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Application of the Brittle Failure Criterion to the Design of Roof Support in the Soft Rocks of Coal Mines

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APPLICATION OF THE BRITTLE FAILURE CRITERION TO THE DESIGN OF ROOF SUPPORT IN THE SOFT ROCKS OF COAL MINES

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ABSTRACT: The bilinear brittle failure criterion that utilizes the unconfined compressive strength (UCS) and a spalling limit of 3.4, together with a tensile strength cut-off can be used to model the height of failure above coal mine roadways. Transverse isotropy can be incorporated into a continuum analysis by using a Young's Modulus/Independent shear modulus ratio of 15. To predict the height of failure, the two key variables are the roof strength index (UCS/pre-mining vertical stress) and the horizontal to vertical stress ratio. By factoring in the stress concentrations that occur about a longwall excavation the criterion can be used to predict heights of failure on initial roadway development and in the maingate. A support design based on dead-weight suspension of the failed mass can be utilised.

NOTATION

CI - Competence Index	UCS/ σ_v
E	Young Modulus
Fa	Resolves principal horizontal stresses into stress acting across the roadway (Figure 10)
Fm	Concentration of horizontal stress at the maingate corner resolved across the roadway (Figure 12)
G	Independent shear modulus
Hmax	Maximum height of failure
K	$\sigma h/\sigma v$
Ki	K ratio before mining
L	Bolt anchorage length
Mv	Concentration of vertical stress at the maingate corner
RSI - Roof Strength Index	UCS/ σ_{vi}
UCS	Unconfined compressive strength
$\sigma_1,\sigma_2,$	Major and minor principal horizontal stresses
σh	Horizontal stress applied to roadway in 2 dimensional model
σ_v	Vertical stress applied to roadway in 2 dimensional model
σ_{vi}	Initial vertical stress

INTRODUCTION

In the longwall mining method, two sets of roadways (gateroads) up to 3 km – 4 km long are driven in coal to define a block of coal that may be between 150 m and 410 m wide. The longwall is installed between the gateroads and can retreat at rates of up to 25 m/d. The maintenance of roof stability in these roadways is fundamental to safe and productive mining. The attainment of such stability is needed to ensure the business requirement for fast development rates. In the modern Australian mining context, maintaining a commitment to zero harm needs to simultaneously accommodate a roadway development rate of at least 50 m/d.

Most coal roof support design is either precedent/practice or empirically based using a rock mass classification scheme (Mark and Molinda, 2005; Colwell and Frith, 2009). A relatively dense bolting pattern is typically installed at the face and this is supplemented by longer tendons installed prior to the retreat of the longwall face. Despite these dense patterns, there are still notable roof failures which, in addition to the evident impacts on safety, also result in the longwall face being stood for in excess of several weeks. Many mines suffer from a development constraint whereby the next coal block is not ready and either the longwall face equipment needs to be stood for weeks or months, or blocks are cut short and coal reserves lost. A significant component of this constraint is the time spent installing roof support.

At the consistent mining rates required, the reactionary/remedial components of the observational method are not appropriate and there is a need for robust estimates of likely roof support densities early

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in the planning stage, as well as in operations. There is a need for some simple tools that can provide such designs. Recent industry-funded research has sought to provide a more analytical approach to roof support design, one that is readily incorporated into mine planning and operations (Seedsman, Gorden and Aziz, 2009). The research identified three basic failure modes for coal mine roadways. First, there is the de-lamination of thinly-bedded strata close to the mining face. Secondly there is the onset of compressive failure of the roof if the induced stresses exceed the strength of the rock. Thirdly, there are situations whereby the vertical stresses are well in excess of the horizontal stresses (especially in coal) and the roof may fail in tension.

This paper discusses the compressive and tensile failure mechanisms and how a simple continuum numerical code can be used to assess different stress and failure conditions and to develop simple design charts.

ENGINEERING GEOLOGY AND GROUND CONTROL

The lithologies encountered in the roof of coal mine roadways include claystones, siltstones, sandstones and conglomerates. In thick coal seams, the roof may consist of coal. The UCS can range between less than 5 MPa to in excess of 100 MPa. Friction angles are in the range of 25^{o} - 30^{o}. The density of coal measure rocks is in the range of 2.4 to 2.5 t/m^3, and for coal it may be in the range of 1.2 to 1.6 t/m^3.

Rock masses in coal measures are dominated by the presence of ubiquitous bedding partings, and typically two joint sets aligned orthogonally. Joint spacing is typically equal to the spacing of bedding discontinuities (Hobbs, 1967) such that a valid model is a rock mass of cubes with dimensions from centimetres to metres. By virtue of the depositional environment and the subsequent diagenesis, the rocks are quartz silicates.

There is relatively little information published on the size of roof falls. In one shallow mine, the maximum height of roof falls is reported to be within 400 mm of the width of the roadway (4.8 m-5.2 m) and roughly triangular (Payne, 2007). There is a widely-accepted folklore that the maximum height of falls in deeper mines (up to 500 m) is no more than the width of the roadways. Underground operations collect a large amount of data on the vertical displacement in the roof based on multipoint extensometers installed close to the face at the roadway centreline (Figure 1). The point of zero displacement is referred to the height of fracturing or softening. As defined, the height of fracturing can be used as a proxy for possible roof fall height. One published database (Strata Engineering, 2001) has the maximum height of fracturing of 5 m above 5.2 m wide roadways and 6.5 m above 8.4 m wide roadways (Figure 2). O'Grady and Fuller (1992) reported a height of softening in excess of 7.4 m above a 4.9 m roadway aligned poorly to the horizontal stress field at 420 m depth.

