Subsidence Management Plan

ANNEX M

Roof Support for Initial Entries

Ashton Coal
Longwall Panels 1 - 4

Subsidence Management Plan
Written Report

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ASHTON COAL OPERATIONS PTY LTD

Roof Support Requirements for the Initial Entries, Ashton Underground Mine

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EXECUTIVE SUMMARY

This report details the findings of an assessment of overburden stability and underground roof support requirements for the initial entries at the Ashton underground mine. The entries pass beneath the New England Highway and the critical nature of the surface infrastructure dictates that careful consideration be given to the long-term potential for appreciable impacts or hazards involving surface movements affecting other stakeholders.

This report updates earlier studies undertaken on these issues, incorporating recent data and recognising the significant re-configuration of the mining layout following initial discussions with DPI representatives (ie the bulk of the main headings has been moved to the south-west of the road reserve, minimising development beneath the highway pavement and eliminating much of the residual risk perceived to be associated with the original layout).

The report presents the outcomes of investigations of underground support requirements for specific areas of interest using a range of appropriate empirical, analytical and numerical design techniques, including methodologies commonly applied in the civil construction field (ie tunnelling).

The outcomes of this assessment can be summarised as follows:

i) A rapid improvement in overburden competency (ie mass strength) has been identified as the three mine entries advance to the south-west from the shallow portal region, passing beneath the highway to access the main coal reserve. The roof of these mine entries would be typically categorised as self-supporting at the design spans in the inbye deeper area and would be amenable (ignoring for the moment the other stakeholder concerns that are the main focus of this study) to low to moderate roof support densities in the shallow outbye area.

ii) The potential for any long-term sinkhole development within the area of the road reserve is considered to be permanently precluded from the outset on the basis of the proposed mining geometry. Any underground instability, such as a roof fall during or subsequent to the life of the project, would have no surface impact.

iii) Furthermore, the analysis of possible modes of deformation and deterioration has facilitated the design of a roof support system that is considered to eliminate any potential for appreciable instability in the mine workings beneath the road reserve during the life of the project. The proposed support densities significantly exceed those that would be stipulated for equivalent key underground excavations in the civil field.

iv) Outside of the road reserve, the likelihood of any appreciable long-term surface impact remains limited and confined to localised areas, largely unconstrained by surface infrastructure of material concern to other stakeholders. Inbye of the road reserve, three underground intersections have been identified with very low long-term subsidence potential in the immediate vicinity of a Powertel fibre-optic cable.

v) A process of strata management has been outlined to firstly confirm the findings herein with regard to roof behaviour and support design and secondly to improve the definition of residual hazards, particularly for the underground mine workings outside the “footprint” of the road reserve with some associated residual risk.

1.0 INTRODUCTION

This report details the findings of an assessment of roof support requirements for the initial entries at the planned Ashton underground operation. This report is intended to supplement earlier Strata Engineering reports on this and related issues, but has been prepared as a "stand-alone" document. The need for this latest assessment and report has arisen from a combination of the following three principal factors:

i) There is now more data available with regard to the disposition of the Pikes Gully Seam and the quality / competency of the overburden than was the case at the time of the original studies (**Hill, August 2005**). In particular, the portal highwall has now been excavated and further drilling has been undertaken.

ii) Representatives of the DPI have expressed concerns with regard to the potential for sinkhole formation in shallow areas and as a result the main headings have been totally re-configured, such that initially only the three critical portal entries pass beneath the pavement of the New England Highway (ie the bulk of the main headings has been moved to the south-west of the road reserve). A copy of the mine plan (ie Drawing No. 01064, dated the 17th of November 2005), is provided herewith as **Appendix A**.

iii) Representatives of the DPI have expressed reservations with regard to the roof support design for the workings beneath the highway and have requested a re-assessment according to a "civil design" approach.

This assessment has therefore aimed to:

i) Update the original analysis done on the potential for sinkhole formation and present further information gained with regard to the prevalence of sinkholes above underground coal mines, including relevant outcomes of discussions between Ashton representatives and key stakeholders (ie. MSB, DPI and RTA) to-date.

ii) Comment on the additional knowledge gained with regard to seam disposition, as well as overburden composition and quality.

iii) Re-assess the roof support requirements of critical areas within the road reserve, using additional design tools commonly applied in the field of underground civil construction (ie tunnelling).

