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Publication Details

This conference paper was originally published as Colwell, M, Frith, R and Reed, G, Tailgate 802 - Grasstree Mine: A Case Study in Pragmatic Roof Support Design in Aziz, N (ed), Coal 2008: Coal Operators' Conference, University of Wollongong & the Australasian Institute of Mining and Metallurgy, 2008, 75-91.

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TAILGATE 802 - GRASSTREE MINE: A CASE STUDY IN PRAGMATIC ROADWAY ROOF SUPPORT DESIGN

Mark Colwell[1], Russell Frith[2], Guy Reed[3]

ABSTRACT: In February 2007 Colwell Geotechnical Services was commissioned by Anglo Coal's Grasstree Mine (Grasstree) in the Bowen Basin of Central Queensland to assess the future roadway serviceability and secondary roof support requirements associated with the tailgate of LW 802 (i.e. TG 802). In most instances the ALTS (Analysis of Longwall Tailgate Serviceability) Design Methodology can be directly applied to undertake such an assessment. Whilst ALTS formed the basis for the secondary roof support strategy for the vast bulk of the tailgate, there were two particular aspects associated with TG 802 that required the use other design techniques both in combination with and in addition to ALTS.

Firstly, the gateroad development associated with Grasstree is based on a 3-heading rather than the typical 2-heading configuration employed by Australian Collieries (upon which the ALTS database was formulated). To cater for this in assessing tailgate serviceability, ALTS was combined with its US counterpart ALPS (Analysis of Pillar Stability). Secondly, the installation face of LW 802 was located approximately 260 m inbye of the start of LW 801 and due to the tailgate's orientation and direction of longwall retreat in relation to the major horizontal stress direction, a significant stress concentration acting across the tailgate roof was predicted as the face of LW 802 approached and passed the installation face of LW 801. This situation is sometimes referred to as a "Super Stress Notch". An analytical approach was adopted when assessing the secondary roof support requirements associated with this section of TG 802.

While this paper summarises the process by which the secondary roof support strategy was developed and subsequently implemented for TG 802, the paper primarily focuses on three issues 1) characterisation of the roof, 2) the analytical design procedure associated with the "Super Stress Notch" zone and 3) what roadway performance outcome constitutes a successful design.

INTRODUCTION

Grasstree Mine (Grasstree) is located in the Bowen Basin Coalfield of Central Queensland and is operated by Anglo Coal (Capcoal Management) Pty. Ltd. The resource area is traversed by the Grasstree Dyke, a 10 m to 15 m thick very strong dolerite dyke that effectively divides the reserves into two separate mining blocks, referred to as the 900's block on the north side and the 800's block on the south side.

In relation to Grasstree, mining in the German Creek Seam commenced in November 2003 with gateroad development initially focused on the 800 series longwall panels. Longwall extraction commenced in Longwall Panel 801 (LW 801) in September 2006 and currently extraction is taking place in LW 802. Figure 1 depicts the location of the 800 and (proposed) 900 series longwall panels with respect to the adjoining mining areas of Central, Southern and Bundoora Collieries. The 800's are bounded to the west by the Cattle Yard Fault and to the east by another system of major faults. The northern limit is the Grasstree Dyke, whilst the southern limit is determined by the lease boundary.

Prior to the extraction of LW 802, Colwell Geotechnical Services (CGS) was commissioned by Grasstree to assess the future roadway serviceability associated with the tailgate of LW 802 (TG 802, which is also designated as 'A' Heading - MG 801 in Figure 2) and if necessary recommend secondary support strategies so as to maintain an adequate level of tailgate serviceability during the extraction of LW 802.

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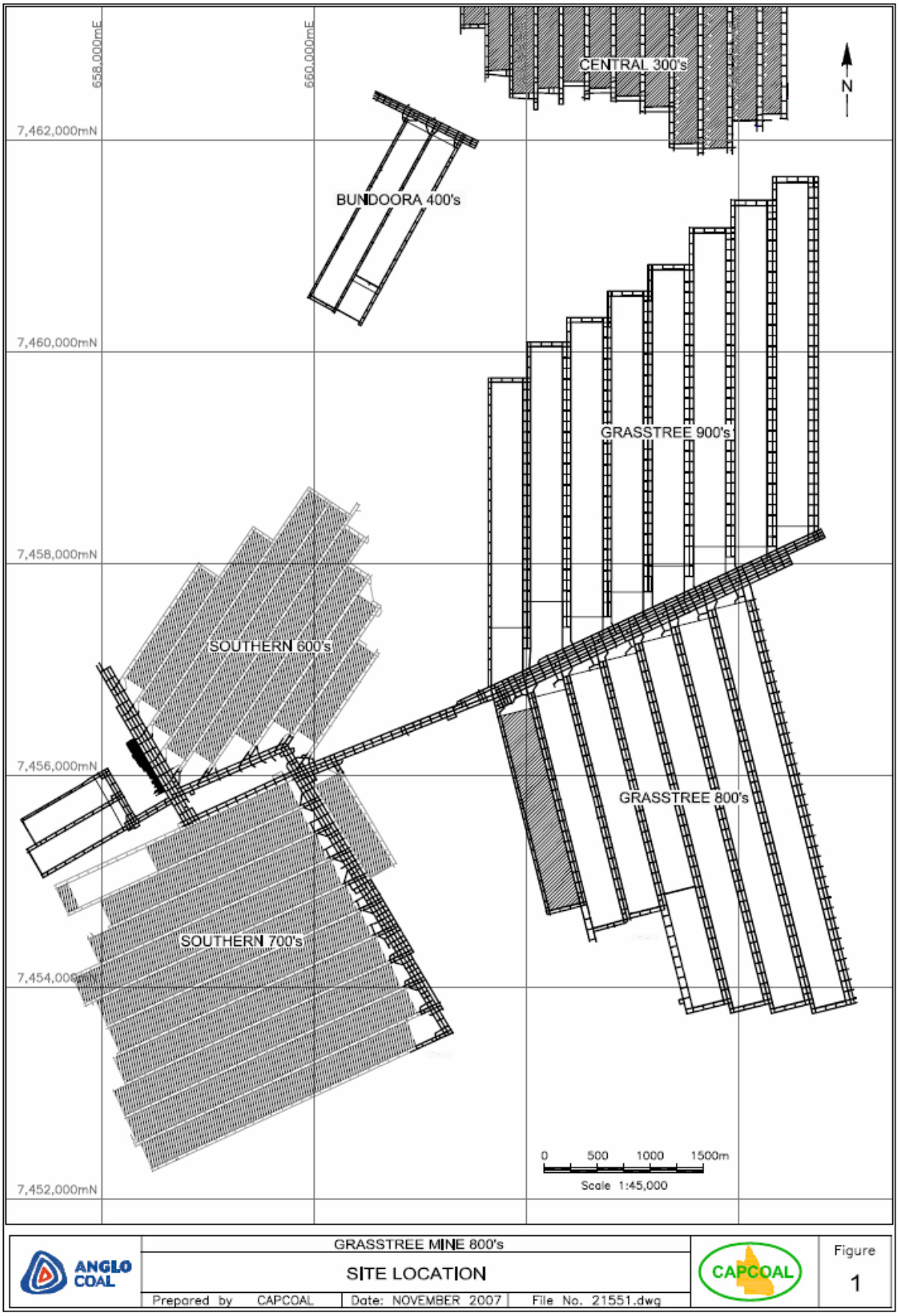

