Colliery. Hence, there was considerable doubt on the applicability of the results obtained. Strictly speaking, \(k_0\) in the Salamon & Munro formula is not the in situ strength but a mathematical curve-fitting constant, which was found to be 7.18 for the original analysis of South African conditions. Accordingly, the calculations were repeated with \(k_0 = 7.18\), the results being shown on Figure 10.

**Figure 10 - Stability of development pillars**

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The original Salamon & Munro formula has been modified (Salamon & Wagner, 1985) to take account of seam strength variations as well as the width/height ratio. The revised formula for pillar strength is:

\[
\text{Pillar Strength} = k \sigma^a \beta^b \left[ \frac{K}{R_0} - 1 \right]^{\gamma},
\]

where

- \( a = -0.0667 \)
- \( b = 0.5933 \)
- \( R = 5 \)
- \( \beta = 2.5 \)
- \( V \) = pillar volume
- \( K \) = width/height ratio

\( K, a, b \) and \( \beta \) are constants as shown. Although this formula was not available at the time of the Cental Colliery development pillar study, results of retrospective calculations for \( k = 1.26 \text{ MN/m}^2 \) have been plotted on Figure 10 for comparison.

It can be seen from Figure 10 that the stability curves mentioned so far exhibit a common trend of rapid reduction of stability with increasing depth of cover, at least over the range concerned. This occurs because the calculated pillar strength remains constant regardless of cover load.

An alternative empirical approach is to consider the effect of stress distribution across the pillar whereby peak stress in the rib has a lateral confining, or triaxial, effect on the core. Thus with increasing cover load increased peak stresses exert a greater confining effect and enhance the carrying capacity of the core.

The method used for Central Colliery is that given by Wilson (1962 p.90) for the conditions of allowable load on rectangular pillars, that is the load at the limit of roadway stability. This condition is given when the average stress across the core becomes equal to the peak stress at yield,

\[
\bar{\sigma} = kq + \sigma_0,
\]

where

- \( k \) = triaxial stress factor
- \( q \) = cover load
- \( \sigma_0 \) = in situ stress

The results of the confined core analysis are also shown on Figure 10. The curves have a much flatter slope reflecting the increase in strength as well as tributary load as depth of cover is increased. The original calculations, based on an angle of internal shearing resistance of \( \phi = 30^\circ \) (as indicated from preliminary results of triaxial testing) showed that the factor of safety would not fall much below 2.0 for the planned extraction depths.

Later calculations using \( \phi = 35^\circ \) showed the factors of safety to be significantly higher. Thus it was concluded that there would not be any need to revise the development pillar dimensions for mining at greater depth.

**CHAIN PILLAR DESIGN**

Longwall panels at Central Colliery were designed for strike retreat towards the main downdip development headings with twin entry gateroad drives. The final design layout for the first panels, longwall 501 to 504, is depicted on Figure 4. It was originally proposed to develop gateroads at 50m centres, as per the development headings. Pillar lengths were increased to reduce the number of cut-throughs as it was generally accepted that most of the instability in gateroads during face retreat is centred on the intersections. In addition there was pressure to reduce the width of the chain pillars for the following reasons:

1. to allow faster development,
2. to avoid unnecessary sterilisation of reserves and
3. to reduce the potential for stress transfer onto the chain pillar for the next longwall should full goaf consolidation not be realised.

Accordingly steps were taken to determine the optimum chain pillar width. Firstly an analysis was made using the rigid pillar formula of Salamon & Munro (1967) with \( k = 7.18 \). Two conditions of tributary loading were considered as shown in Figure 11. A factor of safety of 1.5 was used for the first panel extraction where the pillars had to protect the adjacent roadway, with a factor of safety of 1.0 for the second panel extraction whereby the chain pillar became redundant. This produced almost coincidental values for the two respective conditions over the depth range considered so that the results could be combined to give the single design line as shown on Figure 12.

An angle of draw over the goaf of \( \phi = 25^\circ \) was used in the calculation of tributary loads. This was based on published figures for North American mines, which appear to provide the closest approximation to Bowen Basin conditions, and is considered to represent a reasonably conservative value for computational purposes. A physical model, constructed to scale from layers of sand/cement mixes as part of the longwall support investigations (Richmond et al, 1984), developed breaklines at 10" to 15" during periodic caving with a 20" break over the goaf from the rear abutment. This indicated that relatively steep caving could be expected.
Secondly, a confined core analysis over the same depth range was made following the method of Wilson (1962) for yield in seams only but modified to take account of the 25° angle of draw. The results are shown on Figure 12 for comparison. As was the case with the development pillars, the confined core method gives narrower pillars at depth than the rigid pillar method. The curves intercept at a depth of about 185m, which corresponds to a width of about 27.5m. The inflexion in the lines is due to the critical width for caving of 21.5m; otherwise the Wilson results would be linear. All pillar stability calculations were on the basis of horizontal seams.