Roof bolting may need to be specified against several different potential collapse mechanisms (Seedsman, Gorden and Aziz, 2009). Given the presence of bedding partings, there may be a need to reinforce the partings so that a thicker beam is created. If compressive or tensile failure is induced, the ground control strategy may need to be based on the suspension of the immediate roof from more stable ground. There is also a need to install skin restraint to control small scat.

The need to install long tendon support in addition to the standard roof bolts and mesh panels has been related to the ratio of the UCS of the rock to the far-field horizontal stress (Gordon and Tembo, 2005). To determine this ratio, the roof strength index (RSI) has been defined as the ratio of the UCS to the pre-mining vertical stress: this has the same formulation as the competence index used in tunneling (Muirwood, 1972). At the Kestrel Mine a value of 3.5 was found to indicate the need for long tendons during longwall retreat and 2.8 to indicate the need for long tendons during development.

ROCK STRENGTH CRITERION

The initial development of brittle criterion (Martin, Kaiser and McCreath, 1991) referenced the sandstones in the Donkin Morien tunnels accessing the coal seams of the Sydney Basin Nova Scotia. The 1991 version invoked a cohesion value of UCS/6 and a friction angle of zero degree (0^{o}). The criterion was further extended to include the spalling limit (Kaiser, *et al.*, 2000), with the implication that the lower limit for the spalling limit is the Mogi line of 3.4. The 3.4 limit applies to silicate rocks (Mogi, 1966) and it is noted the value is equivalent to a friction angle of 33^{o}. A recent back analysis of the Donkin Morien tunnels (Seedsman, 2009) determined a spalling limit less than five, and a transverse anisotropy ratio of less than 20 for the case of a modulus ratio of two. A preliminary assessment for the spalling limit of coal may be higher – 10. Because of the scale of the excavation and the nature of the rock mass, the failure criterion must also include an absolutely zero tensile strength. The available option is a Mohr Coulomb criterion with a tensile strength cut-off.

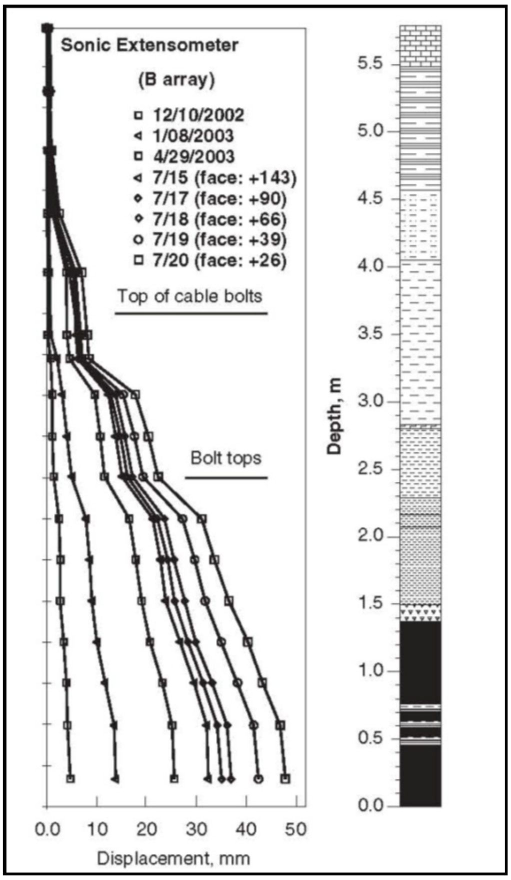

Figure 1 - Vertical movements above a 4.9 m wide roadway showing a height of fracturing of 4.9 m above a 4.9 m wide roadway (from Mark, *et al.,* 2007)

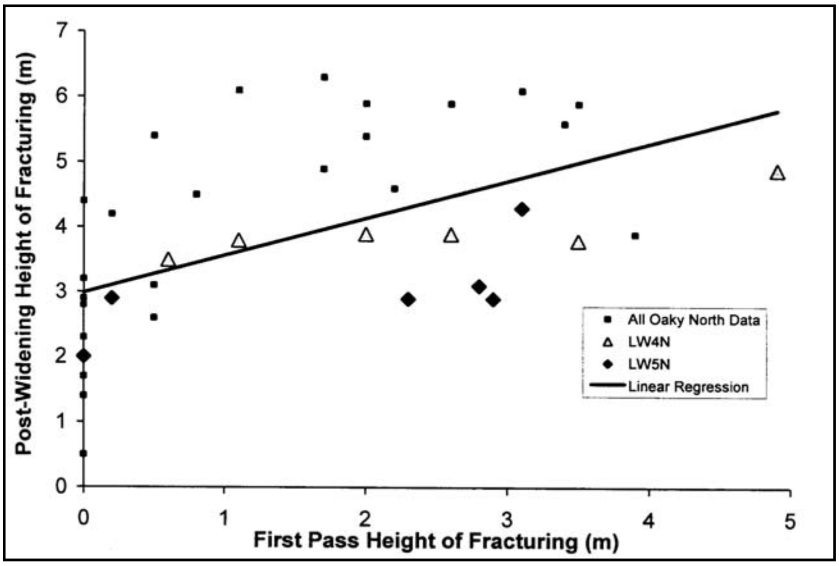

Figure 2 - Compilation of height of fracturing (softening) for shallow longwalls (from Strata Engineering 2001)

Coal measure rocks are transversely isotropic, both by way of the preferred orientation of clay particles within the finer grained lithology and by bedding textures and bedding partings. This can be modelled in a continuum assumption by invoking transverse isotropy, although at the cost of only being able to conduct elastic analyses. Published Young's Modulus/Independent Shear Modulus (E/G) values for sedimentary rocks are in the order of 1-3 (Gerrard, Davis and Wardle, 1972), but these are for intact rock. For a bedded rock mass, higher values are possible and as will be discussed, values of between 15 and 20 are indicated from back analysis, with the simplifying assumption that the two Youngs Modulus values are equal. The inclusion of transverse isotropy is more important than the ability to model yielding.