Figure 1 - Site location

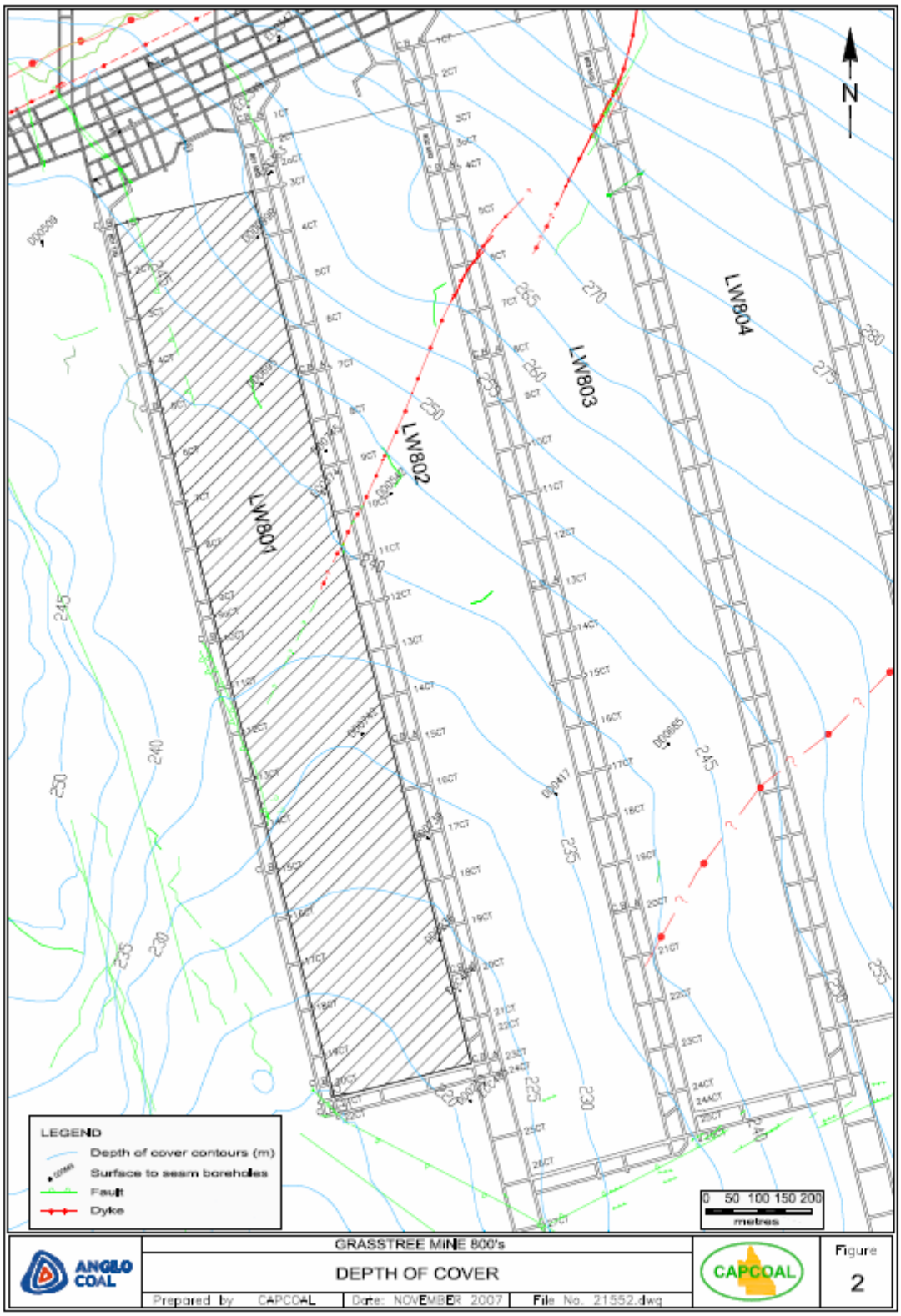

Figure 2 – Depth of cover

PREVIOUS MINESITE EXPERIENCE

Except in proximity to significant geological structure (i.e. faulting) the roof of all roadways was simply supported with the primary support installed off the continuous miner, which incorporated 6 x 1.8 m X-grade bolts every 1.3 m with an applied pre-load of approximately 8 t while utilising full roof mesh. The 4:2 staggered pattern employed is essentially a row of 4 bolts alternating with a 2 bolt row, where the row spacing between the alternating rows is 0.65 m resulting in 6 bolts every 1.3 m.

A substantial proportion of LW 801 had been extracted when commissioning CGS to undertake this study and it was found (based on observation and Tell-Tate data) that during the extraction of LW 801, all associated roadways (i.e. belt road, travel road & tailgate) exhibited adequate roof stability (i.e. minimal roof movement & few stability concerns) without the use of any secondary roof support (except in proximity to significant geological structures i.e. faulting or for operational purposes at the "mouths" of cut-throughs). Therefore the 6 bolt every 1.3 m primary support pattern provided satisfactory roof reinforcement and the use of roof mesh adequately contained any surface slabbing.

SELECTING THE APPROPRIATE DESIGN TECHNIQUE

When assessing tailgate serviceability and roof support requirements, generally the ALTS (Analysis of Longwall Tailgate Serviceability - Colwell et al, 2003) Design Methodology can be directly applied to undertake such an assessment for an Australian Colliery. However (as illustrated in Figure 2) in this instance the maingate development associated with Grasstree is based on a 3-heading rather than the typical 2-heading configuration employed by Australian Collieries (upon which the ALTS database was formulated).

ALTS was developed using an Australian database and is specific to 2-heading gateroad development due to the fact that this is by far the dominant gateroad development configuration utilised by Australian longwall operations. However, ALPS (Analysis of Pillar Stability - Mark et al, 1994) was developed based on US data and is specific to 3-heading gateroad development.

Both techniques are utilised to assess tailgate serviceability and the corresponding chain pillar width(s) and ground support required to maintain satisfactory roadway conditions throughout the longwall extraction cycle. A straightforward method of combining both techniques to assess the 3-heading gateroad configuration for Australian conditions is described below.

In relation to this study ALPS was only utilised to assist with the chain pillar evaluation as any roof support recommendations associated with ALPS should not be directly applied to Australian collieries.

Within ALPS the parameter utilised to quantify the chain pillar systems' contribution to overall tailgate serviceability is referred to as the ALPS Stability Factor or ALPS SF and within ALTS it is referred as the Tailgate Stability Factor or TG SF. Both values are essentially calculated in an identical manner utilising the abutment angle model to calculate pillar load(s) and the Bieniawski (1992) pillar strength equation to evaluate the "strength" or load bearing capacity of the pillar system. The ALPS SF & TG SF are not Factors of Safety; they are pillar ratings employed in the respective analyses and design procedures.

The recommended chain pillar factors for ALPS and ALTS are respectively referred to as the ALP SF_R and TG SF_R, and both parameters are directly related to the Coal Mine Roof Rating (CMRR, refer Mark & Molinda, 2003) via the following equations:

$$\text{ALPS } SF_R = 1.76 - 0.014 \text{ CMRR} \quad (1)$$

$$\text{TG } SF_R = 2.881 - 0.0343 \text{ CMRR} \quad (2)$$

To convert an ALPS SF for a 3-heading gateroad pillar system to an equivalent TG SF (for the CMRR under consideration), which can then be utilised within ALTS to firstly assess the suitability of the chain pillar sizing in terms of the Australian database and secondly the associated roof support requirements, the following equation is employed:

$$\text{Equivalent TG SF} = \text{Calculated ALPS SF} \times (\text{TG } SF_R/\text{ALPS } SF_R) \quad (3)$$

Once an Equivalent TG SF was established, the ALTS technique could then be directly applied in the evaluation of secondary roof support requirements to maintain satisfactory tailgate serviceability of TG 802 outbye of 22 C/T.

Secondary Roof Support Evaluation and Design – inbye of 22 C/T

ALTS specifically relates to the serviceability design of tailgates subject to double pass longwall extraction. When acting as the travel road, the roof is typically subject to some level of *in situ* horizontal stress concentration due to longwall retreat. This is typically referred to as Maingate Stress Notching and in terms of the travel road the level of horizontal stress increase will depend on several factors, including chain pillar width (i.e. separation of the travel road from the retreating longwall face), direction of longwall retreat and orientation and intensity of both the major and minor horizontal stress.

As the longwall face retreats, the adjacent goaf prevents any further concentration of the *in situ* horizontal stress within the roof of the travel road when sufficiently inbye of the faceline or when acting as the tailgate of the next longwall panel. In this instance ALTS provides the means by which a compromise (or the interaction) between chain pillar width and roof support can be assessed in terms of satisfactory gateroad performance.

However, that section of TG 802 inbye of 23 C/T (Figure 2) is subject to single pass longwall extraction and without the adjacent goaf of LW 801 it is considered highly likely that this section of TG 802 will be subject to horizontal stress notching effects during LW 802 retreat.

