Subsidence Management Plan

ANNEX A

Primary Roof Support Requirements

Ashton Coal
Longwall Panels 1 - 4

Subsidence Management Plan
Written Report

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Primary Roof Support Requirements for Maingate 1 and Tailgate 1, Ashton Underground Mine

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REPORT ON :

Primary Roof Support Requirements for Maingate 1 and Tailgate 1, Ashton Underground Mine

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1.0 INTRODUCTION

This report details the findings of an assessment of minimum primary roof support needs for Maingate 1 (MG1) and Tailgate 1 (TG1) at the Ashton Underground Mine. The location of MG1 and TG1 are shown in **Figure 1**. The panels are being developed on the western flank of the Camberwell Anticline, with Longwall 1 (LW1) representing the eastern-most panel on the Pikes Gully Seam. LW1 is to be developed from the Main Headings to the SSW, sub-parallel with the axis of the anticline. As seen in **Figure 1**, the depth varies from a minimum of 35m in the NNE (outbye end of TG1) to a maximum of 95m in the SSW (towards the inbye end of MG1).

The Pikes Gully Seam is 2-2.3m thick in this area, excluding the rider seam. The immediate roof of the seam is typically:

- a layer of carbonaceous mudstone / shale (0.2-0.3m thick),
- overlain by the rider seam (0.1-0.2m thick) and
- followed in turn by a second layer of carbonaceous mudstone / shale, varying in thickness from zero to 0.5m, with an average of 0.25m, see **Figure 2**.

This second carbonaceous mudstone is overlain in turn by sandstone and minor siltstone units. These sandstones are variable in nature, ranging from fine to coarse grained, bedded to massive, with zones of sub-vertical jointing. The general intention on development is cut down the rider seam and the adjacent mudstone units, such that the sandstone forms the immediate roof (ie a typical drivage height of around 2.6-2.7m). However, there are localised areas in which the mudstone above the rider seam thickens to the extent that a portion of this material is left in the roof; the impact thereof on roof support requirements has accordingly been assessed. It should also be noted that the cross-grade (ie the seam dips to the west or the right looking inbye) can result in a varying amount of mudstone across the roof (eg there can be zero cut down on the left hand side and 400mm cut down on the right); the analysis reflects the average roof composition and associated competency.

Empirical design techniques coupled to an in-house database of experience of coal mine roof support have been used to derive the primary roof support design. This has encompassed a review of the available geotechnical information, including bore core, followed by an analysis of roof competency in the relevant area. Roof behaviour in those areas of the Main Headings driven to-date has been reviewed. Key geotechnical issues and recommendations for further work are also outlined.

2.0 GEOTECHNICAL ENVIRONMENT AND SUPPORT DESIGN CONSIDERATIONS

Defining the geotechnical environment and, in particular, the strength or "competency" of the rock mass is fundamental to designing the necessary roof support measures. The relevant geotechnical issues are as follows:

- Rock material and mass properties
- Rock mass structure
- *In-situ* stress

Support requirements are a function of the *in situ* stress regime (the stresses being generally low in this shallow case), the dimensions and serviceability requirements of the excavations and the bulk strength or structural competency of the roof itself. Roof structural competency has been assessed using "CMRR" (Coal Mine Roof Rating) values derived from bore core, associated logs and test results from the study area. The relevant issues are summarised below in turn, prior to discussing roof structural competency and the implications thereof in detail in **Section 2.4**.

2.1 Rock Material and Mass Properties

Rock material and mass properties have been ascertained largely from:

- geotechnical logging and testing of the core from WML 003, 005 and 007,
- a review of geological logs from other boreholes in the study area,
- a review of the Feasibility Study, including the sample of roof rock strength parameters provided therein (**White Mining Limited, October 2000**) and
- a review of previous roof support design work conducted by Strata Engineering at Ashton (**Strata Engineering, August 2005**).

The main features of the overburden in the MG1 / TG1 area are as follows:

i) Depth to the Pikes Gully Seam varies from 35m to 95m. The depth of weathering varies between 7m and 21m in this area, averaging 13m. Based on the current geological data, the area does not include any zones that would be considered critically shallow in the context of gate road development.

ii) The carbonaceous mudstone / shale that tends to form the roof of the seam (ie above the rider seam) is ≤0.5m thick, highly moisture sensitive and very weak.

iii) Above the mudstone, the roof is dominated by fine to coarse grained, bedded to massive sandstones, with minor mudstone and siltstone beds. The sandstones are more durable and stronger, representing a favourable anchorage horizon for roof support and a more competent main roof.

iv) No regional trends with regard to roof composition and competency are readily discernible. Although, for example, local variations are evident with regard to the composition and grain size of the sandstone units (suggesting the presence of channels), no overall trend has been defined.

2.2 Rock Mass Structure

Two prominent conjugate joint sets were identified in the feasibility study, namely:

- Set 1: a NNE striking set dipping at 80^o to the ESE and
- Set 2: a WNW striking vertical set.

Occasional mid-angle (ie 50^o to vertical) joints were also identified.

Joint Set 1 has been evident in the underground workings to-date. The strike is typically very close to N-S, rather than NNE. The average spacing of observable joints is very wide (>5m), but where they are seen they tend to occur as minor swarms of 2 to 4 joints with a spacing of <0.5m. Observed dip angles range from 75^o to vertical, with joints dipping to both the ESE and WNW. Joint surfaces appear slightly rough and planar to undulating.

Joint Set 2, although prominent in the highwall, is virtually unseen underground, suggesting that these joints are tightly closed.

Set 1 is orientated at $<10^o$ to the drivage direction for these gate headings; this sub-parallel alignment is considered unfavourable, although the practical impact on roof stability during development has been minimal to-date. The limited available information (ie very largely from highwall exposure) suggests that Set 2 would be orientated at $\geq 14^o$ (typically around 25^o) to the standard (ie perpendicular) gate road cut-through direction; this alignment is close to sub-parallel (defined as $<20\ ^o$) and is considered moderately unfavourable. However, the recent change to 75^o angled cut-throughs (driven from left to right looking inbye) has improved the angle subtended with Joint Set 2 markedly, to typically 30 to 40^o.

No major geological structures (ie faults or dykes) are evident in the highwall area and the resource as a whole is expected to be largely free of significant structure. Increases in joint density should be expected towards the steep dip zone at the eastern limit of the resource area (ie around TG1).

2.3 *In-Situ* Stress

Given the depth of cover range for the area of interest of 35 to 95m, low *in-situ* horizontal and vertical stresses would be expected. Although no stress measurement has been undertaken in the project area, the feasibility study quotes an approximately NE–SW major horizontal stress orientation, which is generally consistent with experiences from Glennies Creek and other shallower operations in the Hunter Valley. Some rotation of the major horizontal stress direction with increasing depth is common. This NE-SW major stress direction is considered moderately favourable with respect to the orientation of the gate headings (ie within $\sim 37^o$).

This is generally positive, in that low horizontal stresses tend to be associated with 'static' roof behaviour, a largely self-supporting condition in which the *in-situ* horizontal stresses are insufficient to cause the roof beam to break down ('buckle' or delaminate). The roof beam remains intact and typically exhibits ≤3mm of movement, consistent with elastic displacement and negligible bed separation. Visible roof behaviour is characterised by a flat profile with CM pick marks visible and no guttering, cracking, drippers, sag or increase in bolt plate loading. A fuller definition of the main modes of roof behaviour is provided in **Appendix A**.

2.4 Roof Structural Competency and Implications for Roof Support Design

CMRR is a measure of roof ‘quality’ or structural competency for bedded roof types typical of underground coal mines. The technique was developed by the USBM (now part of NIOSH) in the United States and has been widely applied in Australia since 1996. It considers such factors as:

- The thickness of the individual roof beds.
- The shear strength properties of the bedding / planes of weakness.
- The compressive strength of the rock material, as well as its moisture sensitivity.
- The number of different roof units (ie the degree of homogeneity of the roof).
- The presence of ground water.
- The presence of a particularly strong bed, or of weaker overlying beds.

The CMRR system was initially based on field observations at surface highwalls and portals, as well as underground air crossings (overcasts) and roof falls (**Molinda and Mark, 1994**). A methodology was later developed for the assessment of CMRR from bore core (**Mark and Molinda, 1996**), to assist where underground exposures were limited or unavailable. The system was recently revised to incorporate the experiences gained over the last eight years (**Mark and Molinda, 2003**).

Essentially, CMRR is calculated by deriving Unit Ratings for the geotechnical units in the roof and then determining a weighted average for the bolted horizon. CMRR is therefore specific to roof bolt length and can change, for example, if bolt length is increased to anchor into an overlying relatively competent horizon or if a particularly incompetent unit in the immediate roof is cut down on drivage.

CMRR has been determined for the standard 1.8m roof bolt length for core from exploration boreholes WML 003, 005 and 007 and the results are summarised in **Table 1**. For record purposes the details of the CMRR calculations are also contained in **Appendix B**. The holes cover the area from the outbye end of MG1 / TG1 to close to the inbye limit. Reference has also been made to the geological logs from boreholes WML 063, 099 and 100, to provide infill data and verify the level of variability in roof composition.

It has been assumed in this analysis that the minimum working roof for drivage would be the top of the rider seam and that a maximum of 0.3m of any overlying shale / mudstone would also be cut down. **Table 1** specifically shows the effect (ie improvement in CMRR) of cutting 0.3m of the weak carbonaceous shale down from the immediate roof. In the case of WML 007, the absence of any upper shale band leaves the CMRR unchanged.