In summary, an appropriate strength criterion for soft rocks in coal measures in the vicinity of an excavation shown in Figure 3 is:

- An E1/E2 ratio of unity,
- An E/G ratio of 15 - 20 is needed,
- SL of 3.4 is appropriate for stone,
- Tensile strength = 0.0,
- Cohesion = UCS/6,
- Friction angle of 0.0.

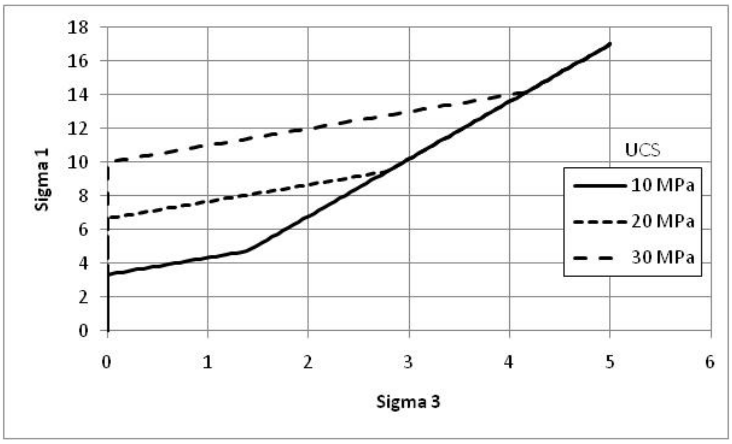

Figure 3 - Adopted failure criterion for coal measure rocks

FAILURE ZONES ABOUT A TYPICAL ROADWAY

The failure criterion discussed above is soundly based on recent applied research into the behaviour of excavations. It also offers great advantages in terms of developing simple design charts for the extent of rock failure. Failure develops when the minor stress is tensile, or when the deviatoric stress is greater than UCS/3, and then the maximum extent is given by a ratio of the principal stresses. This allows elastic stress analyses with only one material variable - the UCS. Simple boundary element codes (Examine 2D) can be used for specific openings, and the following discussion is based on a 5 m wide by 2.5 m roadway.

A further simplification is possible if the UCS is normalized to one of the principal stress components - in this case to the vertical stress. In the following, the normalization is referred to as the Competence Index (CI), which is defined as:

$$CI = UCS/\sigma_v \quad (1)$$

Where: CI = Competence Index, and
σ_v = vertical stress applied to the opening.

Figure 4 shows the distribution of failure zones for different CI values for the case of K = 1.3 and E/G=15, where K= σ_v/σ_h, and σ_h = horizontal stress applied across the opening.

For the case examined, the failure zones are continuous across the roadway only for CI values less than 6.0. For higher CI values, the failure zones are localised near the sides of the excavation, and this may be the basis for the stress guttering reported from underground roadways. The failure heights cannot exceed those given by a spalling limit of 3.4 shown by the dashed line in Figure 4. The shape of the failure zones defined by the spalling limit is parabolic.

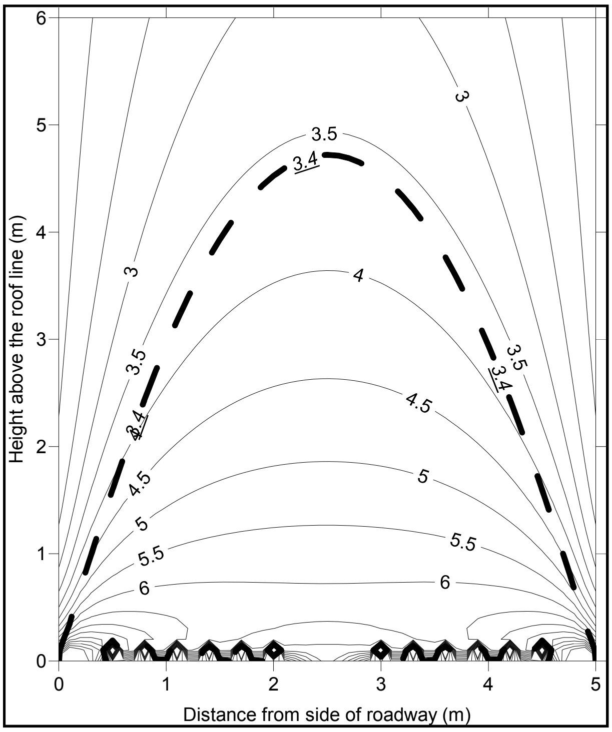

Figure 4 - Extent of failure zones above a 5 m by 2.5 m roadway for different CI values (thin lines) and for a spalling limit of 3.4 (heavy dashes) for K=1.3 and E/G=15

With a series of analyses it is possible to generate design charts that give the height of failure at the centreline of the roadway for different K ratios and CI values. Different charts can be produced for different E/G ratios as shown in Figures 5 and 6. Because the analyses are elastic, it is possible to normalize for different widths if the aspect ratio of the roadway is the same. For different aspect ratios, there is a need to conduct separate analyses.