Down-hole stress measurements indicate a fairly consistent north-northeast direction with respect to the major horizontal stress. The gateroads are orientated at approximately 166° Grid North and therefore in relation to TG 801 and that section of TG 802 inbye of 23 C/T (Figure 2), it was found (based on the down-hole stress measurements) that the major horizontal stress is orientated at an angle in the order of 25° to 45° to the gateroad direction. Such an orientation would likely result in a concentration of the major horizontal stress acting across the "unprotected" tailgate roof as a result of longwall retreat.

Furthermore the area adjacent to 23 C/T will be subjected to a double notch (i.e. concentration) of the major horizontal stress as LW 802 approaches and passes the installation roadway associated with LW 801. This double notch (or concentration) of the major horizontal stress is sometimes referred to as a "super stress notch" or "super-stressing" of the tailgate. The recommendations for this *Super Stress Notch* zone extended from 25 C/T to 22 C/T to allow for both the initial onset of the additional stress increase and any possible horizontal stress increase extending outbye of 23 C/T

While ALTS can be adapted to assess roof support requirements for tailgates not protected by an adjacent goaf (e.g. TG 801 in Figure 2), it was decided that in this instance an analytical approach (as described by Frith & Colwell, 2006) could best be utilised to back-analyse/compare the behaviour of TG 801 to assist in designing the secondary roof support strategies associated with the *Super Stress Notch* zone anticipated for TG 802.

COAL MINE ROOF RATING (CMRR)

The critical input parameter utilised by both ALPS and ALTS for the assessment of tailgate serviceability and roof support requirements/design is the CMRR. When calculating the CMRR an important component is a rock unit's fracture spacing, which also happens to be one of the critical input parameters associated with the analytical model utilised to assess the secondary roof support requirements associated with the *Super Stress Notch* zone of TG 802

Grasstree were able to provide approximately 20 boreholes (near or adjacent to TG 802) of which 17 were suitably geotechnically logged to ascertain credible CMRR values. Those 17 borehole locations are detailed on Figures 2 and 3.

Ward (2006) reports the immediate roof (primarily in terms of longwall geomechanics) is traditionally taken as the strata overlying the German Creek Seam up to the Corvus 2 Seam (approximately 18m thick). This section of roof is customarily divided into five separate geomechanical units (ROF1 to ROF5 in the Capcoal database). However the CMRR is specific to the primary bolt length and is utilised within ALTS to assess roadway roof performance and associated roof support requirements. For this study it is only roof units ROF1 and ROF2 that affect the CMRR.

ROF1 is used to identify the first layer of the immediate roadway roof when it is significantly weaker than the overlying stratum. It is for the most part a fine grained laminated micaceous sandstone, interlaminated to varying degrees with siltstone. In the 700's it frequently contained fossilised ripple marks on some bedding planes, which tended to encourage delamination. This has also been identified in the 800's.

ROF1 has been interpreted to have an average sonic derived UCS strength of about 50 MPa, but drops to as low as 20 MPa in a few isolated cases. The laboratory testing associated with the 17 boreholes used in this study returned an average UCS of 36.4 MPa for ROF1 with a standard deviation of 7.5 MPa. In terms of moisture sensitivity, ROF1 is typically classified as not sensitive while occasionally deemed as slightly sensitive. Such an interpretation is consistent with the limited degree of 'roof flaking' observed during the underground inspection. Figure 3 shows the thickness contours for ROF1 associated with TG 802 where it typically ranges between 0.7 m and 1.4 m thick.

ROF2 is typically a strong fine to medium grained sandstone, thinly bedded to massive, with micaceous siltstone bands and its (sonic derived) UCS strength generally ranges between 85 and 95 MPa, but it can reach 100 MPa or more. Along TG 802 it ranges between approximately 7 m and 9 m thick. Although ROF2 is thick and strong, it frequently contains one or more thin siltstone bands, usually with some degree of bedding plane shearing that can form potential separation planes.

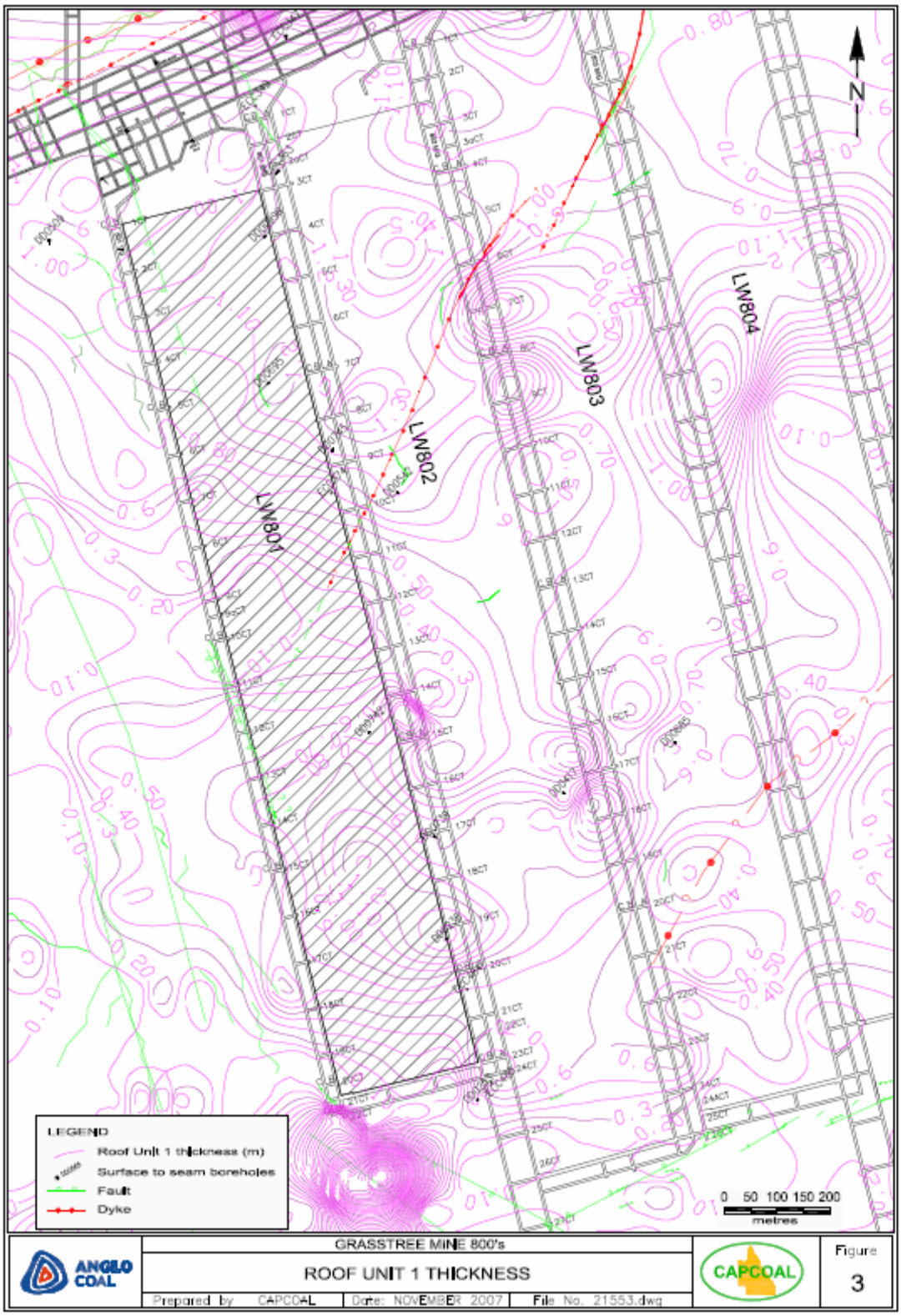

Figure 3 – Roof unit 1 thickness

Calculating the CMRR from borehole core requires that the hole is geotechnically logged (in the splits prior to its placement in the core tray). Specifically, values for fracture spacing, RQD (Deere & Miller, 1966) and/or diametral point load strength ($I_{s(50)}$) are necessary for each geotechnical unit within (and immediately above) the bolted horizon. The fracture spacing (FS) is defined as the average spacing (mm) of actual core breaks or fractures within the geotechnical unit (e.g. if 8 pieces are identified in a 1 m section of core then the FS associated with that 1 m length of core is 125 mm)

It should be noted the CMRR methodology dictates that in terms of the RQD, FS and Diametral $I_{s(50)}$, the value utilised within the individual Unit Rating (UR) calculation is that which results in the lowest UR, however when the FS > 1.22 m then the Diametral $I_{s(50)}$ value should be used.