Table 1: CMRR Results

Borehole Number	Depth (m)	CMRR	
		Roof ‘as is’ (ie top of the Rider Seam)	Up to 0.3m of mudstone cut down
WML 003	41.9	58.1	62.5
WML 005	72.5	45.3	48.2
WML 007	34.9	58.1	58.1

It is noticeable that roof competency does not reduce towards the shallower area, within the depth range of 35m to 73m applicable to these boreholes. In fact, the shallower holes yielded the higher CMRR values.

To place these results in overall context, **Molinda and Mark 1994** suggests the following general categorisation of roof competency:

- CMRR <45 - Weak Roof
- CMRR = 45 to 65 - Moderate Roof
- CMRR >65 - Strong Roof

The roof in the area would be consistently classified as 'moderate'. The results from WML 003 and WML 007 are relatively high (ie 58-63); a review of previous work for Ashton (**Strata Engineering, 2005**) and of other boreholes in the area suggests that a CMRR of around 50 to 55 would be typical. In the context of Australian experience, the following refinement of the NIOSH classification is considered appropriate:

- CMRR <25 - Extremely Weak Roof
- CMRR ≥25, but<35 - Very Weak Roof
- CMRR ≥35 but <45 - Weak Roof
- CMRR ≥45 but <55 - Moderate Roof
- CMRR ≥55 but <65 - Strong Roof
- CMRR ≥65 - Very Strong Roof

This would characterise the roof as 'moderate' to 'strong' in Australian terms, noting that the Australian average CMRR for longwall operations is around 44 at the time of preparation of this report (ie 'weak' roof).

Cutting down up to 0.3m of the weak, moisture sensitive carbonaceous mudstone from the immediate roof has a positive impact on CMRR in the case of WML 003 and 005 (ie 3 to 5 CMRR percentage points, which is quite significant). Given that this upper mudstone band varies from zero to 0.5m in thickness (averaging 0.25m), cutting down up to 0.3m of this unit would leave isolated patches of mudstone ≤0.2m thick in the immediate roof. The presence of this mudstone, even at a reduced thickness and in isolated patches, will necessitate the continued use of mesh on drivage. However, given that the roof is meshed the presence of thin remnant patches of mudstone is considered unlikely to have any significant impact on the stability of the bolted roof as a whole; the main operational effect is likely to be a need to reduce the cut length in the presence of the mudstone (noting that a maximum cut distance of approximately 10m is planned).

A key practical application of these results is the relationship between CMRR, depth and the stability of extended (>6m) cuts taken during place changing ("cut and flit") operations (**Mark, 1999).** Although the mine is not contemplating the application of place changing for drivage, the successful application of this technique depends heavily on the roof behaving in a largely self-supporting fashion (such that cuts longer than 6m will tend to stand unsupported, often for extended periods prior to bolting).

Strata Engineering has defined this effectively self-supporting condition as 'static'; essentially the *in situ* horizontal stresses are insufficient to cause the roof beam to break down ('buckle' or delaminate). The roof beam remains intact and typically exhibits displacements of ≤3mm, consistent with elastic displacement. Visible roof behaviour tends to be characterised by a dry, flat profile with CM pick marks visible, no increase in roof bolt plate loading and no sag, tensile cracking or guttering. The implication to Ashton is that if roof behaviour is predicted to be static, the roof will generally be self-supporting and amenable to the application of low to moderate support densities. Although the bolts are to some extent "cosmetic", they assist in the retention of static roof behaviour, noting that buckling may develop in the longer-term and also provide some protection against localised ‘key-block’ (structurally-controlled) roof failure.

Figure 3 summarises the US information, together with data from Australian operations, as well as the Ashton borehole data. The US database derives from a survey of place changing operations requesting operators to rank their experiences regarding extended cut stability. Also seen in **Figure 3** is the discriminant equation trend line derived by **Mark 1999**, which is the line that best splits the “always stable” from the “sometimes stable / never stable” cases.

This is given by:

CMRR = 40.9 + H/30.5

where ‘H’ is the depth of cover in metres.

Effectively, the higher the CMRR, the more likely place changing is to be a success and the more likely the roof is to retain static behaviour, depending in part on the depth of cover (and the associated levels of *in situ* stress).

It can be seen from **Figure 3** that the Ashton (ie **Table 1**) data lies above the discriminant equation line. It may therefore be concluded that the MG1 / TG1 roof will generally be self-supporting on development. It is Strata Engineering’s experience that CMRRs of ≥50 are required for consistently productive place changing at shallow to moderate depths (ie to achieve an effectively “always stable” situation at depths of ≤300m). This is consistent with the results for WML 003 and 007, although the result for WML 005 would be regarded as marginal, the CMRR of 48 reflecting the increased mudstone thickness in the immediate roof in this area. In fact, the mudstone that forms the immediate roof has a Unit Rating of <35 and would tend to delaminate on drivage (ie it would be prone to ongoing skin failure or ”slabbing” in the unsupported cut). As stated previously, cutting out the mudstone (or at least the bulk thereof) is therefore preferred (and indeed is planned).

In the context of minimum primary roof support requirements for the drivage of MG1 and TG1 at Ashton, the following may be concluded:

i) With the exception of the carbonaceous mudstone that forms the skin, the roof is generally self-supporting. It follows that a relatively light primary roof bolt density will be appropriate in typical roof conditions. The main functions of this support system would be to suspend weak mudstone patches and provide reinforcement to resist potential longer-term roof buckling (eg during longwall extraction).

ii) The presence of the mudstone, even as localised remnant patches, dictates the use of mesh. The currently available mesh products have a maximum panel width of 1.7m, enabling a maximum practicable row spacing of ~1.5m (allowing for mesh overlap). Practical experience with the use of 1.7m mesh modules at Ashton to-date suggests that a minimum 4:2 pattern (ie six roof bolts per 1.5m) is required operationally, given the position of the rigs on the continuous miners.

iii) Given that these gate headings are primarily required to service the extraction of LW1 and that the re-distribution of stresses associated with longwall retreat may result in roof buckling, fully encapsulated bolts with a minimum capacity of 30t (T or X grade) are recommended. These bolts should be fitted with low friction (ie molybdenum coated) nuts and 150mm plates matched to the bolt capacity.

The issue of primary roof bolt length is not specifically addressed by the preceding analysis, although it can be implied from the anticipated static roof behaviour and the limited thickness of the mudstone (the element chiefly requiring support on drivage) that relatively short bolts would be appropriate. With regard to mining industry experiences of roof bolt length, 1.8m to 2.1m is the typical range applied in Australia, with limited applications of shorter (1.2m and 1.5m), as well as longer (2.4m and 2.7m) bolts. Cumnock Colliery successfully utilised 1.8m long roof bolts in the Lower Pikes Gully Seam for a number of years, with some application of 1.5m long bolts over the last three years of operation; longer tendons were applied in atypical conditions.

The most comprehensive study known to Strata Engineering regarding roof bolt length was undertaken by NIOSH in the USA (**Molinda et al, 2000**). An extensive database covering a wide range of roof types / competencies, support systems and experience with regard to roof stability was compiled. **Figure 4** is derived from this database and illustrates average roof bolt length versus CMRR, with the latter again divided into the commonly quoted ranges (ie weak, moderate and strong roof).

It can be seen from **Figure 4** that, as would be expected, average bolt length progressively reduces as roof competency improves, from 1.6m in weak roof to 1.4m in moderate roof and finally to 1.2m in strong roof. Note that with regard to the bolt lengths, many of the roof bolts included in the US survey are of the forged head type and effectively have no tail. Therefore, it is considered appropriate to add 0.1m to the roof bolt length, to provide a comparison with Australian conditions. Average roof bolt length in the moderate CMRR range (ie 45 to 65) is accordingly 1.5m. This CMRR range is applicable in the case of Ashton MG1 / TG1 and in fact the US database average of 51.6 for moderate roof is considered typical for the area of the mine under consideration, particularly given that some of the mudstone is cut down as planned. It is therefore concluded that the US database covers a range of circumstances applicable to Ashton and that 1.5m would be the roof bolt length typically adopted in the US in similar circumstances.

However, given the currently limited site-specific experience, particularly with regard to gate road roof behaviour during longwall extraction, a reasonably conservative approach would be considered prudent and a minimum 1.8m roof bolt length is accordingly recommended for the initial gate roads. A 1.5m long bolt remains technically feasible, but this option would require further experience and monitoring, with regard to roof behaviour both on development and during longwall extraction.

2.5 Roof Support Density

Having selected the roof bolt length, the minimum required level of support can be derived using empirical relationships between CMRR and bolt density. Two such relationships have been applied herein, namely:

- the US relationship developed by NIOSH (**Mark, 2000**) and used previously in the design of the Ashton Main Headings support (**Strata Engineering, 2005**) and

- a similar Australian relationship utilised in ALTS (**Colwell, 1998**).

The two methods focus on different roof support design aspects. The NIOSH methodology is a general primary support design tool, whereas the ALTS methodology is specific to longwall gate road primary roof support design. The ALTS methodology is more conservative, as it to an extent reflects the need to resist the stress re-distribution associated with LW extraction (although there usually still remains a need for at least some secondary support).

2.5.1 NIOSH Methodology

The NIOSH methodology derives the minimum support density using the following equation between PRSUP and CMRR:

Suggested PRSUP = 15.5 – (0.23 x CMRR) (for depths of <120m, as in this case).

Although the preceding equation is for intersections (NIOSH have not derived an equivalent relationship for headings), it is considered appropriate also for headings, given its associated conservatism, the limited site-specific experience and the strategic importance of these gate roads.