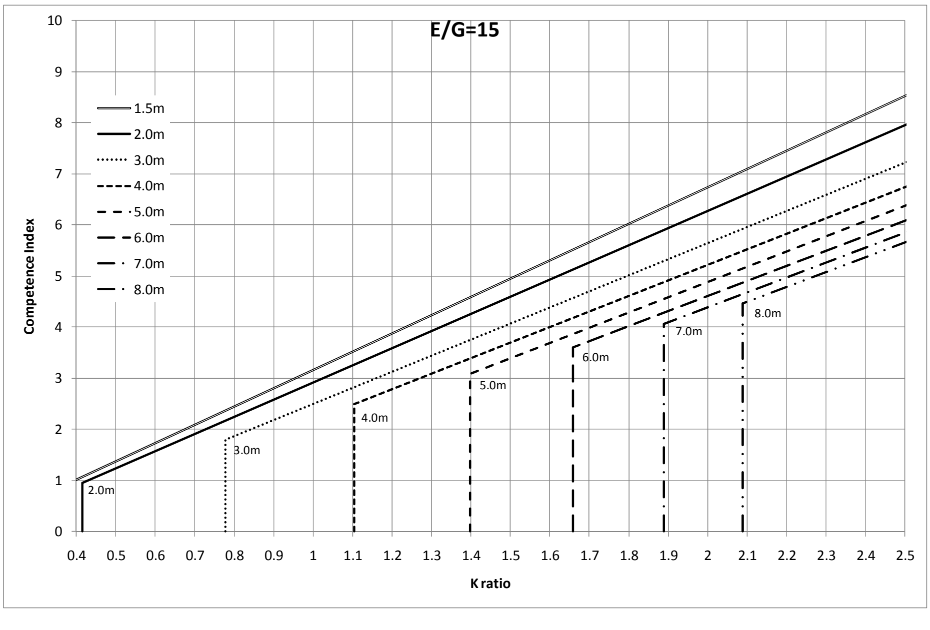

Figure 5 - The height of failure above a 5 m by 2.5 m roadway as a function of the K ratio and the competence index (E/G=15)

Close inspection of the Figures 5, 6, and 7 reveals that there is little difference in the heights of failure controlled by the CI for the various E/G ratios, but a large difference for the heights controlled by the spalling limit in Figure 7. The appropriate value to use for design can be determined by back analysis.

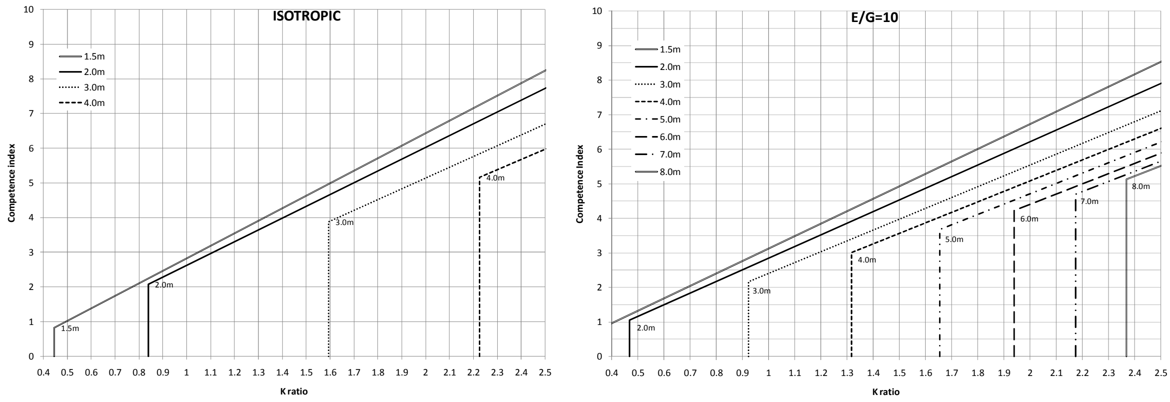

Figure 6 - The height of failure above a 5 m by 2.5 m roadway as a function of the K ratio and the competence index (a: Isotropic, b: E/G=10)

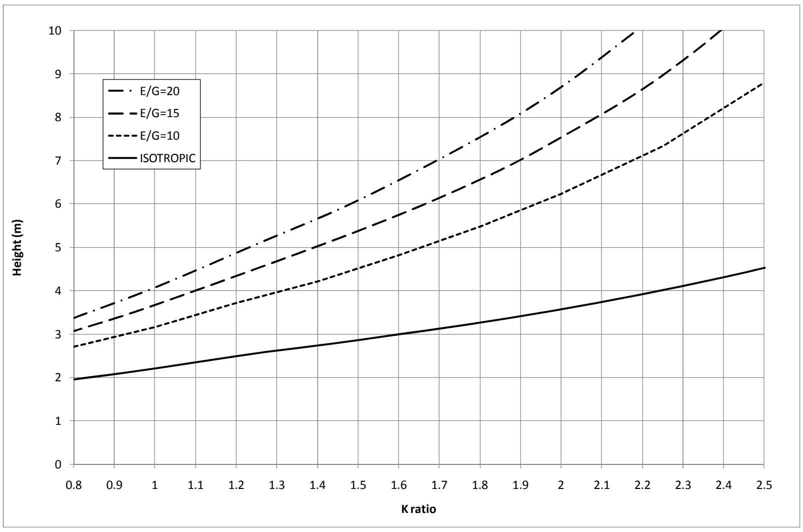

Figure 7 - Sensitivity of failure height to the E/G ratio when the spalling limit controls the onset of failure

For low values of K, there is no compressive failure in the roof as the immediate roof stresses may be tensile. Recognising that the joints are vertical, the zone of the tensile horizontal stress component may indicate where vertical shear along joints may develop as shown in Figure 8. The size of the tensile zone does not change with increasing stress magnitudes, while the magnitude of the tensile stresses within the zone does increase. The tensile zone vanishes for K ratios in excess of approximately 0.55 shown in Figure 9; at a K ratio of 0.2, the zone is about 1 m high and 4 m wide.