The boreholes provided by Grasstree were generally suitably geotechnically logged detailing the location and nature of the core breaks, which allowed for a reasonably accurate determination of the fracture spacing associated with roof units ROF1 and ROF2 for most of the boreholes provided. A limited amount of diametral point load strength testing had been undertaken and that was predominantly in relation to ROF1.

It became apparent that utilising the FS in relation to the weaker and often laminated ROF1 for those boreholes where Diametral $I_{s(50)}$ values were not available would result in an overestimate of ROF1's Unit Rating and therefore an overestimate of the borehole's CMRR value. When there are a limited number of boreholes or in this instance where a full suite of geotechnical testing is not available for each borehole or each unit, then to gain a better appreciation of the CMRR (particularly its variability) to utilise for design purposes, it is often valuable to "pool" all of the available borehole geotechnical data in terms of mean and related standard deviation values.

Based on the borehole data, the mean and standard deviation associated ROF1's Diametral $I_{s(50)}$ and UCS is respectively 0.43 MPa /0.28 MPa and 36.4 MPa /7.5 MPa, while the mean and standard deviation associated ROF2's Fracture Spacing are 349mm/88mm. Table 1 summarises CMRR calculations for various combinations of these parameters utilising the mean and the mean less one standard deviation with respect to ROF1's Diametral $I_{s(50)}$ and ROF2's Fracture Spacing.

Table 1 - Summary of CMRR calculations for 1.8m bolt length

Case	ROF1 Thickness	Unit No.	Description of Geotechnical Units which form the Immediate Roof	UCS (MPa)	Unit Rating	SBADJ	CMRR	CMRR - SBADJ
1	0.7m	1	ROF1 with $Is_{(50)}$ = 0.43 MPa	36.4	41.7	3.0	**54.9**	**51.9**
		2	ROF2 with 349mm fracture spacing	80.0	58.3			
2	1m	1	ROF1 with $Is_{(50)}$ = 0.43 MPa	36.4	41.7	4.7	**53.8**	**49.1**
		2	ROF2 with 349mm fracture spacing	80.0	58.3			
3	1.3m	1	ROF1 with $Is_{(50)}$ = 0.43 MPa	36.4	41.7	6.0	**52.3**	**46.3**
		2	ROF2 with 349mm fracture spacing	80.0	58.3			
4	0.7m	1	ROF1 with $Is_{(50)}$ = 0.15 MPa	36.4	37.7	4.3	**54.6**	**50.3**
		2	ROF2 with 349mm fracture spacing	80.0	58.3			
5	1m	1	ROF1 with $Is_{(50)}$ = 0.15 MPa	36.4	37.7	6.4	**53.3**	**46.9**
		2	ROF2 with 349mm fracture spacing	80.0	58.3			
6	1.3m	1	ROF1 with $Is_{(50)}$ = 0.15 MPa	36.4	37.7	7.9	**51.3**	**43.4**
		2	ROF2 with 349mm fracture spacing	80.0	58.3			
7	0.7m	1	ROF1 with $Is_{(50)}$ = 0.15 MPa	36.4	37.7	3.8	**53.1**	**49.3**
		2	ROF2 with 261mm fracture spacing	80.0	56.7			
8	1m	1	ROF1 with $Is_{(50)}$ = 0.15 MPa	36.4	37.7	5.7	**51.8**	**46.1**
		2	ROF2 with 261mm fracture spacing	80.0	56.7			
9	1.3m	1	ROF1 with $Is_{(50)}$ = 0.15 MPa	36.4	37.7	7.1	**50.1**	**43.0**
		2	ROF2 with 261mm fracture spacing	80.0	56.7			

A variation in a unit's UCS of say 10 MPa does not have a dramatic effect on the eventual CMRR and therefore the UCS for both units is held constant using the average UCS of 36.4 MPa for ROF1 and a realistic minimum value of 80 MPa for ROF2 (Table 1).

Table 1 also summarises the Strong Bed Adjustment (SBADJ) component of the CMRR. One of the most important concepts incorporated into the CMRR is that of the SBADJ. Many years of experience with roof bolting has found that the overall structural competence of bolted roof is very often determined by the quality of the most competent bed within the bolted interval. However current research strongly suggests that primary support densities should be assessed or based on the CMRR minus the SBADJ (last column in Table 1).

To provide an appreciation of the effect of ROF1's unit thickness on the SBADJ and CMRR, the CMRR calculation is undertaken for three unit thicknesses being 0.7 m, 1 m & 1.3 m. The colour coding associated with Table 1 has the following meaning:

1. Yellow (Cases 1 to 3) indicates that the mean Diametral $I_{s(50)}$ for ROF1 (0.43 MPa) and mean FS (349mm) for ROF2 are utilised to calculate the respective Unit Ratings.

2. Green (Cases 4 to 6) indicates that the mean less one standard deviation Diametral $I_{s(50)}$ for ROF1 (0.43 – 0.28 = 0.15 MPa) and mean FS for ROF2 (349mm) are utilised to calculate the respective Unit Ratings.

3. Blue (Cases 7 to 9) indicates that the mean less one standard deviation Diametral $I_{s(50)}$ for ROF1 (0.43 - 0.28 = 0.15 MPa) and mean less one standard deviation FS for ROF2 (349 – 88 = 261mm) are utilised to calculate the respective Unit Ratings.

Except for a small zone adjacent to 8 C/T TG 802 (refer Figure 3), the thickness of ROF1 is $\leq$ 1.3 m along the length of TG 802. The CMRR values associated with Table 1 and the CMRR values associated with the individual boreholes strongly suggested that as long as the primary support pattern (of 6 x 1.8 m X-grade bolts at 1.3 m) attained *"solid anchorage"* within ROF2 and reinforced ROF1 to build a largely self-supporting unit, then a CMRR of between 50 to 55 could be confidently used to evaluate the suitability of the chain pillar design associated with TG 802 (i.e. MG 801) and secondary support requirements.

However the CMRR analyses also contained a strong warning particularly in relation to those zones where the thickness of ROF1 is $\geq$ 1m; that is if for any reason *"solid anchorage"* is not attained by the 1.8m bolt within ROF2 or the primary support density is insufficient to satisfactorily reinforce ROF1 then an effective CMRR of approximately 43 to 46 would result. The recommended support strategy took the above into consideration.

It was decided to characterise TG 802 roof in terms of three zones when utilising ALTS to assess secondary support requirements (outbye of 22 C/T) prior to longwall retreat, with those three zones being:

Zone 1. Where ROF1 $\leq$ 0.7 m the CMRR is taken to be 54 with a SBADJ of 4. With respect to Figure 3 this applies to those sections of TG 802 from 10½ C/T to 16½ C/T and inbye of 24 C/T.

Zone 2. Where 0.7 m < ROF1 $\leq$ 1 m the CMRR is taken to be 52 with a SBADJ of 5. With respect to Figure 3 this applies to those sections of TG 802 from 9½ C/T to 10½ C/T, 16½ C/T to just outbye of 18 C/T and 20 C/T to 24 C/T (inclusive).

Zone 3. Where ROF1 > 1 m the CMRR is taken to be 51 with a SBADJ of 7. With respect to Figure 3 this applies to those sections of TG 802 outbye of 9½ C/T and 18 C/T t to just outbye of 20 C/T.

ANALYTICAL MODEL (Factor of Safety Approach)

The stability of many engineering structures can be and indeed is evaluated based on a Factor of Safety (FOS) concept, this being a measure of the load applied to that structure in comparison to its ability to accommodate that load without undergoing yield or failure. This is usually expressed as:

$$FOS = \text{load bearing ability/applied load} \qquad (4)$$

This approach is commonly used in coal pillar design worldwide with the UNSW Pillar Design Procedure (Galvin et al, 1999) being one such example. In this case the strength of the coal pillar is given by a specific equation that has been determined empirically, based on an industry database of stable and failed pillar cases, typically under reliably inferred Full Tributary Area loading conditions.

Figure 4 details the secondary support installed within these zones.