PRSUP is the required primary support capacity (kN/m), defined as:

$$PRSUP = \frac{Lb \times Nb \times C}{14.5 \times Sb \times We}$$

Where:

Lb	=	installed bolt length, allowing 100mm for the tail (1.7m)
Nb	=	number of bolts per row
C	=	bolt yield capacity (180kN yield and 300kN uniaxial tensile strength),
Sb	=	bolt row spacing (1.5m is the practicable maximum)
We	=	roadway width (5.4m)

For the Ashton CMRR values, this results in the PRSUP values presented in **Table 2**.

Table 2: NIOSH PRSUP Values Derived for Relevant Ashton Boreholes

Borehole Number	CMRR (Assumes 0.3m of mudstone cut down)	NIOSH PRSUP Values
WML 003	62.5	1.1
WML 005	48.2	4.4
WML 007	58.1	2.1
Average	**56.3**	**2.6**

It can be shown that a pattern of four 1.8m roof bolts per 1.5m (ie the minimum practicable) would equate to a PRSUP of 10.4kN/m. This results in a PRSUP value typically four times higher than that suggested by NIOSH (ie 10.4 / 2.6 = 4). Also, the PRSUP value is more than double the maximum value derived using the NIOSH method (ie 10.4 / 4.4 = 2.4). Effectively, this reflects the fact that the roof is largely self-supporting on drivage.

2.5.2 ALTS-Based Methodology

The design equation provided in ALTS is as follows:

$PSUP_R$ = 1.35 – (0.0175 x CMRR)

where $PSUP_R$ is the Recommended Primary Support Rating.

Lower and upper bounds for PSUP are also defined in ALTS as follows:

- the lower bound, $PSUP_L$ = 1.24 – (0.0175 x CMRR)
- the upper bound, $PSUP_U$ = 1.45 – (0.0175 x CMRR)

For the Ashton CMRR values, this results in the range of PSUP values presented in **Table 3**.

Table 3: ALTS PSUP Values Derived for Relevant Ashton Boreholes

Borehole Number	**CMRR (Assumes 0.3m of mudstone cut down)**	**ALTS PSUP Values**		
		Lower Bound	Recommended	Upper Bound
WML 003	62.5	0.15	0.26	0.36
WML 005	48.2	0.40	0.51	0.61
WML 007	58.1	0.22	0.33	0.43
Average	**56.3**	**0.26**	**0.37**	**0.47**

PSUP is in turn determined as follows:

$$PSUP = \frac{Lb \times Nb \times Db}{84 \times Sb \times We}$$

where:

Lb = bolt length (1.8m in this case)
Nb = number of bolts per row
Db = bolt diameter (22mm)
Sb = bolt row spacing (m)
We = roadway width (5.4m)

It follows that a range of roof support patterns and densities can be derived from the **Table 3** PSUP values. However, the lower bound and recommended values are considered to be of more relevance at the shallow depth involved, given the low *in situ* stresses and static to low level buckling roof behaviour expected on development. The relevant average PSUP values are therefore 0.26 (lower bound) and 0.37 (recommended); these are considered to define the target range for this design.

The six 1.8m long roof bolts per 1.5m pattern typically applied to-date in the Main Headings effectively meets this requirement, as it equates to a PSUP value of 0.35, very close to the recommended average and also close to the highest of the individual borehole 'lower bound' values (ie the 0.40 for WML 005).

As noted previously, the Australian ALTS-based methodology, with its LW gate road focus, produces more conservative results. Given that MG1 and TG1 are indeed required to service the first longwall panel, it would appear obvious that the ALTS-based methodology is more applicable in this case. However, the NIOSH method has relevance in those circumstances involving roadways that are mainly required for development purposes, such as the belt road (ie "off" block roadway or outer perimeter road) of TG1.

Depending on the serviceability requirements of this roadway during LW1 retreat (which may be zero), a reduced support density could be appropriate. The minimum practicable would still be the four bolts per 1.5m pattern recommended for the Main Headings, which as noted previously represents a significant increase on the density derived using the NIOSH method. The serviceability requirements of this roadway could therefore warrant further consideration.

2.6 Roof Behaviour To-Date

The outbye areas of these panels was inspected by D. Hill, accompanied by James Grebert of Ashton Coal, on Wednesday the 12th of July 2006. At that time, MG1 had been driven to 3C/T to permit installation of the conveyor drive head and TG1 was approaching 2C/T. Observed roof behaviour in the area driven to-date would typically be characterised as static to low level buckling (ie ≤10mm of displacement). This is evident from the borescope data collected in the initial entries, see **Appendix C**, and also from subsequent Tell-Tale results, which generally have indicated little or no appreciable displacement. With the exception of one Tell-Tale in the immediate portal area suggesting moderate level buckling, the Tell-Tales indicate generally static to at worst low level buckling roof behaviour.

Typically, the roof is flat and dry, with CM pick marks visible over >75% of the surface. Deterioration is largely limited to localised frittering of the skin, where remnants of mudstone remain in the immediate roof. The main roof has a high level of self-supporting capability, both at standard roadway widths and at increased spans (for example, at intersections). High angled (sub-vertical), persistent joints running with the gate headings have generally had minimal impact on roof behaviour or stability, other than often to bound localised skin falls. In conclusion, actual roof behaviour to-date is consistent with the theoretical analysis herein.

3.0 RECOMMENDED MINIMUM PRIMARY ROOF SUPPORT DESIGN

Based on the preceding assessment, the following minimum primary roof support design is recommended for MG1 and TG1:

- Mesh throughout (1.7m wide modules).
- 1.8m long 'T' or 'X' grade bolts.
- A minimum 4:2 pattern (ie six bolts per 1.5m), installed as per current practice.
- Bolts should be fitted with low friction (ie molybdenum coated) nuts and 150mm diameter / square domed plates, matched to the bolt capacity.

The primary roof bolt pattern resulting from the application of the recommendations outlined above is illustrated in **Figure 5**. This support pattern is the minimum appropriate in typical roof conditions and at normal roadway and intersection spans. A possible exception involves the potential to reduce the support density in the TG1 perimeter road (as outlined in **Section 2.5**), depending on the ongoing serviceability requirements of that specific roadway.

Additional support should be installed in 'atypical' conditions or wider roadways. The application of additional roof support should generally be under the control of a formalised Strata Management Plan, as outlined in **Section 4.0**.

4.0 STRATA MANAGEMENT

It is emphasised that these minimum primary roof bolting rules should only be applied in the context of a formal strata management process incorporating Trigger Action Response Plans (ie "TARPs") with mechanisms and provisions for:

i) categorising and defining roof behaviour (ie inspection, monitoring and mapping),

ii) confirming the adequacy and quality of the primary roof support (ie bolt testing),

iii) triggering the installation of secondary roof support, with defined cable densities,

iv) catering for the expected range of geotechnical environments and, in particular, any geological anomalies (such as joint swarms or faults),

v) catering for geometrical anomalies, such as roadways wider than 5.5m (and any intersections with diagonal spans greater than 10m),

vi) defining associated responsibilities at various levels of the workforce.

Although very little roof movement is anticipated, monitoring should be conducted to confirm the maintenance of adequate stability from drivage through to longwall extraction, noting in particular that these initial gate roads should be used as an opportunity to develop a local database of roof behaviour. Accordingly, it is recommended that:

i) 8m Tell-Tales be installed at all intersections in these gate roads.

ii) All instruments should be installed within 5m of the face, on the cut-through side of the headings and prior to breaking away the intersections.

iii) Monitoring should be integrated into the strata management process. Given that little roof displacement is expected, the first stability review trigger should be set at 10mm of movement.

Given the nature of the geotechnical environment, the following are considered to be the key visual triggers of a deterioration in roof behaviour (ie of 'atypical' conditions) that would then warrant an increase in support density and a reduction in cut distance:

- An increase in roof skin loss / slabbing prior to bolting (ie skin loss to a height of >200mm would be considered abnormal).
- Wet roof.
- Inclined roof joints (ie joints at angles of 50 to 70^{o} from the horizontal).
- An increase in the residual thickness of the mudstone in the roof to >0.3m.
- Loss of encapsulation or an inability to tension the roof bolts.
- Tensile cracking, guttering or visible sag.
- An increase in bolt plate loading.

5.0 CONCLUDING REMARKS

This report has addressed the likely stability of the Pikes Gully Seam roof on development in MG1 and TG1 and the associated minimum primary roof support requirements. It has been assessed that a roof bolt pattern based on six 1.8m long bolts per 1.5m would be adequate in typical roof conditions. Although this roof support recommendation is based mainly on an empirical methodology that focuses on primary support design for longwall gate roads, there is likely to remain a requirement to install some secondary roof support prior to LW1 retreat. Future secondary support design work should take cognisance of quantified roof behaviour on development (ie ongoing monitoring results), as well as the likely impacts of the stress re-distribution associated with LW extraction.

Some potential exists to reduce the support density in the TG1 perimeter road to, (say) a four bolts per 1.5m pattern, depending on the ongoing serviceability requirements of that specific roadway.

The recommended minimum primary support pattern is also consistent with practice to-date in the Main Headings (ie six bolts per 1.5m). This has been associated with the maintenance of an adequate level of stability, noting that roof behaviour has been typically static.

A series of recommendations have also been put forward with regard to strata management, including triggers for abnormal / ‘atypical’ conditions. This report does not, however, address the specific details of support requirements in atypical conditions (including wide drivages) or the issue of likely roof behaviour and secondary roof support requirements for the purpose of longwall retreat (as noted above).