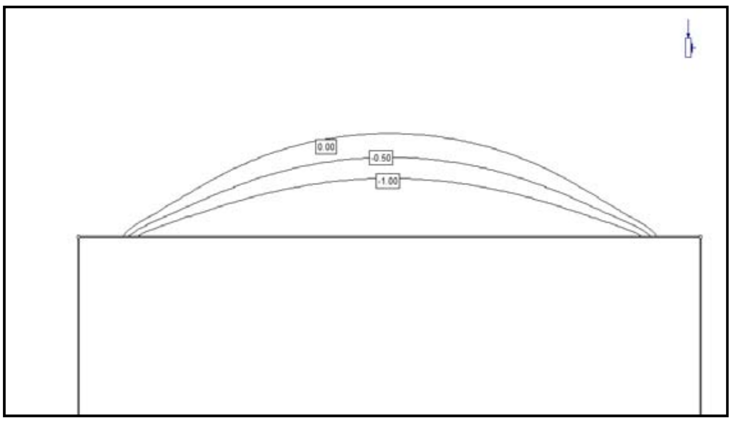

Figure 8 - Distribution of tensile horizontal stresses for K=0.3 (5 m by 2.5 m opening)

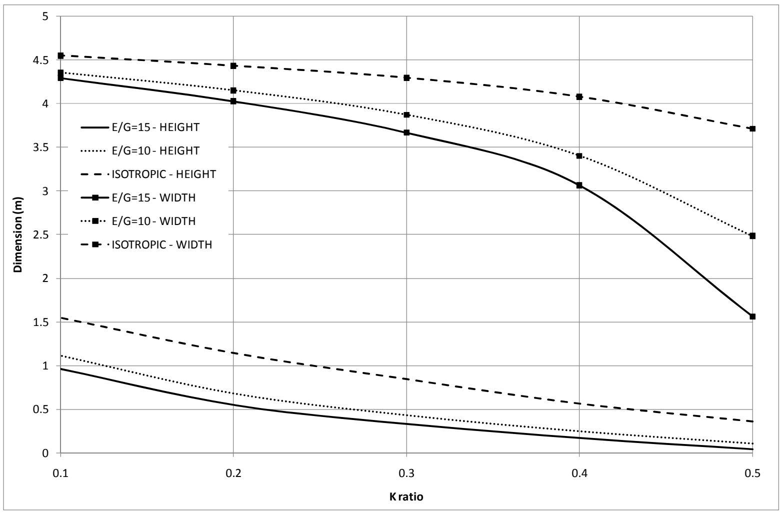

Figure 9 - Dimensions of the zone of tensile horizontal stresses decrease as the K ratio increases

HORIZONTAL AND VERTICAL STRESSES AROUND A RETREATING LONGWALL

In-situ stresses

Underground coal mining in Australia is conducted at depths as low as 50 m and in excess of 500 m and in excess of 1000 m in other countries. For gently dipping coal seams, the vertical stress in stone can be considered to be a principal stress with its magnitude related directly to the depth of cover. The *in situ* stress field in stone is characterised by Ki ratios that are close to unity in some European coal fields to in excess of 2 in some Australian coal fields (Mark and Gadde, 2010). For the coal seams themselves, the situation is different, with Ki values less than 0.2 measured (Seedsman, 2004). At depths, in excess of about 250 m, the direction of the major principal horizontal stress may be related to the latest tectonic events. Different alignments can be expected at shallower depths or near major fault structures.

Development roadways

Mine roadways are rarely aligned parallel to one of the principal horizontal stress directions. The two dimensional failure model discussed above requires that the component acting normal to the roadway centreline is used (Figure 10). The Fa factor is applied to the ratio of the *in situ* major horizontal/vertical stresses to determine the K ratio acting across the roadway (K=Ki * Fa). Immediately after the roadway is formed, the deformations in the immediate roof (bedding dilations, brittle fracturing) cause the roof to "soften" and the horizontal stress field is redirected higher into the roof (Mark, *et al.*, 2007).

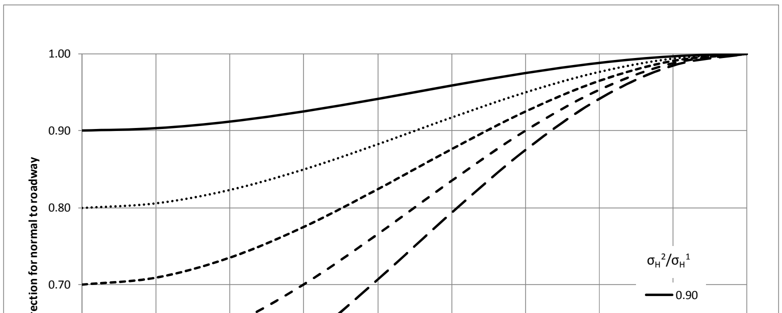

Figure 10 - Stress factor for roadway alignment (Fa)

Maingate

At the maingate corner at Position a, in Figure 11, during the retreat of the longwall, there are large changes in the stress regime. Based on numerous pillar stress instrumentations, longwall pillar design methods indicate an approximate doubling of the vertical stress at the maingate (Mv=2) (Mark, 1991; Colwell, 1996). Gale and Matthews (1992) not only reported similar data on the vertical stresses, but also presented a summary of the concentration of horizontal stresses. At heights about 5 m-10 m above the roof line there can be up to a doubling of the horizontal stress magnitudes if the roadway is aligned 45-55^o to the principal stress direction as shown in Figure 12a.