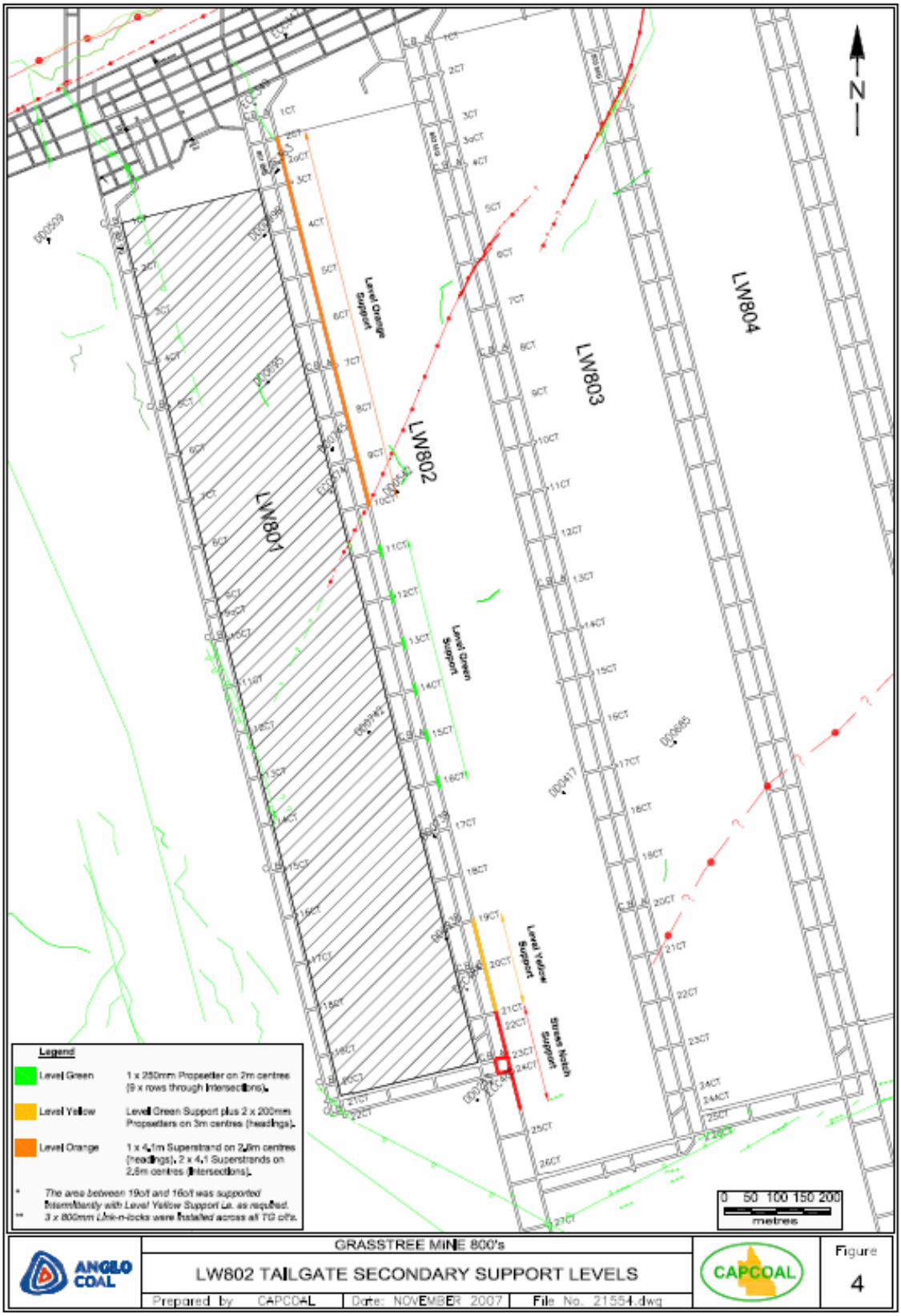

Figure 4 – LW 802 tailgate secondary support levels

The Factor of Safety is essentially a risk based measure of the likelihood of the design being inadequate, acceptable values being related to the likely consequence of the design being inadequate and the associated impacts (business, safety or otherwise).

Whilst it is far less common to do so, there is no obvious technical reason as to why roof stability in mine roadways cannot be evaluated and designed for using a similar concept. The problem has always been in being able to reliably assign magnitudes or quantities to the various components of the equation. However as with the approach taken for coal pillar design (i.e. the use of an empirically derived strength equation rather than one based on first principles), industry or individual mine site experience can potentially be used to "calibrate" various elements of the problem and so allow a site specific Factor of Safety approach to be adopted.

It is noted though that it is still critical to have a "cause and effect" understanding of the impact of the various technical parameters, simply that assigning numerical values can be based in part on mining experience rather than purely from first principles. For the problem of roadway roof stability, the general design equation can be re-written as:

FOS = load bearing ability (roof strata + roof support)/applied load (5)

In this instance the applied load acts horizontally across the roof and is a product of the *in situ* horizontal stress and concentration thereof as a result of the mining process. Therefore the resolution of equation 5 is across the roof, which necessitates that the load bearing ability roof strata and the load bearing ability roof support is also resolved accordingly.

The authors assess (based on industry research/experience) that for small vertical roof displacements (up to around 50mm and possibly to 100mm), slender beam behaviour or buckling is typically the dominant behavioural mechanism occurring within the immediate coal mine roof measures.

Uncontrolled roof behaviour of this type may then lead to other failure mechanisms occurring and to large scale roof displacements or roof falls. By understanding slender beam behaviour, it allows for the most pragmatic way of evaluating the initial load bearing ability of the strata (P_{roof}) and subsequently determining the lateral resistance offered by the roof support ($P_{support}$). In terms of the use and application of the analytical model there are three basic components:

1. Evaluation of horizontal stress acting across the roof within individual roof units (typically at various key points in the mining process).

2. Determination of the material properties (including Modulus, UCS as well "beam" thickness & length) associated with the immediate roof units, which are required both in terms of Point 1 above and in evaluating the load bearing capacity of the strata.

3. Utilising a load-balance approach (which incorporates well established load-bearing characteristics of slender beam behaviour and mechanical advantage) a Factor of Safety (FOS) is calculated (refer equation 5). Engineering judgement needs to be applied in selecting a suitable FOS for design purposes this being a risk-based consideration that is always discussed with mine management as part of finalising design outcomes.

It is noted that in the case of design for longwall retreat purposes, the calculated Factor of Safety has the following general definition:

"Factor of Safety against the onset of a process (i.e. stress driven roof deterioration), that if allowed to sufficiently propagate, could lead to a major roof fall".

It is not a Factor of Safety against a roof fall occurring as (a) the conditions under which a roof fall finally occurs are not well defined and (b) practical mining considerations requires that the roof be maintained as stable as possible during longwall retreat so as to minimise any potential impact on face production. Clearly losses can occur by simply excessive roof convergence trapping equipment or deteriorating visible roof conditions necessitating the installation of additional roof support.

Therefore the consequence of an inadequate design is logically the triggering of the longwall retreat TARP and the installation of additional support. It is not the imminent occurrence of a major roof fall and this always needs to be kept in mind when considering the actual magnitude of an adequate design Factor of Safety.

Based on the analyses it became apparent that in terms of overall satisfactory roof performance, the stability of ROF2 was the critical determinant. It was calculated (and subsequently found) that while ROF2 remained stable the installed level of primary support would adequately reinforce ROF1 with respect to the anticipated stress increase. However, should ROF2 buckle (i.e. become unstable) it was considered highly likely the significant roof softening would occur and that such softening could lead to a major roof fall.

Given the critical nature of ROF2 and due to space constraints understandably associated with a technical paper of this type, the following discussion is focused on the analyses associated with ROF2.

Evaluation of Horizontal Stress acting within ROF2

The general equation for the major horizontal stress acting within a rock unit (refer Nemcik et al, 2005) can be written as:

$$\sigma_H = \nu/(1-\nu).\ \sigma_V + TSF \times E \qquad (6)$$

where:

ν = Poisson's Ratio
$\nu/(1-\nu) = K_o$
E = Young's Modulus (in GPa)
σ_V = vertical stress acting (where σ_V is approximately equal to 0.025 H, MPa)
TSF = empirically derived constant (Tectonic Stress Factor)

Therefore by knowing the depth of cover (H) as well as Young's Modulus and Poisson's Ratio of the host material, a credible estimate of the major horizontal stress acting within a specific roof unit can now be made. It is noted that in relation to equation 6 Young's Modulus is quoted in GPa, while the stress outcome is in MPa.