Given the critical nature of the surface infrastructure (ie primarily the New England Highway), the likelihood of measurable surface impacts must be negligible. In the context of support design, this implies that either the main headings roof must be permanently stable or that the consequences of underground instability, such as a roof fall, be demonstrated to have no impact on the surface. If an appreciable possibility of measurable surface impacts remains, then a scheme for long-term preventative or remedial measures is required. The design basis therefore differs markedly from the typical approach of designing the roof support to be consistent with the project life for the excavation.

As with the original Strata Engineering investigations of these issues, one of the main aims of this assessment has been to totally preclude the potential for surface damage or measurable impacts on other stakeholders.

2.0 LONG-TERM STABILITY OF THE ROOF OF THE PIKES GULLY MAIN HEADINGS

As stated in the introduction, the portal entries cross beneath the New England Highway, at a depth below the pavement of ≥28m. The initial section of the main headings has been removed from the highway pavement area, although one roadway remains a short distance (~15m) within the bounds of the road reserve. The likelihood of measurable surface impacts must be negligible and in the context of designing underground roof support, the criticality of the surface infrastructure demands that either the main headings must be stable in perpetuity (at least in human terms) or that the consequences of any local underground instability (ie a major roof fall) be demonstrated to have no surface impact. If any appreciable possibility of measurable impact remains, then a scheme for long-term preventative or remedial measures is required to safeguard the highway.

In practice, it is impossible to guarantee that the roof of the main headings will remain stable in perpetuity and once the mine is closed (ie beyond 20 years) it will become difficult, if not practically impossible, to adequately monitor underground stability on an ongoing basis. It is therefore concluded that the potential for the workings to impact appreciably on the surface must be precluded, preferably at the mine design stage, but at worst prior to mine closure.

Accordingly, the theoretical impact on the overlying strata of a mine roadway roof collapse (irrespective of the actual probability of such an event occurring) is assessed in the following section. The aim in this case is to “design out” the potential for appreciable surface damage wherever possible and apply adequate controls if this is not achievable in any particular area.

2.1 Caving Mechanisms Associated with Roof Falls in Mine Roadways

When a roof fall occurs in a coal mine roadway, the strata tend to form a natural ‘arch’ or ‘dome’, depending on the spanning capabilities of the beds of strata. Typically, the strata will tend to form this arch at an average angle from the vertical of ≥20^{o}, as is shown in **Figure 1**. The roof fall progresses upwards at decreasing span, until it reaches a stratum sufficiently competent to span across and ‘cap’ this arch. This span at the crown of the arch is invariably ≥1.5m. Based on these common observations, it is possible to model theoretical maximum roof fall heights in coal mine roadways of varying span, as shown in **Figure 2**. The following comments are made regarding the figure:

i) In a typical coal mine roadway ≤5.5m wide, the maximum theoretical fall height is also about 5.5m.

ii) In a typical three-way intersection with a span of ≤10m, the maximum theoretical fall height is 11-12m.

iii) In a large four-way intersection with a span of ≤12m, the maximum theoretical fall height is 14-15m.

iv) In an extremely large, four-way intersection with a span of ≤14m, the maximum theoretical fall height would be 17-18m.

These theoretical maximum fall heights are slightly more conservative than the limits of our practical experience, in that falls to a maximum height of around 5m have been observed at standard roadway widths of ≤5.5m and falls of up to 11m in intersections. More common roof fall heights are 3-4m in roadways and 5-6m in intersections. Underground monitoring of roof behaviour indicates that deformation is typically characterised by an upwards progression of deformation from the roof line (ie as total movement increases, the height into the roof of the related bed separation or fracturing also progressively increases), as illustrated in **Figure 3**. Sudden, massive collapse is rare in the absence of major geological structures (eg a fault).

From a subsidence perspective, the possibility of a surface ‘sinkhole’ developing as a result of a ‘chimney’-type roof failure above a bord and pillar intersection has been investigated extensively by **Whittaker and Reddish (1989)**, drawing on experiences from ironstone and coal workings in the UK and USA. Their model has subsequently been widely applied for predicting sinkholes above coal mine bord and pillar workings, most notably in South Africa (**Hardman, 1990; Canbulat et al, 2002**).