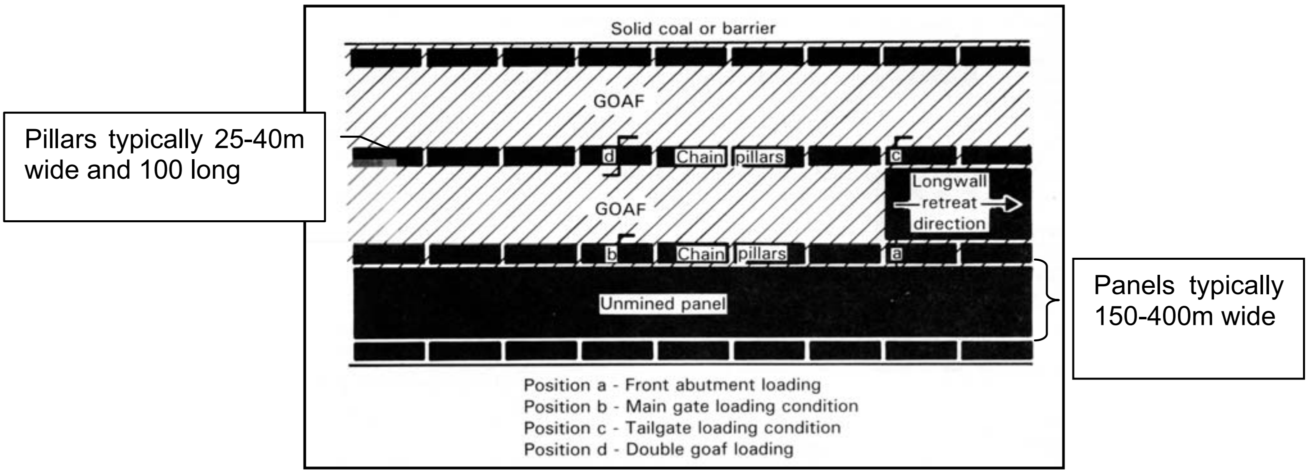

Figure 11 - Nomenclature of a longwall

The presentation in Figure 12a is not directly applicable to the two dimensional model presented above as it requires the magnitude of the stress acting across the roadway. It is known that during longwall retreat extensive fracturing develops, not only above the extracted seam, but also into the floor. The frequent reports of gas emanating from lower seams (up to 40 m below) indicate that fracturing extends at least that far. On this basis, it is valid to assume that plane strain conditions exist at the coal seam level and hence simple two dimensional models can be used to assess the stress field. By selecting an offset distance from an excavation that gave similar stress concentration patterns as shown in Figure 12a, a nomogram has been produced to allow the determination of the component of the induced horizontal stresses acting normal to the roadway centerline as a function of the orientation of the roadway and the ratio of the principal horizontal stresses, see Figure 12b. Using this figure, the K ratio, resolved across the roadway is given by the following relationship:

$$K = Ki\ Fm/Mv \qquad (2)$$

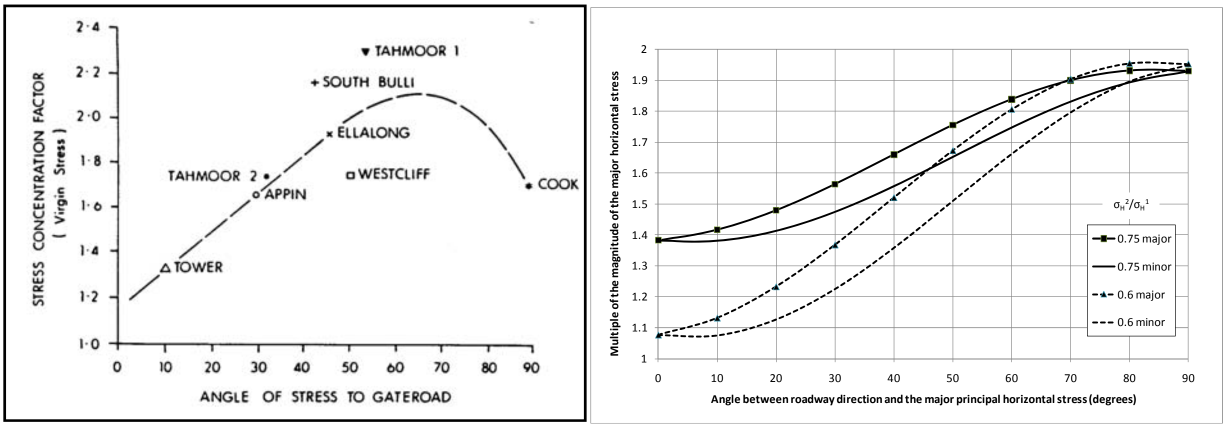

Figure 12 - Concentration of horizontal stresses at the maingate corner (a:-principal stress, b: stress factor for component normal to roadway centreline - Fm)

Tailgate

At the tailgate, Position c, in Figure 11, there are further increases in the vertical stress, but a reduction in the horizontal stresses due to proximity of the adjacent previously extracted longwall. The end result is a reduction in the K ratio. Tailgate stresses have not been extensively monitored so quantification of this impact is not yet possible.