6.0 REFERENCES

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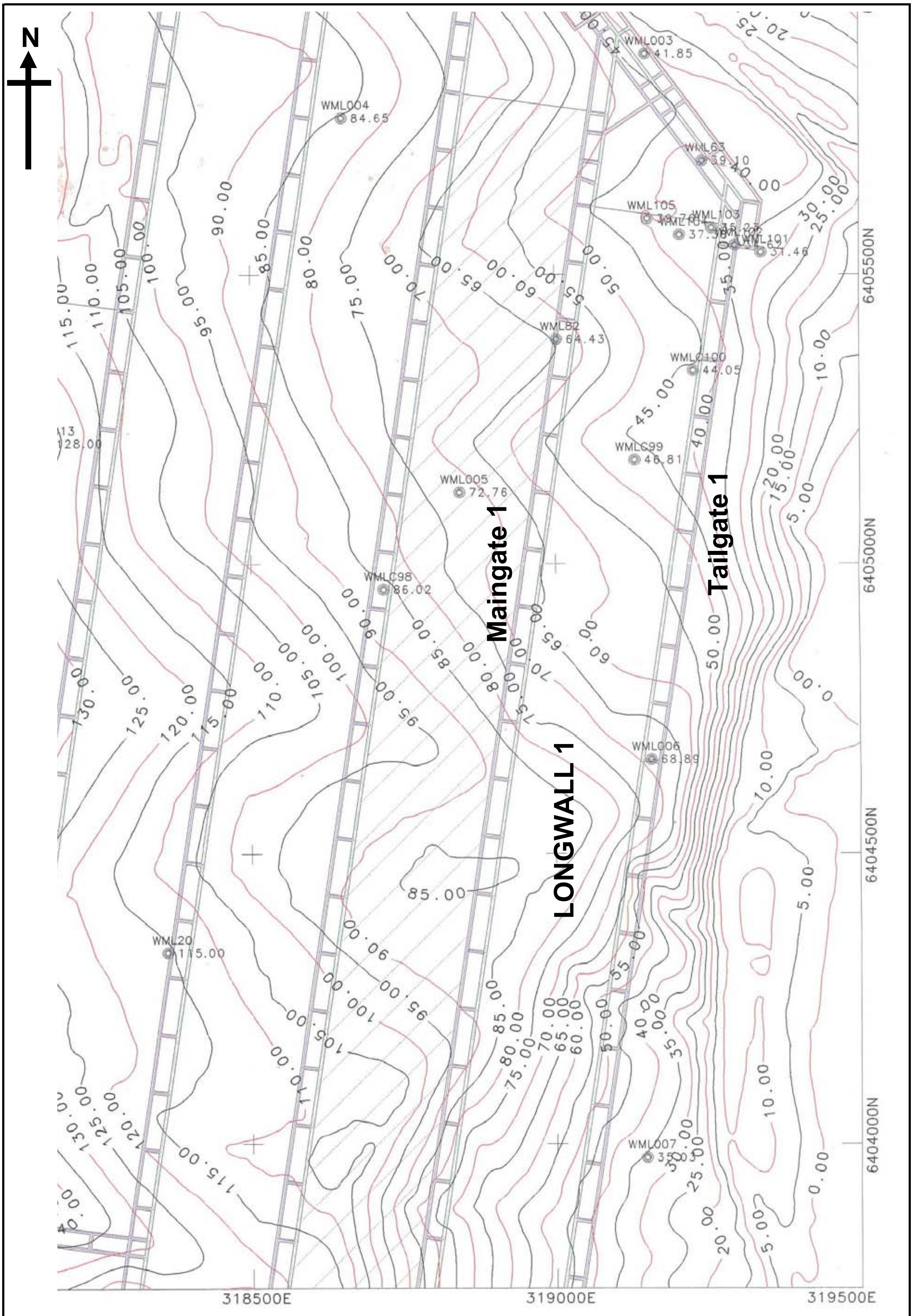
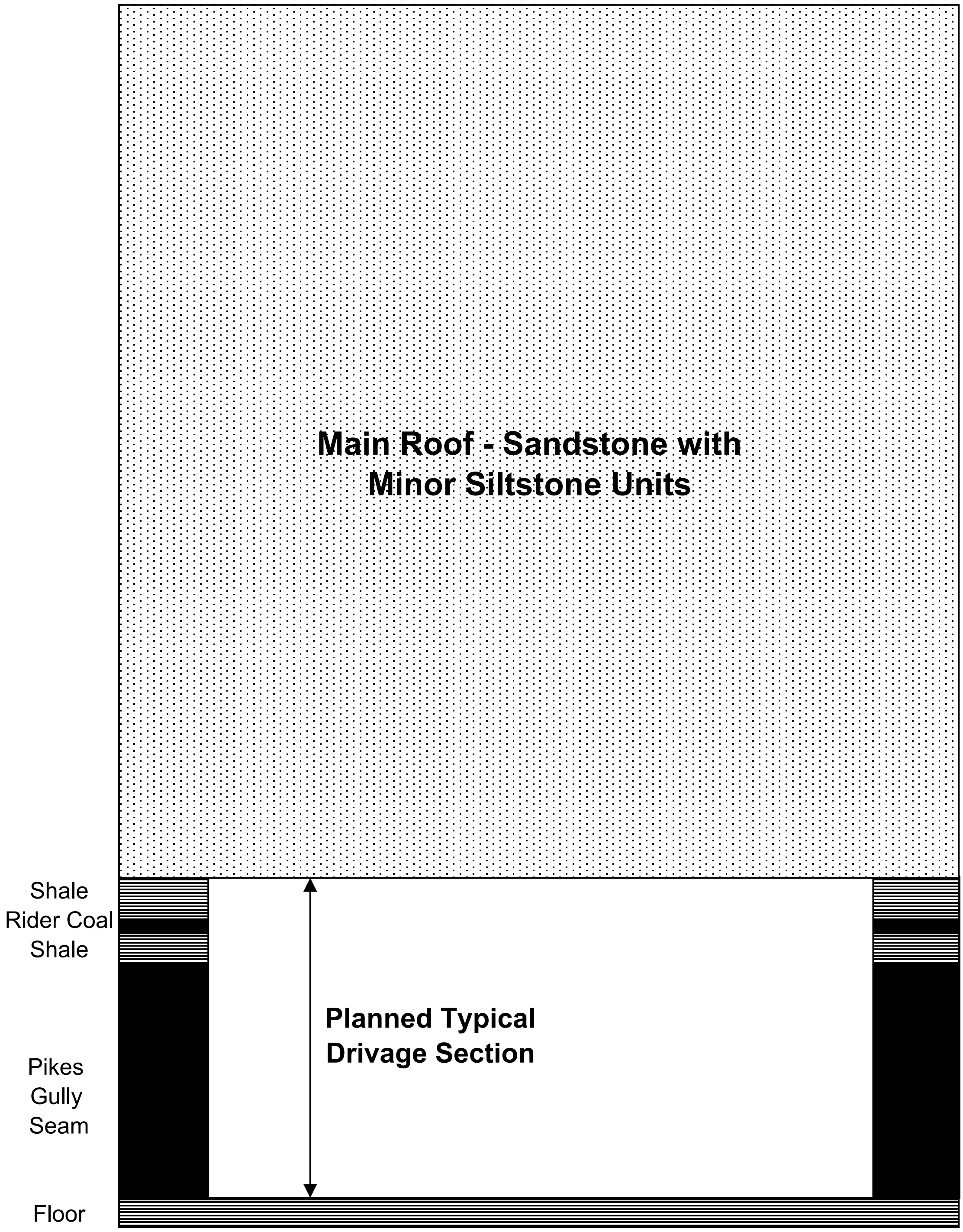
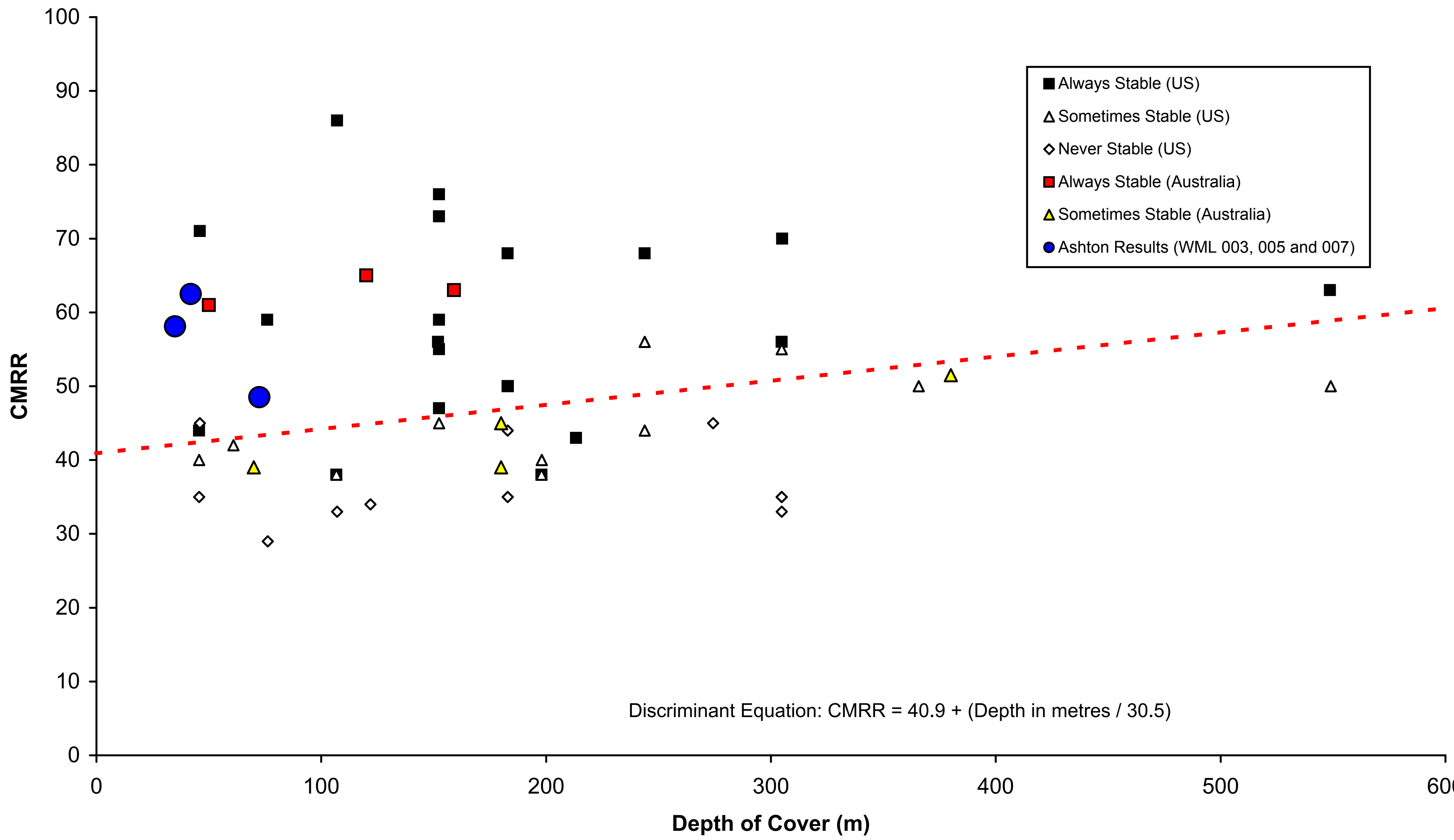