The model is shown in **Figure 4** and the maximum height (t) of a potential collapse above the roof line, based on a volume balance between the *in situ* rock in the “chimney” and the caved (and bulked) material can be expressed by the following equation:

$$t = \frac{4 \times h\,[0.5 + (h/(We \times \tan \Phi))]}{(k - 1) \times \pi}$$

Where:

h	=	mining height (m)
Φ	=	angle of repose of the caved rock
k	=	bulking factor
We	=	roadway width

Note that the intersection span (d) in **Figure 4** is defined as a function of the roadway width by the diameter of the largest circle that can be drawn in the area (ie as We√2). The ‘largest circle’ convention for intersection span is used throughout this report.

For the sandstones that dominate the Pikes Gully overburden, the natural angle of repose of the rock is expected to range between 35^o and 45^o, averaging 40^o. A 35^o angle of repose is considered to represent a conservative, lower-bound input value for the purpose of analysis and is consistent with values of 35-36^o adopted by the South African industry.

Observations at previous roof falls and experience with stockpiles suggest that the bulking factor would range between 1.5 and 2 for these materials. **Whittaker and Reddish** suggest that 1.5 would be a typical, realistic input value, although an extreme value would be 1.33 for strong rocks, where only minor breakage of large blocks occurs. South African experience is more conservative and suggests that bulking factors for specific materials would range from 1.5 for fine grained sandstone to 1.1-1.2 for mudstone / shale, with overall averages of 1.2-1.4 (**Canbulat et al, 2002**). The first 20m of roof in the study area at Ashton is dominated by (ie typically comprises >75%) fine grained sandstone, with some coarse grained sandstone beds noted to the south. The roof becomes siltier to the north-east, towards the portals.

Therefore, it is considered that the following constitute appropriate bulking factor input values for the purpose of sinkhole analysis:

- Maximum : 2.0
- Expected value : 1.5
- Minimum : 1.3

Based on the above material property ranges, the maximum caving height trends shown in **Figure 5** have been derived for the anticipated development heights at Ashton. As can be seen, potential caving height is highly sensitive to bulking factor and excavation height. The solid lines in **Figure 5** define the most likely range of caving heights, based on the bulking factor range of 1.3-2.0 and the lower bound (ie conservative) 35^o angle of repose.

At a development height of 3m (ie typical of the Australian industry and likely to be the upper bound of the normal development height at Ashton), a maximum intersection caving height range of between 4.9 and 16.3m is predicted, with an expected value of 9.8m. The range of 4.9 to 9.8m associated with bulking factors of 1.5 to 2 is characteristic of the range of actual observed fall heights at underground intersections. The predicted worst case value of 16.3m associated with the bulking factor of 1.3 is beyond the range of such practical experience.

At the maximum expected Ashton drivage height of 4.5m (only for key excavations, such as conveyor drive-head intersections and then outside of the road reserve area), a maximum intersection caving height range of 9.6-31.9m is predicted, with an expected value of 19.1m. These values range well beyond the practical experience of underground roof falls. In part, this reflects the fact that such development heights are uncommon within the mining industry, being generally associated with localised, critical excavations that are often relatively heavily supported (eg conveyor drive-head areas). However, the variance may also partly reflect the inherent conservatism associated with the **Whittaker and Reddish** chimney failure model, in that it ignores the natural arching ability of the strata.

The main caveats **Whittaker and Reddish** attach to the use of their model relate to;

- the need to understand the local geology, noting particular the sensitivity of the methodology to bulking factor, as outlined above,
- similarly, the need to understand the subsidence history of the area and
- the potential adverse effect of significant aquifers in the overburden.

They highlight the potential for unconsolidated wet material to flow into the mine, if the caving chimney intersects an aquifer within the overburden and quote an example from an ironstone bord and pillar mine operating at 100m depth in the UK, above which sinkholes developed 3-10 years after the collapse of the underlying intersections. They conclude that material flow associated with aquifers has the potential to prevent the natural choking due to bulking of the caved strata and go on to point to the success of building underground walls to dam potential problem areas. Aquifers, therefore, warrant specific attention.

Overall, it should be evident that the general range of caved heights based on this sinkhole model is similar to that derived previously using the trapezoidal caving angle model.