Utilising the down-hole stress measurement data provided by Grasstree and the process of analysis as outlined by Colwell & Frith (2006) a TSF of approximately 0.55 resulted. In conjunction with laboratory testing, the average Young's Modulus and Poisson's Ratio were estimated to be 20.08 GPa and 0.187 respectively and therefore for ROF2 equation 6 can be re-written as:

$$\sigma_H = 5.75 \times 10^{-3}\ H + 11.04\ (MPa) \qquad (7)$$

A representative depth of cover (H) suitable for back-analysis in relation to TG 801 is 240 m, while the depth of cover (H) associated with TG 802 inbye of 22 C/T is approximately 225 m. Therefore at these respective cover depths the *in situ* major horizontal stress acting within ROF2 prior to mining is estimated to be 12.42 MPa (TG 801) and 12.33 MPa (*Super Stress Notch* zone)

In terms of the horizontal stress change in the roof that occurs along a tailgate without an adjacent goaf, it will be assigned based on the research findings of Gale and Matthews (1992) whereby they linked the Stress Concentration Factor (SCF for a single stress notch as a multiple of the *in situ* stress) with the angle between the gateroad driveage direction and that of the major horizontal stress (Figure 5).

As previously discussed the gateroads are orientated at approximately 166° Grid North and therefore in relation to TG 801 and that section of TG 802 inbye of 23 C/T, the major horizontal stress is orientated at an angle ranging from around 025° to 045° (average 34.3°) based on the down-hole stress measurement. Such an orientation would likely result in a concentration of the *in situ* major horizontal stress acting across the roof as a result of longwall retreat. Based on the orientation of the major horizontal stress to the gateroad direction and with reference to Figure 5, a Stress Concentration Factor (SCF) for a single extraction panel of 1.6 up to around 2 could apply.

On the basis that a super-stress notch is essentially two horizontal stress notches coming together, a Stress Concentration Factor of around 4 could be argued as being applicable (i.e. a stress notch of an already notched *in situ* major horizontal stress). However a slightly reduced Stress Concentration Factor of 3.5 will be applied for design purposes, this making some allowance for the presence of the chain pillar between the two goafs and the assumption that this would reduce the overall horizontal stress concentration ahead of the second longwall. It is noted that to the best of the authors' knowledge, no stress monitoring data has ever been collected to fully quantify this issue.

Evaluating the Load Bearing Capacity of ROF2 (P_{roof})

As previously discussed, roadway roof behaviour and associated instability is primarily based around the uncontrolled buckling of slender horizontal beams under the action of horizontal stress. Early theoretical models for this simply used the concepts of Euler Buckling. However more recent developments by the authors have included other structural concepts that allow a complete range of possible behaviour to be considered according to beam geometry with Euler Buckling representing a relatively small proportion of the full range.

Behaviour outside the Euler range can be defined by a number of different structural concepts. For the purpose of this model use will be made of what is termed as the Johnson formula (see http://physics.uwstout.edu/statstr/statics/ or Beer, Johnston and DeWolf (2006) for more general information on this topic). Utilising these concepts of beam behaviour under axial load in conjunction with the roof's material & physical properties an estimate of its load bearing ability (P_{roof}) can be deduced.

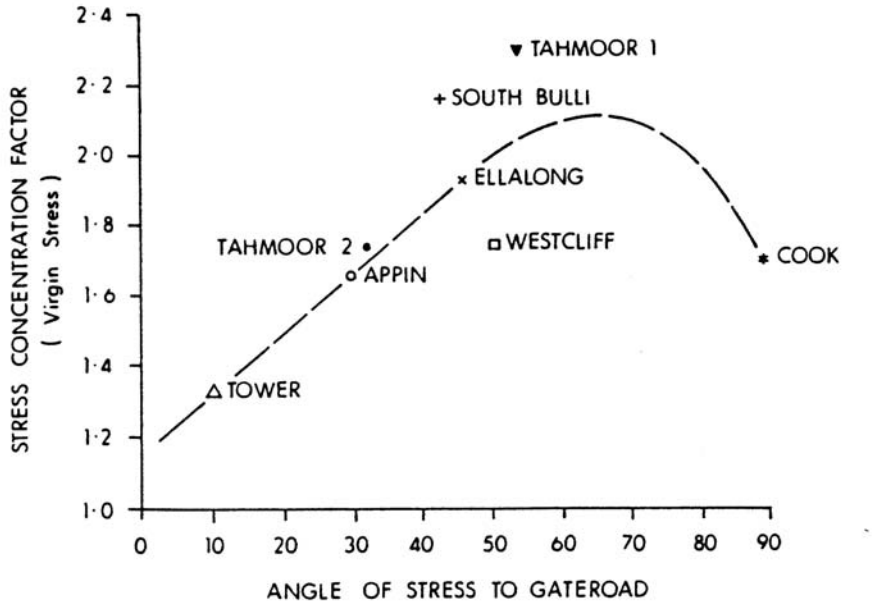

Figure 5 - Relationship between horizontal SCF and angle of gateroad to stress direction (after Gale and Matthews 1992)

In terms of buckling; beams (or columns) have typically been divided into three general types:

(i) Short Beams
(ii) Intermediate Beams, and
(iii) Long Beams.

A short beam will not fail due to buckling, as the ratio of the beam length to the effective cross sectional area is too small. Rather a short, 'thick' beam, axially loaded, will fail in simple compressive failure; that is when the load/area of the beam exceeds the allowable stress. The critical or allowable stress associated with long beams/columns is governed by equation 8 (Euler Formula).

$$\sigma_{crit} = \pi^2 E /[12(L_{eff} /d)^2] \qquad (8)$$

Where E is Young's Modulus, L_{eff} is the effective beam length and d is beam thickness.

The above formula only applies while the material is in the elastic region and therefore the maximum allowable stress is limited by the yield strength (σ_y) of the material, it being taken to be 70% of the UCS herein.

There are a number of semi-empirical formulas for buckling in beams/columns in the intermediate length (and short) range. One of these is the J.B. Johnson Formula. The J.B. Johnson formula is the equation of a parabola with the following characteristics. For a graph of stress versus slenderness ratio, the parabola has its vertex at the value of the yield stress on the y-axis. Additionally, the parabola is tangent to the Euler curve at a value of the slenderness ratio, such that the corresponding stress is one-half of the yield stress. For further information refer http://physics.uwstout.edu/StatStr/statics/Columns/cols62.htm.

The Johnson equation for the allowable stress is as follows:

$$\sigma_{crit} = [1 - (L_{eff} /\acute{r})^2/(2 C^2)] \sigma_y \qquad (9)$$

Where $\acute{r}$ is beam's Radius of Gyration and C is the beam's Critical Slenderness Ratio

$$\acute{r} = I/A \text{ and } C = (2 \pi^2 E/\sigma_y)^{0.5}$$

Where I is the beam's moment of inertia and equals $bd^3/12$ and A is the cross-sectional area of the beam (i.e. A = bd). *Note for plane strain analysis the beam width, b, equals 1 m.*

Essentially when the beam's Slenderness Ratio (L_{eff} /ŕ) is greater than the beam's Critical Slenderness Ratio (C) then equation 8 is used to calculate the beam's load bearing capacity and when the beam's Slenderness Ratio less than C then equation 9 is invoked.

Therefore in undertaking these analyses with respect to ROF2 the information required is Modulus (E) and σ_y (where σ_y = 0.7 x UCS) of the rock unit and the beam's effective length (L_{eff}) and thickness (d). As previously discussed for ROF2, its Modulus (E) is taken to be 20.08 GPa while a realistic minimum value for the UCS is 80 MPa and therefore its yield strength (σ_y) is taken to be 56 MPa.

In terms of the individual beams that will form within ROF2; firstly it is assumed the end fixing condition is pinned and therefore L_{eff} equals the roadway width of 5.2 m and secondly the beam thickness is equal to the fracture spacing as previously discussed under the CMRR section of this paper.

Therefore based on the above for an ROF2 beam thickness of 349 mm (i.e. average fracture spacing) the Slenderness Ratio equates to 51.6 while C equals 84.1 and therefore the load bearing capacity equals 45.5 MPa. Consistent with the CMRR calculations an estimate of the load bearing capacity was also made for a beam thickness of 261 mm (mean less one standard deviation) and based on this beam geometry a load bearing capacity of 37.2 MPa is returned.