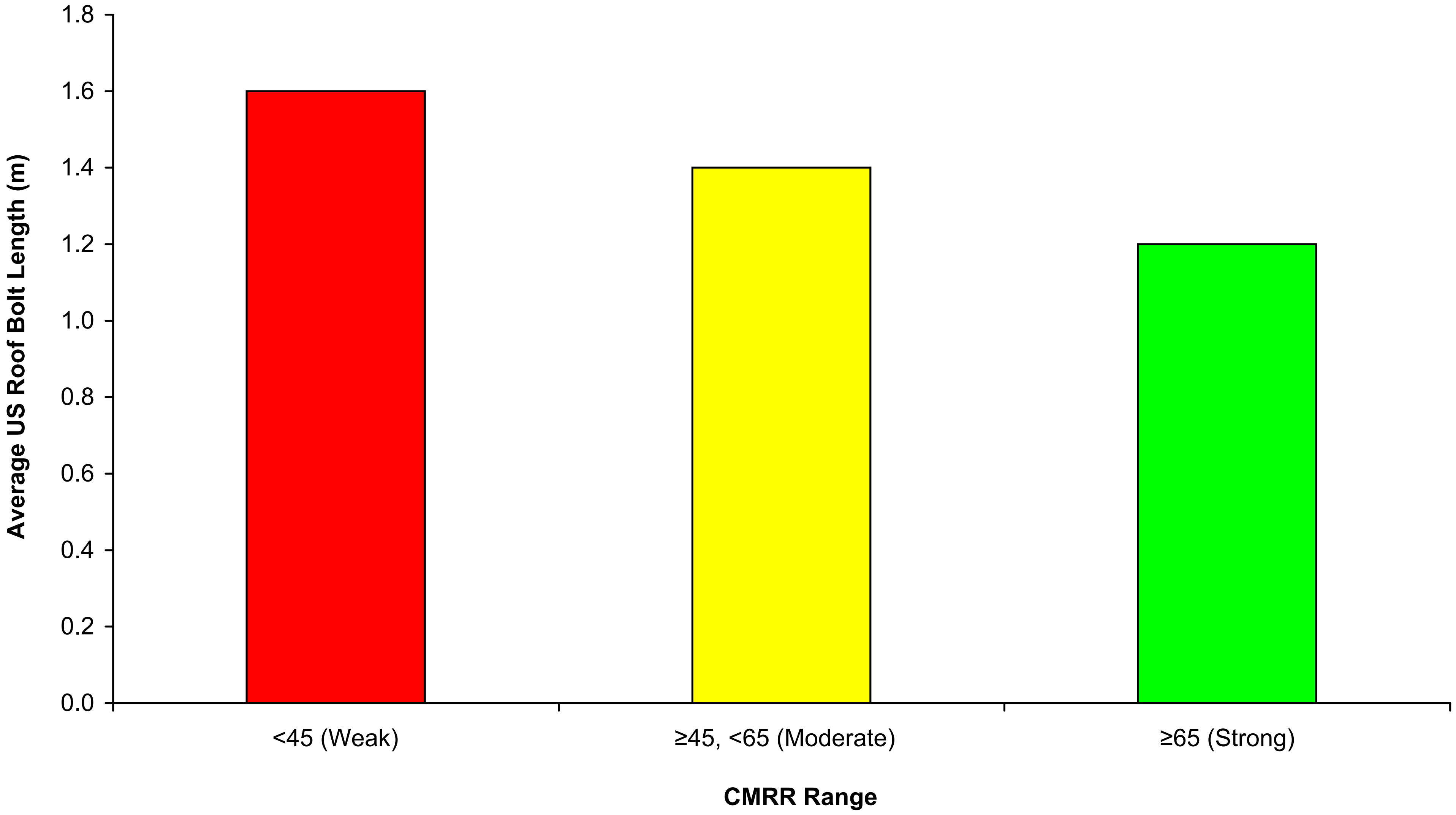
ENGINEER:	D. Hill	CLIENT:	Ashton Mine	STRATA ENGINEERING	
DRAWN:	D. Hill		R04-001-ASH-8	(Australia) Pty Ltd	
DATE:	25.07.06	TITLE:	Section of Ashton Mine Plan Showing MG1 and TG1,		FIGURE
SCALE:	NTS		plus Borehole Locations and Depth of Cover Contours		1



ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**
DRAWN:	D. Hill		R04-001-ASH-8	**(Australia) Pty Ltd**
DATE:	25.07.06	TITLE:	Schematic Showing Planned Drivage Section, Seam and	FIGURE
SCALE:	NTS		Roof Composition	2

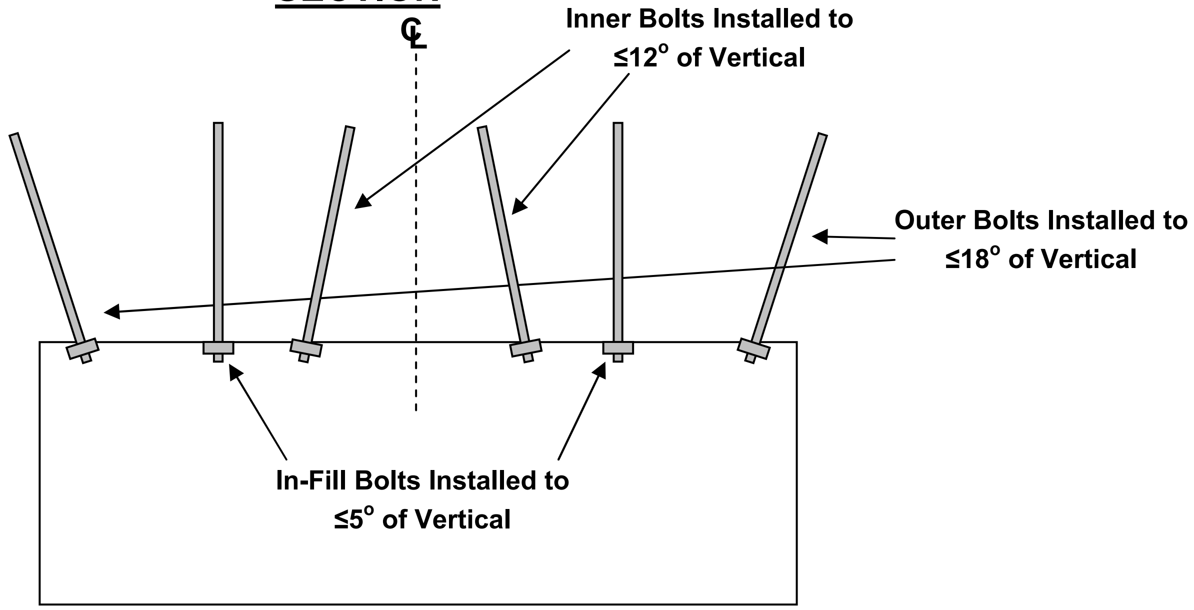


ENGINEER:	D. Hill	CLIENT:	Ashton Mine R04-001-ASH-8	**STRATA ENGINEERING (Australia) Pty Ltd**	
DRAWN:	D. Hill				
DATE:	27.07.06	TITLE:	CMRR vs Extended Cut Database showing Ashton Bore Core Results for WML 003, 005 and 007		FIGURE 3
SCALE:	NTS				