In conclusion:

- Maximum credible caving heights in roadways are of the order of 5 to 6m.
- Likely caving heights at intersections are of the order of 5 to 6m.
- Maximum credible caving heights at three-way intersections of standard (≤3m) height are of the order of 12m.
- Maximum credible caving heights at large (four-way), relatively high intersections (ie 4-4.5m) are in the range of 16 to 19m.

- Theoretical worst-case caving heights at large (four-way), 4.5m high intersections are up to 32m, according to the sinkhole model.

The implications of these caving models to surface subsidence are explored in **Section 2.2**.

2.2 Surface Subsidence Implications

The preceding analysis of underground caving mechanisms can be used to estimate the potential surface impact of intersection instability. Referring again to **Figure 4**, a sinkhole is considered possible, if the caving height (t) extends upwards to the base of the top soil (S). The soil is considered to have negligible strength or bulking capability and would collapse into the chimney. The potential depth of the sinkhole therefore depends on the thickness of the soil member and how far this soil member can fall.

In determining the thickness of the soil member, it is considered prudent to also include the thickness of weathered strata. Weathered rock has limited spanning capability and highly weathered near-surface rock or soil would tend to rill or wash into any underlying cavity. The combined thickness of soil plus weathered rock is therefore denoted as 'S' in the previous schematic, see **Figure 4**.

Initial exploration work indicated that the depth of weathering in the area is 10-11m, including around 4m of actual soil (clay). For the purpose of analysis, it has previously been assumed that the first 11m of overburden is totally incompetent. Later excavation of the portal highwall, see **Figure 6**, has confirmed:

i) The 3 to 4m expected soil thickness.

ii) The absence of significant weathering anomalies and, in particular, any channels of increased weathering; weathering of the material is indeed limited to depths of ≤11m.

iii) The absence of any significant aquifers or water-bearing layers above the seam itself; the highwall is effectively dry.

iv) The absence of any other major geological anomaly or change in the overburden profile in the portal area.

The three-dimensional caving and bulking model developed by **Whittaker and Reddish** for four-way intersections and an equivalent two-dimensional model for 5.5m roadways (tunnels) have been used to derive the sinkhole depth (ie subsidence) graphs shown in **Figures 7** and **8** respectively. Planned drivage heights for the study area are shown in **Figure 9**, noting that there are no areas of drivage at a height of >3m within the road reserve area.

Firstly, the following comments are made with regard to intersections (**Figure 7**):

i) Potential sinkhole depth varies widely, depending on bulking factor, mining height and depth of cover.

ii) At the typical maximum mining height of 3m and the expected bulking factor of 1.5, there is no potential for sinkhole formation at a depth of ≥21m. This equates to the depth at the shallowest intersection of the return airway with 1C/T, which is outside (ie outbye) of the road reserve.

iii) At the typical maximum mining height of 3m and the minimum bulking factor of 1.3, potential sinkhole subsidence reduces to zero at a depth of ≥27.2m. Within the road reserve, the shallowest 3m high intersection (which is also a relatively small three-way) lies at a depth of 34m (see **Figure 9**); there are no intersections with the potential for sinkhole formation within the roadway reserve.

iv) At the maximum mining height of 4.5m (which is limited to major conveyor drive installation areas and air-crossings, as seen in **Figure 9**), the maximum depth of cover for sinkhole subsidence is 43m. Six 4.5m high intersections, all of which are outside (ie inbye) of the roadway reserve, lie at depths of 36-42m, with some associated potential for sinkhole formation. These intersections are numbered 1 to 6 in bold red font in **Figure 9**.

Secondly, the following comments are made with regard to headings (**Figure 8**):

i) Potential sinkhole depth / subsidence again varies with the bulking factor, depth of cover and mining height, but within a much reduced range, in comparison to intersections.

ii) At the typical maximum mining height of 3m and the expected bulking factor of 1.5, there is no potential for sinkhole formation at a depth of ≥17m (this equates to the minimum depth at the shallowest (ie return airway) portal.

iii) At the typical maximum mining height of 3m and the minimum bulking factor of 1.3, there is no potential for sinkhole formation at a depth of ≥21m. As previously noted, this equates to the depth at the shallowest intersection of the return airway with 1C/T, outbye of the road reserve.

iv) The minimum depth of cover at the outbye edge of the road reserve is 24m.

v) The minimum depth of cover at the outbye edge of the road pavement is 28m.

vi) At the maximum mining height of 4.5m, inbye of the road reserve, **Figure 9**, the limiting depth of cover for sinkhole subsidence is 26m, assuming the minimum bulking factor of 1.3. The minimum actual depth of cover in these areas is 35m.