THE E/G RATIO IN COAL MEASURE ROCKS

A number of published case studies have been investigated to determine the appropriateness of the design charts and to obtain an appropriate estimate for the E/G ratio. Table 1 indicates that an E/G ratio of 15 provides a good estimate of the height of fracturing above rectangular roadways. The Donkin case study has been reanalysed using a modulus ratio of unity and a E/G ratio value between 15 and 20 was obtained.

Table 1 - Summary of case studies for failure height

Mine	Crinum – 4.8 m roads	Emerald - 4.9 m roads	Tahmoor - 4.9 m roads
Reference	Payne (2007), Gordon and Tembo (2005)	Mark, *et al.* (2007)	O'Grady and Fuller (1992), Keillich, *et al.* (2006), Brown and Windsor (1990)
Depth	120-200 m	Approximately 200 m	420 m
Vertical stress	3-5 MPa	6 MPa measured	10.5 MPa
UCS	8-10 MPa	17.7-36.4 MPa stone, average 27 MPa	48 MPa
RSI = CI (development)	6 – 2	3.5-7.3, average 4.5	4.6
$\sigma_H^1/\sigma_v, \sigma_H^2/\sigma_v$, σ_H^2/σ_H^1	1.6,1.2 0.75	Measured stresses	1.92, 1.29 0.67
Angle	45	Measured stresses	77^o
Fa (Figure 10)	0.87	Measured stresses	0.98
K ratio	=1.6*0.87=1.4	1.67 – Stage 1	=1.92*0.98 = 1.88
Estimated height	5.0 m for rocks with CI =2 if E/G=15, or 5.7 if E/G=20	3.3 m if E/G=15	5 m if E/G=15
MV		2	2
CI(maingate)		2.25	2.3
Fm (Figure 12b)		Measured stresses	1.9
K ratio		1.31 – Stage 2, 1.25 – Stage 3	1.88*1.9/2=1.79
Estimated height		4.2 m if E/G=15, 5.3 m if E/G=20 4.9 m if E/G=15, 5.0 m if E/G=20	6.6 m if E/G=15, or 7.5 m if E/G=20
Comment	Maximum heights on development within 0.3 m of 4.8 m	3.3 m in stage 1, 4.2 m in stage 2, and 4.9 m in stage 3	7.2 m above a 4.9 m roadway

APPLICATION TO PREVENTING ROOF COLLAPSE IN LONGWALL GATEROADS

Roof bolting in Australian mines is conducted from platforms located on the continuous miner about 2.5 m to 3.0 m from the face. The range of bolt locations and possible angles is limited by machinery constraints. Typically 1.8 m or 2.1 m long roof bolts (34 t ultimate tensile capacity) are installed vertically in 27 mm diameter holes using resin grouts. About 0.1 m of the bolt is not inserted into the drill hole. A typical 6 bolt pattern is shown in Figure 13. Depending on ground conditions, 4 m to 8 m long tendons with tensile capacities of around 60 t are used as supplementary support – sometimes these are installed off the miner, or they can be installed at a later stage of the mining process.

Suspension of the failed mass from the un-failed zone higher in the roof is an appropriate basis for support design. Assuming a parabolic shape to the failure zone, it is possible to determine the weight of the block and then to specify the required bolt or tendon length.

For convenience, the following relationships are used: RSI = UCS/(Depth*density) (3)
For development roadways, CI= RSI, K = Ki*Fa (4)
For maingates, CSI=RSI/Mv, K = Ki*Fm/Mv (5)
Where: Ki is the ratio of the horizontal to vertical pre-mining stresses,
Fa is a factor to determine the stress acting normal to the roadway direction from Figure 10,
Fm is a factor to account for the concentration of horizontal stress (Figure 12) and,
Mv is a factor to account for the increase in vertical stress at the maingate.

Based on the approach of Littlejohn (1993) the minimum anchorage length can be readily estimated from the relationship:

Anchorage (m) = 0.4/UCS (MPa)*bolt load (t) (6)

The longer tendons may be installed in larger diameter holes, so a separate anchorage relationship would be required.

Geometry of failure zone

The maximum height of failure can be selected from the E/G =15 chart in Figure 7. The parabolic shape means that the cross sectional area of the failure is given by 0.67 * Hmax*5, and the height as a function of distance across the roadway is shown in Figure 13. For this failure mode, the key bolts are those located nearer the side of the roadway as the more centrally located bolts may be overridden by the failure.

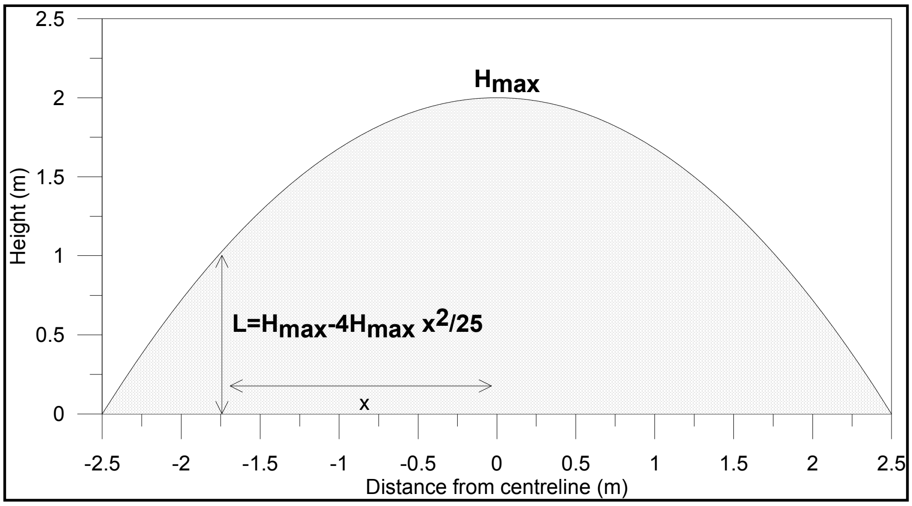

Figure 13 - Typical bolting pattern compared to the geometry of a parabolic shaped failure