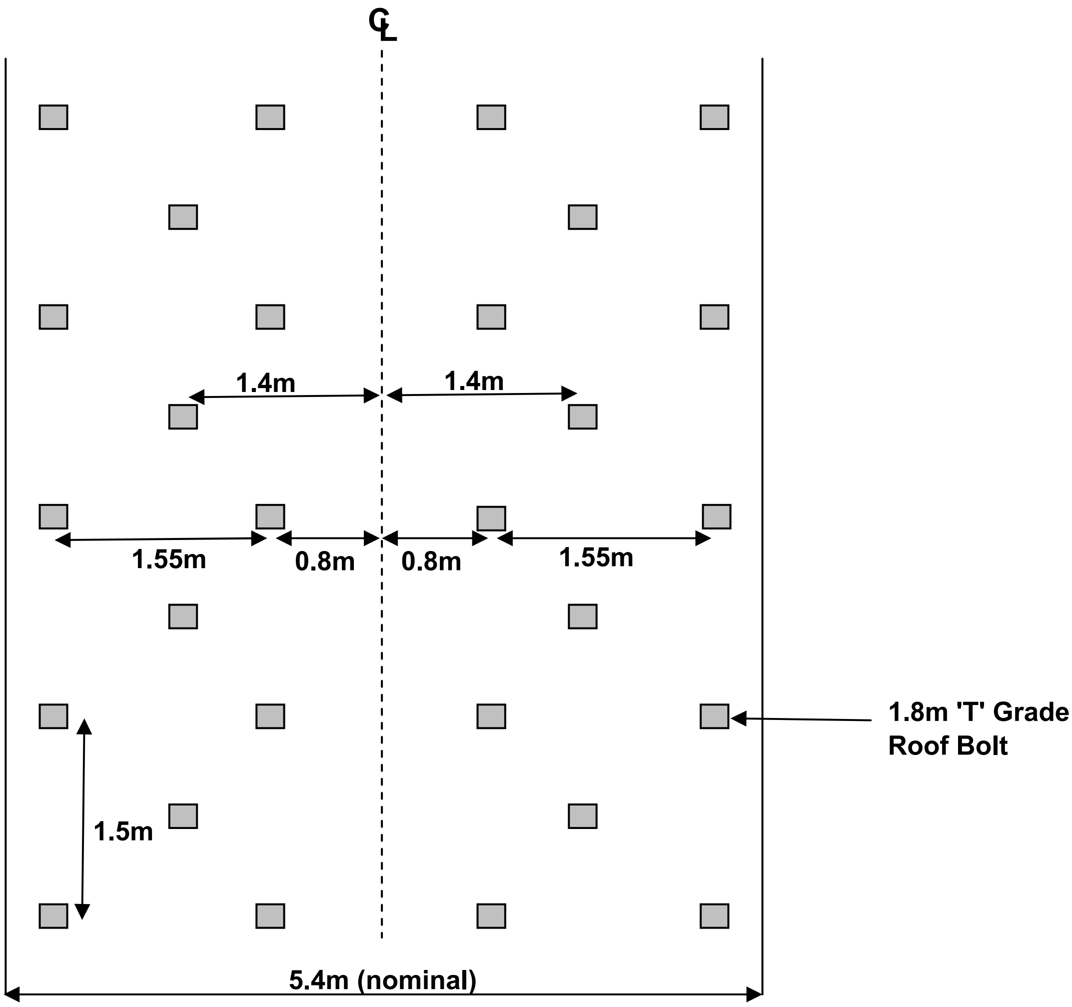
ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING (Australia) Pty Ltd**
DRAWN:	D. Hill		R04-001-ASH-8	
DATE:	27.07.06	TITLE:	Average US Roof Bolt Length versus CMRR derived from the NIOSH Database (**Molinda *et al*, 2000**)	FIGURE 4
SCALE:	NTS			

SECTION



PLAN

Roadway Centre Line



Note: A lateral tolerance of 100mm should be applied to all bolt positions

ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING (Australia) Pty Ltd**	
DRAWN:	D. Hill		R04-001-ASH-8		
DATE:	27.07.06	TITLE:	Six Bolt Pattern for Joy 12CM12		FIGURE:
SCALE:	NTS				5

APPENDIX A: STRATA ENGINEERING'S ROADWAY BEHAVIOUR AND SUPPORT DESIGN MODELS

Considering the number of undefined variables associated with *in situ* properties of rock and ground stresses, Strata Engineering believes it is inappropriate to assess roof behaviour and associated support requirements via a first principles-type theoretical analysis. Extensive use is therefore made of two models that combine basic structural engineering principles and the routine use of strata monitoring and mapping.

To aid understanding of Strata Engineering's approach to support design (the framework for this assessment), the structures of the Roadway Behaviour and Support Design Models are summarised briefly below.

A.1 Roadway Behaviour Model

Essentially, there are two distinct modes of strata behaviour in coal mine roadways, static and buckling, both of which are associated with several failure conditions that can potentially lead to a fall if not adequately controlled. These are outlined briefly below.

A.1.1 Static behaviour

This suggests that the level of stress is insufficient to cause rock mass and/or bedding plane failure. Due to the inherent competency of the strata and the level of stress (ie confinement) retained across the structure, 'static' beam behaviour typically represents the most stable condition possible in an underground coal mine. A roof exhibiting static behaviour typically undergoes ≤3mm of displacement and is illustrated schematically in **Figure A1**.

However, in static roof conditions a fall of ground can occur without any change in the state of the roof measures as a "plug" type failure. This type of fall is typically associated with:

i) low in-situ horizontal stresses,

ii) persistent mid-angled to vertical structure, aligned sub-parallel to the roadway and

iii) weak bedding planes at or above the top of the bolts.

A.1.2 Buckling behaviour

This occurs once one or more discontinuities (eg, bedding in the roof or cleat planes in the rib) undergo some degree of tensile and / or shear failure and the strata break down into a number of thin discrete units. The onset of buckling is associated with:

i) an elevation in stress magnitude,

ii) a reduction in the in-situ competency of the strata and/or

iii) inefficiencies in the support system.

Buckling behaviour is associated with >3mm of displacement (>3mm, but ≤10mm is reported as 'low level' buckling, >10mm, but ≤30mm as 'moderate level' buckling and >30mm as 'high level' buckling). Roof displacement associated with buckling is illustrated schematically in **Figure A2**.

In spite of the breakdown in the condition of the strata, a stable buckling structure can still be achieved if sufficient beam action is retained, as evidenced by the controlled deceleration in roof displacement that generally occurs some time after the onset of buckling.

Conversely, if beam action is lost, the resulting rock mass breakdown and stress reduction associated with the shortening and / or shear failure of the beam(s), increases the likelihood of a fall. Furthermore, where strata behaviour is characterised by ongoing beam breakdown and the mechanical interlock of a fractured rock mass, associated strata behaviour tends to be characterised by increased displacements and/or unpredictable trends.

In terms of engineering adequate stability in a buckling structure (either on development or longwall retreat), the design of the support system should focus on controlling displacement by retaining the inherent load-bearing capacity of any beam action naturally present in the roof. Considering the nature of strata deformation in a buckling environment, it is imperative that the support system is designed according to the following basic principles:

i) The maximisation of the support system pre-load – points to consider include the level of applied pre-load, the length of the bolts, type of resin system used and the ability of the bolts to retain high pre-loads over an extended period of time.

ii) The maximisation of the reactive resistance offered by the support system to any ensuing displacement – points to consider include load transfer and length of the bolts.

iii) The utilisation of the mechanical advantage inherent within a buckling beam(s) – points to consider include the location of the bolts across the buckling structure and the timing of support installation, as mechanical advantage reduces laterally away from the axis of maximum deflection and with ongoing roof displacement.

A.2 Roadway Support Design Model

In accordance with the basic principles of the Roadway Behaviour Model, the Design Model aims to determine firstly the type of strata behaviour, followed by any potentially appropriate actions for improving the design of the support system.

This model is divided into a number of elements which are summarised below:

i) The analysis of underground monitoring and mapping information.

ii) An assessment of the anticipated range of strata conditions.

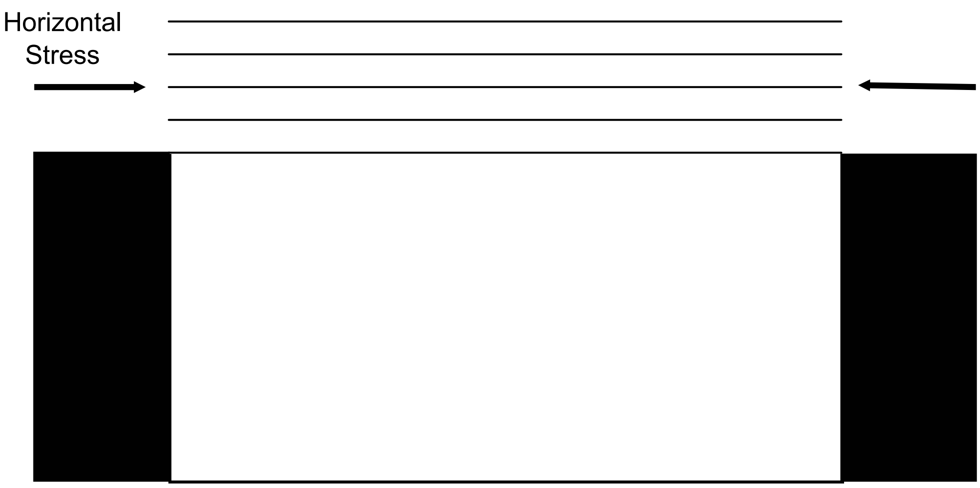
iii) An assessment of operational issues that may impact on support requirements.

iv) A comparative analysis of the structural stability of the strata and the installed effectiveness of the support design.

v) The implementation of an appropriate management plan.