It is understood that during the extraction of LW 801, TG 801 exhibited adequate roof serviceability with few stability concerns (except in proximity to significant geological structures) without the use of any secondary roof support. In relation to the ROF2 unit, the horizontal stress acting at the tailgate corner with the longwall face is taken to be a maximum of 24.86 MPa (i.e. 2 x 12.43). When this is compared to the allowable stress range within this unit at 5.2 m width of approximately 37.2 MPa to 45.5 MPa, a Factor of Safety (in terms of stability) at the TG corner of 1.50 to 1.83 is calculated. This outcome is judged to be consistent with the satisfactory extraction experience in TG 801.

The horizontal stress assumed to be acting in the ROF2 unit in the *Super Stress Notch* zone is some 43.2 MPa (i.e. 3.5 x 12.35 MPa). As previously indicated the allowable stress range for the ROF2 is approximately 37.2 MPa to 45.5 MPa, giving an overall Factor of Safety range without secondary support of 0.86 to 1.05. This was judged to be inadequate given that the load bearing capacity offered by the primary roof support is critically dependent on the bolt or tendon anchoring above the height of softening (i.e. anchoring within a stable ROF2).

Achieving an overall Factor of Safety in the range of 1.50 to 1.83 (as found for ROF2 from the back-analysis of TG 801) required an additional ≈30 MPa of load-bearing capacity to be offered by longer tendon support. Utilising the concept of Mechanical Advantage inherent in a buckling beam (refer Frith, 2000), it was calculated that this could be achieved by installing 3 x 6.1 m High Strength Tendons/Cables at 1.6 m spacing with an applied pre-load of 30 t. The three tendon pattern was one of equal spacing across the roof at 1.3 m, 2.6 m and 3.9 m in from either rib side and it was recommended that they be post-grouted.

The above design option (for the *Super Stress Notch* zone) was provided to Grasstree on the basis of a "cribless tailgate: i.e. such that standing secondary support should not be required and therefore trigger levels should not be exceeded. However (as requested by the mine) several secondary roof support strategies (utilising various hardware including tendon & standing support) were considered. These various strategies included both the incorporation of standing support as well as the exclusion of such roof support (i.e. "cribless tailgate").

Various options were presented to Grasstree, which were then carefully considered as part of the colliery's risk assessment process prior to finalising the secondary roof support strategy to be implemented. In considering these various options and recommendations as a part of the minesite risk assessment process the colliery decided on the support levels delineated in Figure 6.

With respect to the zone where the highest concentration of the *in situ* horizontal stress was expected (i.e. from approximately 50 m outbye of 23 C/T to 50m inbye of 24 C/T) it was decided by the colliery to install 2 x 6.1 m Bowen Cables at 2 m spacing, which were tensioned to 25 t and subsequently post-grouted. Standing support was also installed which incorporated 1 x 1m^2 Link-n-Lock at 3.5 m centres.

Figure 7 is a cross-sectional view of the primary and secondary roof support installed within the designated *Super Stress Notch* zone. However it should be noted that in relation to 23 C/T and 24 C/T, 2 x 4.1 m Superstrands at 2 m spacing with an applied pre-load of 25 t had previously been installed.

The standing support was biased toward the blockside ribline so as to create unequal roof and floor spans. The purpose being; that if any buckling of the roof or floor occurs it is more likely to occur on the pillar side as compared to the blockside thereby protecting the tailgate corner of the longwall face.

The secondary tendon support utilised by the colliery within this zone provided approximately 13 MPa of additional load-bearing capacity resulting in an overall Factor of Safety range of 1.16 to 1.35 with respect to ROF2 stability. The standing support is a passive rather active support and there to "catch" the roof should it buckle and displace to level where a roof fall may occur (if the standing support had not been installed). Therefore it does not actively prevent ROF2 from buckling and as such is not included in the FOS calculation.

RESULTANT TG 802 ROOF BEHAVIOUR

The roof behaviour associated with *Super Stress Notch* zone is typified by the Tell Tale data presented in Figure 8. This level of roof movement is considered high by the colliery as evidenced by the colliery trigger levels (also displayed on Figure 8), which are fairly typical by Australian standards as a part of a colliery's Strata Management Plan. Furthermore it is highly likely that this level of roof movement would have led to roof falls (possibly stopping longwall production) had standing support not also been installed

In this area the thickness of ROF1 is approximately 0.6 m (Figure 3) and therefore roof softening (i.e. delamination) has occurred for a considerable distance into the ROF2, certainly beyond the primary bolted interval of 1.8 m. As previously indicated, once ROF2 becomes unstable the additional load bearing capacity offered by the primary support to ROF1 would be significantly reduced and in this instance it is highly likely that significant roof softening and displacement of ROF1 would occur. The total roof displacement shown in Figure 8 is indicative of such behaviour.

In deference to the *Super Stress Notch* zone, the level of roof movement (i.e. centreline deflection) associated with TG 802 outbye of 22 C/T was generally less than 25 mm.