Tributary loading is the most significant variable in empirical calculations, and, in the absence of subsidence monitoring, is also the least reliable. Thus in order to improve the reliability of this parameter the distribution of loads on the chain pillars was calculated by computer modelling using the MINLAV displacement-discontinuity program (Wardle, 1974). The displacement-discontinuity method does not require a finite element mesh as it is based on replacing the mined areas by thin slits of the same plane area and hence can be used to model any configuration of mine layout relatively inexpensively. Accordingly, the redistribution of load on chain pillars of various widths was computed for both one and two panel extraction.

For the displacement-discontinuity analysis an anisotropic rock mass model was developed based on the results of a previous study at Daleham No. 1 Colliery, South Blackwater, which provided a close correlation between observed and predicted conditions (Wardle & McNabb, 1985). Ratios of horizontal to vertical Young's Modulus of 1.0 and shear modulus to vertical Young's Modulus of 0.05 were used, together with a Poisson's ratio of 0.25. A Young's modulus of 1.0 MN/m² was used for the German Creek seam and 12.0 MN/m² for the roof rock, the latter being scaled by a factor of 0.2 in the model. A modulus of 20.0 MN/m² was used for the roof as recorded from large scale in situ plate bearing tests conducted at Daleham No.1 Colliery. (Wardle & Enever, 1983).

The results of the MINLAV analysis (Wardle & McNabb, 1986) for various pillar widths at a depth of 150m is shown in Figure 13, together with the pillar strength as per the Salmon & Munro formula. The intersections of the strength and stress curves give the critical pillar dimension, that is, where the factor of safety is 1.0. For example, for the first panel extraction any pillar less than 17m wide might not support the adjacent gateroad, whereas any pillar greater than 27m might not yield following the extraction of the second panel.
As the average pillar stress predicted by MINLAY for the anisotropic rock model was linear with respect to the overburden pressure the curves could be scaled for other depths. This allowed the critical pillar widths to be plotted on the pillar width versus depth diagram to give the relationship shown on Figure 14.

Here the MINLAY results indicate a design band within which any pillar width would satisfy the presumed requirements of a yield pillar. However, to allow for an element of safety with respect to the first panel extraction, pillar width design in the upper quarter of the band is obviously recommended.
Interestingly, the entirely independent Wilson method in this instance gave values within the MINLAY band, generally within the upper half. It was thus proposed that chain pillars should be designed on the basis of widths approximately midway between the Wilson value and the upper MINLAY value as shown on Figure 14. The gateroads for longwall 301 were thus based on 30 x 100m centres, giving 24m to 25m wide chain pillars. This is equivalent to a factor of safety of 1.53 for 301 panel extraction and 0.81 for the remnant pillar following the extraction of 302 panel, according to the relationship on Figure 15.

The empirical analysis described above provided guidelines for the determination of pillar size with respect to overall stability. This was followed up by an FEM study (Mardle & McNabb, 1986) of the chosen pillar dimensions for the first maingate drive in order to predict the behaviour of the chain pillar in greater detail. The stress distribution across the pillar predicted by the FEM is shown on Figure 15. (This was actually for a 24m pillar as the original finite element mesh was on the basis of 6m wide roadways at 30m centres).

The average vertical stresses for the pillar worked out a little higher than those predicted from the MINLAY analyses, however, the pillar strength according to the FEM is significantly higher than that derived from the Salamon & Munro formula. The net result is shown on Figure 16 where it can be seen that the average factor of safety for 301 maingate pillar during 301 panel extraction is 1.81, reducing to 1.46 (for the remnant pillar) following the extraction of 302 panel.

This would seem to confirm the application of empirical analysis at Central Colliery, at least in the determination of a suitable width for the more critical first panel extraction case. On the other hand it also would suggest that a fully yielding pillar will not be achieved without reducing the width to the point where the adjacent roadway would be at too great a risk following first panel extraction.

**GATERoad STABILITY**

An intrinsic part of the chain pillar study was the behaviour of the gateroads when subjected to the additional mining induced stresses arising during longwall panel extraction. If the yield zone developed in the chain pillar is excessive it will lead to an increase in roof span and potential for roof failure.

First estimates of yield zones were given by the Wilson method which showed a variation from 2.0m at a depth of 120m to 2.6m at 300m, with a value of 2.1m for longwall 301 maingate. These are estimated depths of yield on the extraction side of the chain pillar.

A more detailed assessment was provided by the FEM (Mardle & McNabb, 1986). Referring to Figure 16 there is no measurable yield on development but about 1.3m of yield for longwall 302 tailgate side and probably as much as 2.0m for longwall 301 maingate side following the extraction of 301 longwall panel. The maingate side agrees more or less with the Wilson estimate but this may be fortuitous and does not necessarily validate the Wilson predictions for greater depth.