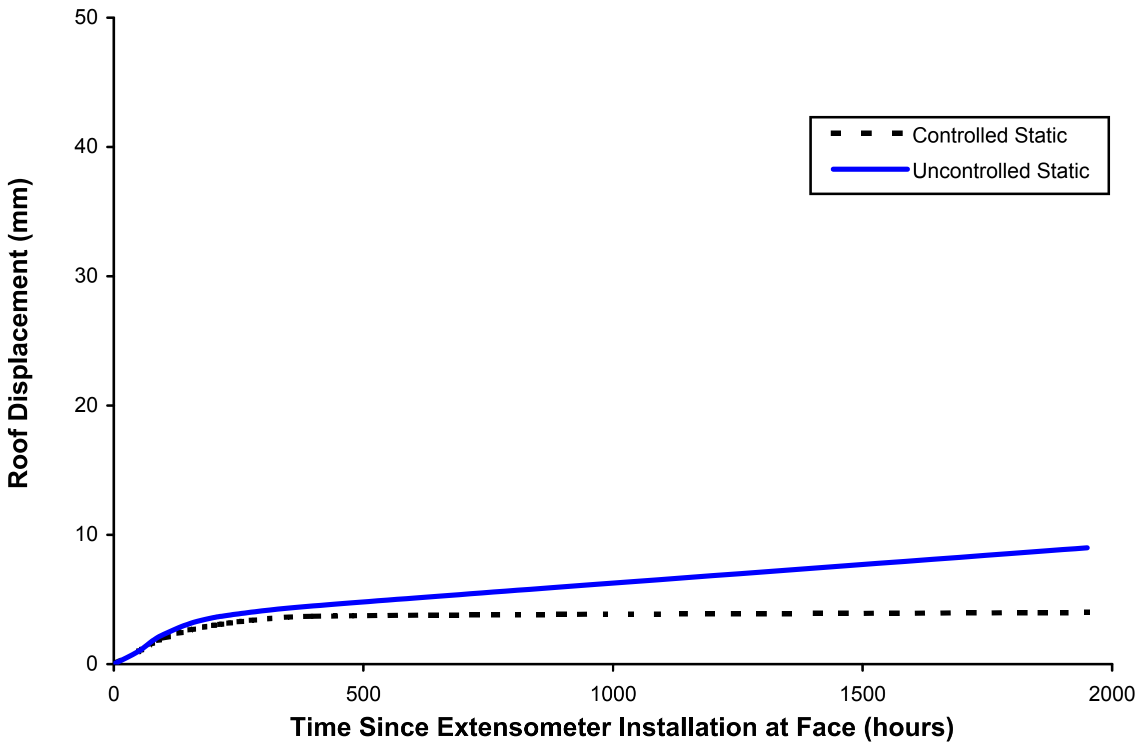
Where appropriate, each issue is applied to the matter or project under consideration.

Figure A1a



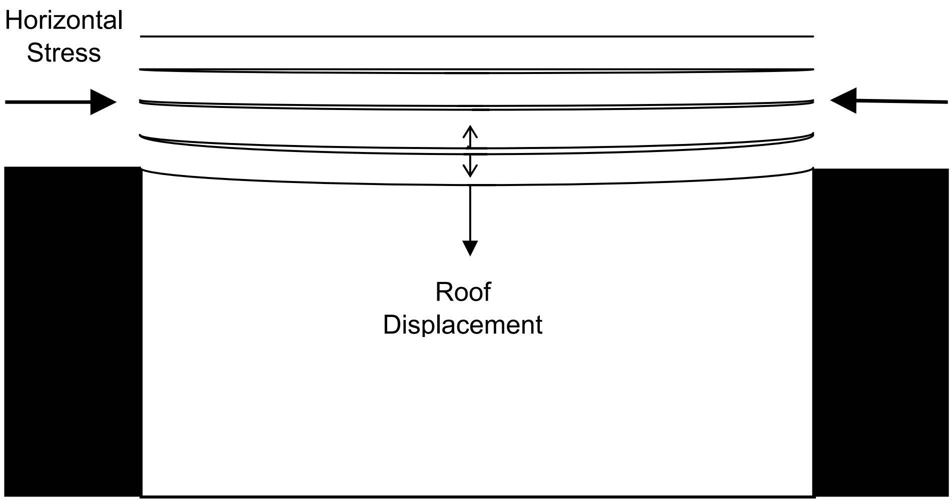
* bedding planes remain intact
* roof measures behave as one thick unit
* low displacements

Figure A1b



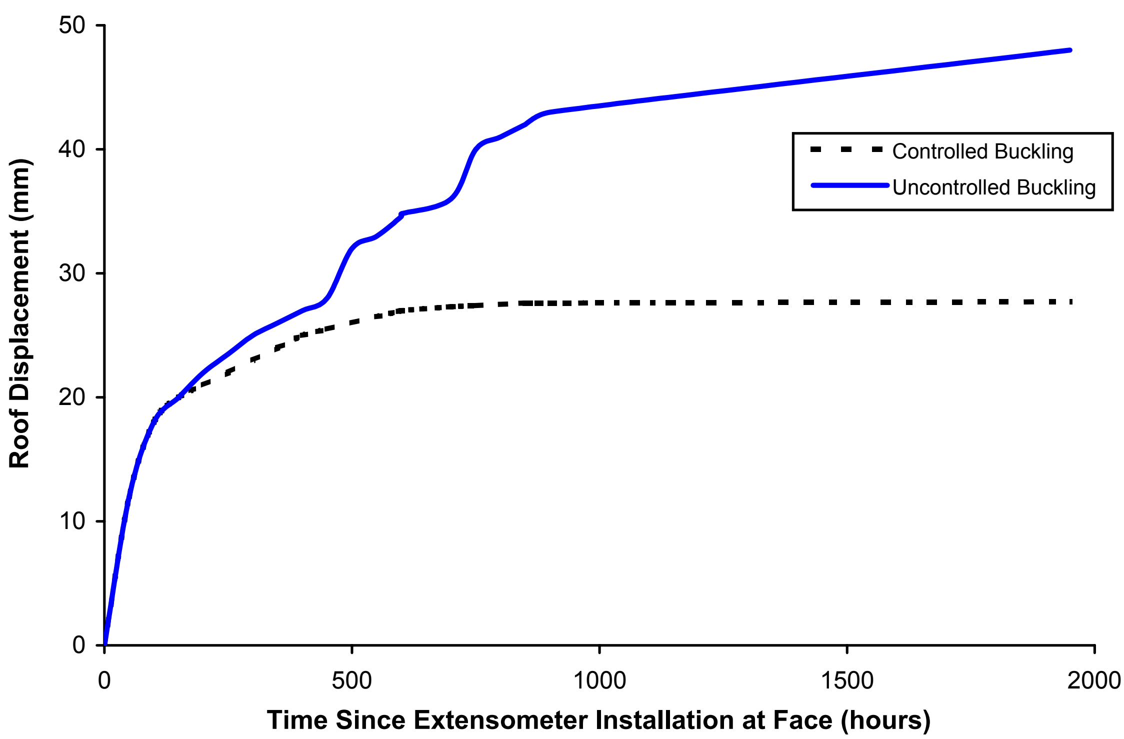
ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING (Australia) Pty Ltd**	
DRAWN:	D. Hill		R04-001-ASH-8		
DATE:	27.07.06	TITLE:	Schematic Illustration of Static Roof and Typical Time Dependent Displacement Trends		FIGURE: A1a/b
SCALE:	NTS				

Figure A2a



* tensile / shear failure occurs along bedding planes
* roof measures sub-divide into thinner discrete units
* increased displacements

Figure A2b



ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING (Australia) Pty Ltd**	
DRAWN:	D. Hill		R04-001-ASH-8		
DATE:	27.07.06	TITLE:	Schematic Illustration of Buckling Roof and Typical Time Dependent Displacement Trends	FIGURE:	A2a/b
SCALE:	NTS				

APPENDIX B: CMRR UNIT RATING RESULTS

COAL MINE ROOF RATING

UNIT RATING SHEET

CMRR = 62.5

Mudstone cut down

Colliery:	Ashton
Location:	WML 03
Date:	29.12.05
Engineer:	DH
Details:	Unit Ratings for First 8m of Roof

ENTER UNIT No: 1

Enter Unit Description: 0.22m of carbonaceous shale / mudstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	18
Enter Fracture Spacing (mm)	70
Intermediate Rating	18.7
Discontinuity Spacing Rating	20.0

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0
Diametral Strength Rating	25

3. Minimum of DSR and Diametral PLT Strength 20.0

4. Calculate UCS Rating

Enter First Axial IS_{50}	0.3
UCS (MPa)	6.3

UCS Rating 6.4

5. Unit Moisture Sensitivity -13

UNIT RATING (UR) 13.4

ENTER UNIT No: 2

Enter Unit Description: 1.88m of meduim / coarse grained, thickly bedded / massive sandstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	940
Discontinuity Spacing Rating	44.4

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	3.95
Diametral Strength Rating	102.56

3. Minimum of DSR and Diametral PLT Strength 44.4

4. Calculate UCS Rating

Enter First Axial IS_{50}	4.8
Enter Second Axial IS_{50}	2.7
UCS (MPa)	78.75

UCS Rating 18.1

5. Unit Moisture Sensitivity | 0

UNIT RATING (UR) | 62.5

ENTER UNIT No: 3

Enter Unit Description: 2.92m of fine / medium grained sandstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	95
Enter Fracture Spacing (mm)	580
Discontinuity Spacing Rating	41.7

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	2.60
Diametral Strength Rating	74.34

3. Minimum of DSR and Diametral PLT Strength | 41.7

4. Calculate UCS Rating

Enter First Axial IS_{50}	3
Enter Second Axial IS_{50}	2.3
Enter Third Axial IS_{50}	3.6
UCS (MPa)	62.3

UCS Rating | 16.0

5. Unit Moisture Sensitivity | 0

UNIT RATING (UR) | 57.7

ENTER UNIT No: 4

Enter Unit Description: 3.08m of fine grained sandstone, rare silty phases and shale bands

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	94
Enter Fracture Spacing (mm)	260
Discontinuity Spacing Rating	37.2