This analysis indicates that sinkhole formation (of any magnitude) is only practically possible at nine intersections, namely the three along 1C/T (ie in the area shallower than 27m outbye of the road reserve) and the six previously mentioned 4.5m high intersections, inbye of the road reserve, again see **Figure 9**.

Having arrived at this conclusion, practical experiences of, and approaches to, the sinkhole issue are explored further in the following section.

2.3 Global Experiences of Sinkholes above Bord and Pillar Coal Mine Workings

Internationally, there is a considerable body of published and anecdotal evidence with regard to sinkhole formation, including relevant guidelines. This experience is outlined below.

2.3.1 South Africa

One of the first guidelines regarding sinkhole formation above bord and pillar workings was provided thirty years ago by COMRO, the Research Organisation of the Chamber of Mines of

South Africa **(Salamon and Oravecz, 1976)**, who suggested that sinkholes are possible at a depth of 4 to 5 times the roadway width (ie ≤30m, given typical South African roadway widths of about 6m). Later work by COMRO suggests that sinkhole formation is a potential issue at depths of ≤25m **(Hardman, 1990)**.

Hill 1994 stated that:

- sinkholes are usually circular in shape, 5-10m in diameter, with vertical sides,
- erosion may cause funnelling at the surface,
- surface subsidence may vary from a few metres to the full depth of the workings,
- most sinkholes are formed above intersections and occasionally the connecting roadways collapse to form a trough-like subsidence feature and
- the deepest workings where sinkholes have occurred are 35 to 40m; beyond this depth the risk of a sinkhole is minimal.

The South African mining industry continues to apply the **Whittaker and Reddish** model and the exclusion of top soil / incompetent, weathered rock is recommended in a similar fashion to that applied in the preceding analysis **(Canbulat et al, 2002)**. Excluding 10m of top soil is suggested, as compared to the 11m excluded in the Ashton analysis. As per **Whittaker and Reddish**, the South Africans emphasise caution in the presence of water, with its potential to wash away collapsed material. The author's own observations of sinkholes in the Witbank Coalfield of South Africa, in the period from 1987 to 1993, involved depths of cover of ≤20m.

2.3.2 Australia

In recent decades, the design of shallow bord and pillar workings beneath tidal waters in the Lake Macquarie area has been largely in accordance with the "**Wardell Guidelines**" **(1975)**. In many respects, such a situation is more onerous than that involved with undermining the New England Highway at Ashton, in that:

i) There is generally an operational need to safely maximise extraction in these areas, whereas the Ashton main headings layout is focussed on developing the minimum practicable underground infrastructure to support subsequent longwall extraction of the reserves to the south and west.

ii) The level of certainty with regard to the geological environment is reduced prior to mining beneath a water body, given that drilling becomes onerous and surface lineaments cannot be reliably mapped in advance of mining.

iii) The consequences of an appreciable surface impact (such as the formation of a sinkhole) would almost certainly be significant water inflow / inrush, which could be catastrophic and impossible to remediate.

With relevance to pothole / sinkhole-type subsidence, **Wardell** states:

> *"In general terms, collapse above a height of 5t (where t is the thickness of the extracted seam) is unusual, although possible. Collapse above a height of 10t would be quite exceptional."*

This equates to 15-30m at the typical maximum height of 3m at Ashton and 22.5-45m at the maximum height of 4.5m (the latter only for localised, key excavations, outside of the road reserve).

Wardell goes on to specify a minimum of 46m of solid "bedrock" when conducting bord and pillar mining beneath tidal waters. Subsequently, some mines obtained permission to work to the 40m solid bedrock contour **(Galvin and Anderson, 1986)**, noting that mining heights are typically 2 to 3m. In a risk management context, this is considered to imply that the likelihood of sinkhole formation is practically impossible at depths of >40m (plus soil). The weathering depth in the Lake Macquarie area is typically 10-12m, effectively the same as Ashton.