The ratio of development and first panel extraction peak stresses on the tailgate side would suggest, simplistically, that the effect of the stress transfer across the pillar would be equivalent to the conditions applying to a virgin heading at a depth of 334m. The previous detailed FEM study of heading stability (McNabb & Mardle, 1985) indicated a yield zone of approximately 1.5m at a depth of 300m.

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All the evidence thus suggested that a yield zone would be developed to a depth of 1.3 to 1.5m into the pillar on longwall 302 tailgate side following the extraction of longwall 301 panel. What this actually meant in practical terms was unclear but rib spall to that depth would be undesirable and could prejudice roof stability. The conclusion was that some form of rib support might be needed in longwall 302 tailgate prior to the extraction of 301 longwall panel to reduce the propensity for spalling.

**MONITORING**

Regardless of the degree of sophistication of the method of analysis, the validity of the result is still governed to a large extent by the accuracy of the input parameters. Some of these, such as triaxial strength and elasticity, were measured in the laboratory, but others, such as coal loading, could only be estimated. The latter are obviously open to question but even the results of laboratory tests, measured with considerable precision, may not necessarily reflect in situ values.

With FEM, however, once the mesh has been set up, it is relatively easy to re-run the program with a change to one or more parameters. Thus parameters can be varied, without the need for determination of absolute values, until the computer predictions match the observed conditions. Provided that suitable instrumentation is installed and monitored the computer model can be “tuned” to reproduce a similar response. This should then allow the model to be used with greater confidence for predicting stability under the more adverse conditions expected at depth.

Predicted convergences from FEM heading stability investigation (McNabb & Wardle, 1986) are shown in Figures 17 and 18. Routine roof to floor convergence monitoring in the main development intersections suggest typical vertical closure values of the order of 1 to 10mm, with local cases up to 20mm. Most of the convergence is attributable to floor heave. Specific monitoring sites in longwall 301 tailgate, at a depth of 15m, gave an average of 7mm roof to floor convergence. This is of the right order of magnitude for the predicted values but is not sensitive enough for reliable correlation at such low orders of convergence. For example the monitoring recorded only those movements occurring behind and away from the face whereas the FEM predicted total displacement including movements ahead of the face, which could represent 50% of the total.

Rib to rib convergence is much more significant and, moreover, is primarily due to block sliding so that the amount of horizontal elastic closure ahead of the face will be...
negligible in comparison to the total. Hence measured post mining values should provide a more sensitive correlation to the predicted values. Two monitoring sites were installed during 301 maingate drive and these gave rib to rib convergence of 43mm and 67mm respectively. The latter value was recorded from an area where the coal was more closely jointed than normal.

![Graph showing predicted rib to floor convergence](image1.png)

**Figure 17 - Predicted roof to floor convergence**  
(after McNabb & Wardle, 1986)

The results of convergence monitoring in longwall 301 maingate drive have been plotted on Figures 17 and 18 for comparison with the predictions. For the rib to rib measurements the 67mm case showed an equal degree of slippage on both top and bottom shear planes, for the 43mm case only the top rib block movement could be recorded although the indications were that both blocks had moved. At face value the results correlate well with the top shear plane case, however, this may only be fortuitous, particularly since it is suspected that the value of Young’s modulus for the coal used in the numerical modelling was too high. Nevertheless initial results were encouraging and suggested that fine tuning of the model should be possible with a few more data points.

At the time of writing, the development of longwall 301 panel had been completed but longwall extraction had not started, so there has been no opportunity to monitor the gate road and chain pillar performance. However it is proposed to install stress cells, multipoint roof and rib extensometers and convergence stations to monitor pillar loading, yield zones and displacements around the headings as the face line passes by.

![Graph showing predicted rib to rib convergence](image2.png)

**Figure 18 - Predicted rib to rib convergence**  
(after McNabb & Wardle, 1986)

**CONCLUSIONS**

The German Creek seam was shown to be very weak by normal Australian coal standards and to contain intraformational shear planes that dictate the reaction of the ribs during main heading development.

Application of finite element methods to heading stability analysis demonstrated the significance of the shear planes in relieving mining induced stress and predicted that rib instability would not become manifest until the depth of cover exceeded 200m.

The numerical modelling also produced quantitative predictions of convergence for comparison with measured values which should allow recalibration of the model and hence enhance the reliability of future predictions as mining proceeds to depth.

Traditional rigid pillar formulae showed that main development pillar stability would decrease rapidly with depth and, in conjunction with equivalent in situ strength values would result in unacceptable factors of safety for development pillars below 200m depth. However,