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	1.2
Diametral Strength Rating	45.08

3. Minimum of DSR and Diametral PLT Strength | 37.2

4. Calculate UCS Rating

Enter First Axial IS_{50}	2.2
Enter Second Axial IS_{50}	2.3
UCS (MPa)	47.25

UCS Rating | 14.0

UNIT RATING (UR) | 51.2

ROOF RATING SHEET

1. **Calculate** the Weighted Average of the Unit Ratings (RRw)

UR			**Enter** Unit Thickness		
UR No. 1	13.4	x	0	=	0
UR No. 2	62.5	x	***1.7***	=	106.2343
UR No. 3	57.7	x	***0***	=	0
UR No. 4	51.2	x	0	=	0

Enter Bolted Interval (BI) 1.7

106.2343 / 1.7 = 62.5 (RRw)

2. Calculate Strong Bed Difference (SBD)

Largest UR	62.5
Strong Bed = Largest UR	62.5
SBD	0.0

3. **Calculate** Strong Bed Adjustment	0
4. **Calculate** Unit Contact Adjustment	0
5. **Calculate** Ground Water Adjustment	0
6. **Calculate** Surcharge Adjustment	0
CMRR	62.5

COAL MINE ROOF RATING - Up to 10 Units

UNIT RATING SHEET

CMRR = 48.2

Colliery: Ashton
Location: WML 005
Date: 20/06/06
Geologist: Grant Vote **Reviewed by S. McDonald and D. Hill**
Details: Drilled 24/02/00
Logged: 20/06/06

0.3m of mst / slt cut down

ENTER UNIT No: 1

Enter Unit Description: Dark grey, laminated mudstone, carbonaceous in parts
72.48 - 72.33m (0.15m)

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	61
Enter Fracture Spacing (mm)	75
Discontinuity Spacing Rating	31.6

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.3
Diametral Strength Rating	26.27

3. Minimum of DSR and Diametral PLT Strength 26.3

4. Calculate UCS Rating

Enter First Axial IS_{50}	0.5
UCS (MPa)	10.5

UCS Rating 7.3

5. Unit Moisture Sensitivity -4

UNIT RATING (UR) 29.6

ENTER UNIT No: 2

Enter Unit Description: Grey, laminated siltstone, numerous carbonaceous planes,
occasional iron stained planes 72.33 - 72.01m (0.32m)

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	320
Discontinuity Spacing Rating	38.3

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.2
Diametral Strength Rating	25

3. Minimum of DSR and Diametral PLT Strength 25.0

4. Calculate UCS Rating

Enter First Axial IS_{50}	0.5
UCS (MPa)	10.5

UCS Rating	**7.3**
5. Unit Moisture Sensitivity	-15
UNIT RATING (UR)	**17.3**

ENTER UNIT No: **3**

Enter Unit Description: Lt grey, thinly bedded, very fine grained sandstone, numerous carbonaceous planes, rare silty planes 72.01 - 71.09m (0.92m)

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	460
Discontinuity Spacing Rating	40.4

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	1.85
Diametral Strength Rating	58.665

3. Minimum of DSR and Diametral PLT Strength **40.4**

4. Calculate UCS Rating

Enter First Axial IS_{50}	1.9
Enter Second Axial IS_{50}	3.4
UCS (MPa)	55.65

UCS Rating	**15.1**
5. Unit Moisture Sensitivity	-3
UNIT RATING (UR)	**52.5**

ENTER UNIT No: **4**

Enter Unit Description: Lt grey, med bedded, very fine grained sandstone 71.09 - 70.88m (0.21m)

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	105
Discontinuity Spacing Rating	32.0

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	3.6
Diametral Strength Rating	95.24

3. Minimum of DSR and Diametral PLT Strength **32.0**

4. Calculate UCS Rating

Enter First Axial IS_{50}	3
UCS (MPa)	63

UCS Rating	**16.1**
UNIT RATING (UR)	**48.1**

ENTER UNIT No: 5

Enter Unit Description: Brown / red, med bedded, very fine grained sandstone (sideritic)
70.88 - 70.51m (0.37m)

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	370
Discontinuity Spacing Rating	39.2

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	2.7
Diametral Strength Rating	76.43

3. Minimum of DSR and Diametral PLT Strength 39.2

4. Calculate UCS Rating

Enter First Axial IS_{50}	2.2
UCS (MPa)	46.2

UCS Rating 13.9

UNIT RATING (UR) 53.1

ENTER UNIT No: 6

Enter Unit Description: Thinly interbedded, grey siltstone and lt grey, fine grained sandstone (60:40)
70.51 - 70.31m (0.2m)

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	55	55
Enter Fracture Spacing (mm)	67	
Discontinuity Spacing Rating	30.5	

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.2
Diametral Strength Rating	25

3. Minimum of DSR and Diametral PLT Strength 25.0

4. Calculate UCS Rating

Enter First Axial IS_{50}	0.8
UCS (MPa)	16.8

UCS Rating 8.7

UNIT RATING (UR) 33.7

ROOF RATING SHEET

1. **Calculate** the Weighted Average of the Unit Ratings (RRw)

UR			**Enter** Unit Thickness				
UR No. 1	29.6	x	0	=	0		
UR No. 2	17.3	x	0.17	=	2.93913		
UR No. 3	52.5	x	0.92	=	48.30305		
UR No. 4	48.1	x	0.21	=	10.10359		
UR No. 5	53.1	x	0.37	=	19.63433		
UR No. 6	33.7	x	0.03	=	1.009872		
Enter Bolted Interval (BI)			1.7		81.98997		
					———	=	48.2 (RRw)
					1.7		

2. Calculate Strong Bed Difference (SBD)

Largest UR	53.06576
Strong Bed = Largest UR	52.5
SBD	4.3

3. **Calculate** Strong Bed Adjustment	0
4. **Calculate** Unit Contact Adjustment	0
5. **Calculate** Ground Water Adjustment	0
6. **Calculate** Surcharge Adjustment	0
CMRR	48.2

COAL MINE ROOF RATING - Up to 10 Units

UNIT RATING SHEET

CMRR = 58.1

Colliery: Ashton
Location: WML 007
Date: 20/06/06
Geologist: Grant Vote
Reviewed by S. McDonald and D. Hill
Details: Drilled March 2000

ENTER UNIT No: 1

Enter Unit Description: Thickly interbedded, lt grey, coarse grained sandstone and lt grey / variegated conglomerate (70:30) 34.89 - 34.04m (0.85m)

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	850
Discontinuity Spacing Rating	43.8

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	2.83
Diametral Strength Rating	79.21667

3. Minimum of DSR and Diametral PLT Strength 43.8

4. Calculate UCS Rating

Enter First Axial IS_{50}	2.69
Enter Second Axial IS_{50}	1.9
Enter Third Axial IS_{50}	1.9
UCS (MPa)	45.43

UCS Rating 13.8

5. Unit Moisture Sensitivity 0

UNIT RATING (UR) 57.7

ENTER UNIT No:	**2**

Enter Unit Description: Lt grey, massive, very coarse grained sandstone, pebbly in parts, rare carbonaceous wisps 34.04 - 33.19m (0.85m)

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	520
Discontinuity Spacing Rating	41.1

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	3.95
Diametral Strength Rating	102.555

3. Minimum of DSR and Diametral PLT Strength **41.1**

4. Calculate UCS Rating

Enter First Axial IS_{50}	3.4
Enter Second Axial IS_{50}	3.6
UCS (MPa)	73.5

UCS Rating **17.4**

UNIT RATING (UR) **58.5**

ROOF RATING SHEET

1. **Calculate** the Weighted Average of the Unit Ratings (RRw)

UR			**Enter** Unit Thickness				
UR No. 1	57.7	x	0.85	=	49.00945		
UR No. 2	58.5	x	0.85	=	49.70766		
Enter Bolted Interval (BI)			1.7		98.7171	=	58.1 (RRw)
					1.7		

2. Calculate Strong Bed Difference (SBD)

Largest UR	58.47959
SBD	0.4

3. **Calculate** Strong Bed Adjustment	0
4. **Calculate** Unit Contact Adjustment	0
5. **Calculate** Ground Water Adjustment	0
6. **Calculate** Surcharge Adjustment	0

CMRR 58.1

APPENDIX C: ROOF BORESCOPES IN THE MAIN HEADINGS

Borescope No. (Date)	Separation @ 0.5m Horizon (mm)	HoF (m)	HoB (m)
A1 *(02.02.06)*	*0*	*N/A*	*N/A*
A1.47 *(02.02.06)*	*0*	*N/A*	*N/A*
A1.95 *(02.02.06)*	*1*	*N/A*	*N/A*
B0.40 *(02.02.06)*	*4*	*2.9*	*0.8*
B1 *(02.02.06)*	*8*	*4.8*	*3.6*
B1.10 *(02.02.06)*	*1*	*N/A*	*N/A*
B1.48 *(09.03.06)*	*11*	*4.0*	*4.0*
C0.17 *(06.01.06)*	*8*	*1.2*	*1.2*
C0.40 *(06.01.06)*	*7*	*5.1*	*3.8*
C0.65 *(06.01.06)*	*1*	*N/A*	*N/A*
C1.48 *(09.03.06)*	*4*	*4.4*	*0.5*

Notes:
(i) N/A – insufficient displacement to determine accurately.
(ii) *Italics* – borescope undertaken >2m outbye of the face.
(iii) HoF – Height of Fracturing is the height into the roof where the displacement consistently exceeds 1.0mm.
(iv) HoB - Height Of Buckling is the height into the roof where the displacement consistently exceeds 3.0mm.