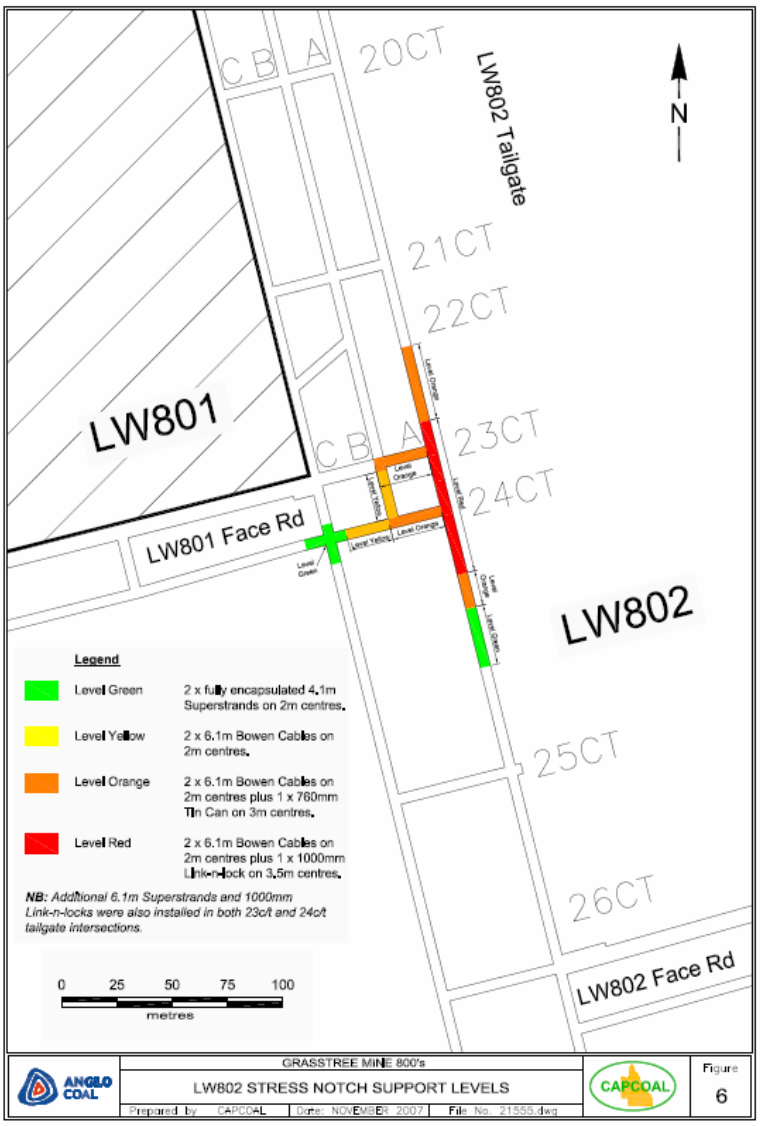

Figure 6 –LW802 stress notch support levels

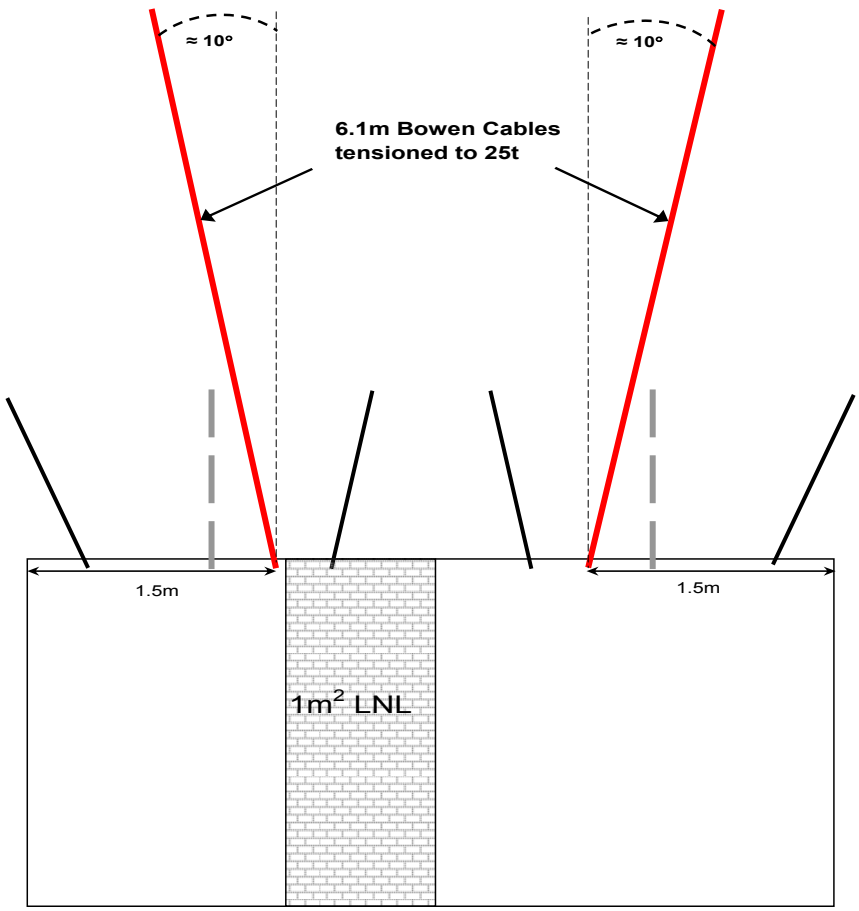

Figure 7 - Roof support installed within the super stress notch zone

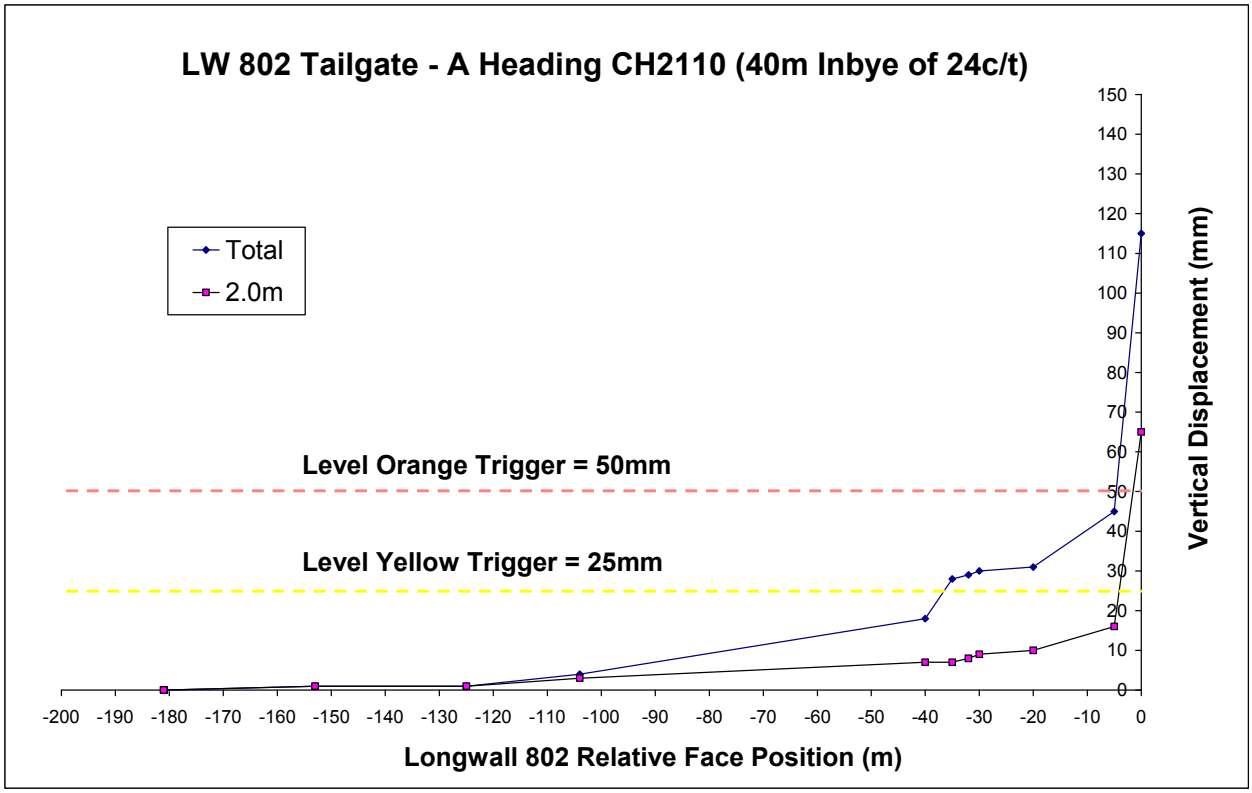

Figure 8 - Tell tale data associated with the super stress notch zone

CONCLUSIONS

The slender beam behaviour/buckling model accurately predicted the behaviour of roof unit ROF2 under super stress notch conditions when there was insufficient active reinforcement to resist the increase in horizontal stress. There are few other ground behaviour models (if any) without significant calibration that would predict such behaviour associated with 80 MPa to 100 MPa sandstone.

Although the secondary tendon support pattern (employed within the *Super Stress Notch* zone) of 2 x 6.1 m Bowen Cables every 2 m resulted in an FOS > 1, significant roof softening and displacement occurred and it is highly likely that roof falls (probably stopping longwall production) would have resulted had standing support not also been installed. It demonstrates that at this stage the resultant FOS needs to be used in a site specific comparative rather than absolute manner, which is typical in geotechnical engineering due to the various uncertainties faced.

The authors' were advised that (for a number of reasons) the colliery was not in a position to use the preferred secondary tendon support strategy of 3 x 6.1 m High Strength Tendons/Cables at 1.6 m spacing with an applied pre-load of 30t and planned to use the Bowen cables at the reduced density. However it should be noted that the standing secondary levels were not selected on an *ad hoc* basis and were actually designed utilising the ALTS approach whereby a trade-off between tendon and standing support (within limits) can be assessed in terms of a serviceable tailgate.

Overall the support strategy employed along the full length of TG 802 was successful in terms of preventing any production stopping falls and providing a serviceable tailgate for all other operational considerations without being unnecessarily conservative. The process of site characterisation, back-analysis, design, risk assessment, implementation and monitoring resulted in what would be considered a *mining success.* However what is a *mining success*?

The level of roof movement associated with the *Super Stress Notch* zone strongly suggests that this section of the tailgate was at a moderate risk with respect to a production stopping fall. Given that the analytical model or approach at this stage attempts to balance loads and therefore results in a design for little or no movement (i.e. similar to a slope stability assessment) a challenging question to ask and hopefully resolve is, *"what is an acceptable mining FOS as opposed to a civil engineering FOS"* (i.e. a colliery roadway as opposed to a highway tunnel).

Case studies such as that from Grasstree demonstrate what can be achieved in geotechnical design using established methods of structural analysis in combination with the diligent collection and use of fundamental geotechnical data. As well as being effective, such methods are transparent in their content and are therefore amenable to audit by third parties, both of which should be mandatory design requirements.

Currently the Analytical Model is being effectively utilised as a consulting tool and when used in this manner is essentially calibrated on a site by site basis. This in fact is typical of how many (if not most) analytical and numerical models are utilised by experienced geotechnical engineers. While the authors' consider the model has proven itself (on numerous occasions) when utilised in this manner, it is a significant challenge to formulate a process by which the Analytical Model can be effectively utilised industry wide by minesite Strata Control/Geotechnical Engineers. This is a primary goal of a current industry sponsored project.

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