Worked examples

Table 2 list the steps in the calculation, with the focus being the two bolts installed 2 m from the centreline or 0.5 m from the rib. The parameters used in Table 2 are based on the Gordon and Tembo (2005) observations from Kestrel Mine whereby an RSI of 3.5 indicated the need to install long tendon support in the maingate prior to longwall retreat, and that a RSI of 2.8 indicated the need to install long tendons during development. The UCS values were estimated from the average sonic velocity over the first 2 m of stone roof. The bolting pattern was 6 by 2.1 m long bolts/metre.

Practical considerations about suspension design

Some cautions are necessary if this model is to be used. First, there are other failure modes in the roof which must also be considered. A design against compressive failure is necessary but may not be adequate for safe outcomes. In particular, the potential delamination of thinly bedded roof must be addressed and this may require more bolts acting in a bedding reinforcement mode. Secondly, reduction in bolting densities to just the two outer bolts is not recommended – the other bolts have a role to play in developing some bridging action onto the anchoring bolts.

Of particular importance is the possibility that the roof may unravel around the tendons. This is particularly the case for the tensile zones associated with low K values. Consideration needs to be given to how the loads are transferred from the centre of the roadway to the anchoring bolts. While the tensile capacities of the anchors are high (34 t to in excess of 60 t) the mesh through which they are installed fails at about two tonnes (Thompson, 2004).

Decisions on the length of the bolts and long tendons need to acknowledge the shape of the failure zone. Support elements installed towards the roadway centreline need to be longer so that they are anchored above the failure zone – a more efficient design may involve shorter tendons installed in pairs and fanned outwards. It would be advisable to install bolts at least 0.5 m from riblines to avoid encountering the possibly highly broken rock at the roof/rib corners shown in Figure 14.

Table 2 - Calculations for a suspension strategy for roof stability based on reports from Kestrel

	Kestrel 300 series	Kestrel 300 series	Kestrel 300 series	Kestrel 200 series	Kestrel 100 series
UCS (MPa)	26	23	18	23	15
Depth (m)	260	260	260	230	230
RSI	4	3.5	2.8	4.0	2.6
$\sigma_H^1/\sigma_v, \sigma_H^2/\sigma_v, \sigma_H^2/\sigma_H^1$	1.6, 1.2, 0.75				
Angle, Fa	10^o, 0.75			30^o, 0.81	
CI	4.0	3.5	2.8	4.0	2.6
K = Fa * Ki	1.2	1.2	1.2	1.3	1.3
Hmax (development)	1.4	2.0	4.0	2.0	4.7
Weight (t)	11.7	16.8	33.5	16.8	39.4
L (at 2.0m)	0.5	0.72	1.44	0.72	1.69
Anchorage length (m)	0.09	0.15	0.38	0.15	0.54
Required bolt length (m)	0.70	0.97	1.92	0.97	2.33
Comment	2.1 m bolt length adequate	2.1 m bolt length adequate	Marginal bolt length if anchorage is not ideal	2.1 m bolt length adequate	2.1 m bolts not adequate
CI (maingate) =	2.0	1.75	1.4	2.0	1.3
Fm	Major stress concentrated, 1.4			Minor stress concentrated, 1.47	Major stress concentrated, 1.58
K (maingate) = Ki*Fm/Mv	1.12			1.18	1.26
Hmax (maingate)	4.1	4.1	4.1	4.2	4.6
Weight (t)	33.5			35.2	38.5
L (at 2.0m)	1.44			1.51	1.66
Anchorage (m)	0.26	0.30	0.38	0.31	0.52
Required bolt length (m)	1.8	1.84	1.91	1.92	2.28
Comment	Marginal bolt length if anchorage is not ideal	Marginal bolt length if anchorage is not ideal	No major change in support requirements	Marginal bolt length if anchorage is not ideal	No major change in support requirements

Figure 14 - Common roof deformation pattern is consistent with outer bolts being the only ones anchored

Finally there are a number of uncertainties involved in the quantification of various parameters. A number of judgment calls are required: the selection of the average strength of what is always layered roof, the confidence in the stress model and particularly Figure 12b, and also the selection of a factor of safety value to apply to the anchorage calculation.

CONCLUSIONS

The brittle failure criterion can be applied to coal measure rocks with the impact of bedding incorporated by invoking transverse isotropy through an E/G ratio of 15. With currently available software, a convenient implementation is by the Mohr Coulomb criterion with a friction angle of 0.0 and a tensile strength cut-off of zero. The simplicity of the criterion allows normalization of some of the variables and the production of design charts.

A support design approach based on the suspension of failure zones has been developed. The two key data sets are the roof strength index (RSI) which can be readily determined in mine exploration programs and the nature of the stress field. Because of the simplicity of the failure criterion, back analysis of previous failures can be used to refine the estimates of some of the key parameters.

Suspension of failed roof is a valid approach to design. When applied to existing case studies, the validity of the approach is strongly supported. Further considerations leads to the recognition of the importance of having a better tensile member across the roof line. This may be a serious weakness in current roof support designs.

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