Burton (1988) outlines the aims of the NSW Mine Subsidence Board (MSB) with regard to subsidence management. Pot or sinkhole-type subsidence is indicated to be a problem at a depth of <20m and a case study involving a cover depth of 10-11m in Wallsend, Newcastle is quoted. It is worth noting that Newcastle has been extensively undermined at depths of cover as low as 5m. The MSB controls any surface development in shallow areas and, in particular, limits the size of buildings that can be constructed above such workings.

Multi-storey buildings in the Newcastle area have often required remediation work on shallow underlying workings. For example, **Pells et al (1988)** describe backfilling the bord and pillar workings on the Yard Seam for the construction of the Tax Office. Cover depth was 22-23m, of which around 10m was unweathered, and the bord width was 5m (extraction was ≥60%).

Ditton and Love (1998) describe the remedial works for the Western Suburbs League Club extension in New Lambton, involving underlying Borehole Seam workings at a depth of 13 - 17m (of interest, the foundations of the adjacent existing motel had been designed to span a potential 5m diameter pothole, without remediation of the workings at that location). In this case, weathering extended to the floor of the seam. Bord widths ranged from 2.0 to 4.5m; the typical pillar width was 2.8m and the mining height was 1.8m. Some caving was defined at bord widths of 3.5-4.5m and the maximum height of caving was found to be 7.6m above the roof line (4.2 times the mining height). A remediation scheme involving an array of concrete plugs on 6m centres and partial backfilling was adopted.

In the course of discussions with relevant stakeholders aimed at finalising the layout of the Ashton main headings, it has been possible to consult with a number of local subsidence experts, most notably from the MSB and DPI. At a meeting of representatives of Ashton, DPI, MSB and RTA at the RTA Office, Newcastle, on the 21st of October, 2005, the following local examples of sinkhole formation were referred to:

i) A case history from the Lake Macquarie area, involving the formation of a pothole above first workings at a depth of 15-20m. This case involved conglomerate roof and a period of heavy rain (it is understood that the case in point involves Awaba Colliery and that this sinkhole formed on a creek line, which would be associated with an increased depth of weathering).

ii) An extreme case from Bimbadeen (ie Muswellbrook Coal) at 40m of cover, but involving weak strata and a 6m extraction height.

iii) Sinkholes at 4m depth during the construction of the West Charlestown Bypass.

It was also noted that sinkhole formation at depths of <30m is typical and that most sinkholes occur at 11-12m depth. The importance of surface water control was emphasised, noting that most sinkholes occur in undeveloped areas (ie in the absence of drainage measures).

These experiences, especially those of the representatives of the Mine Subsidence Board, who are regarded as the pre-eminent authority and repository of history regarding this issue in NSW, are entirely consistent with the analysis undertaken previously in **Sections 2.1 / 2.2**.

2.3.3 Experiences from Elsewhere

Canbulat et al 2002 also report on a subsidence-related fact finding visit to the USA and the United Kingdom and the relevant outcomes are summarised as follows.

i) In Pennsylvania, it was found that sinkholes were:
- expected at depths of <15m,
- rarer at depths of >30m and
- not expected at depths of >45m.

ii) In the United Kingdom, sinkholes were not expected at a depth of >20m.

Carter 2001 reports on an empirical "scaled span" methodology for assessing the stability of crown pillars above metalliferous mines in Canada. The underpinning database covers over 200 hard rock case histories, including 42 failures. The approach links excavation geometry (typically involving sub-vertical open stopes, 4 to 5m wide) to rock mass competency using the Norwegian Geotechnical Institute's 'Q' system of rock mass characterisation (**Barton et al, 1974**), as well as the South African Council for Scientific and Industrial Research's Rock Mass Rating (RMR) system (**Bieniawski, 1974**). Both systems have been widely applied in the civil engineering field for over 30 years.

The span, normalised with respect to the crown pillar thickness, is referred to as the "Scaled Crown Span", C_S, and is expressed as follows (for a horizontal orebody):

$$C_S = SZ^{0.5}$$

And $$Z = \frac{\gamma}{0.6t(1+S_R)}$$

where:

S	=	crown pillar span (m)
γ	=	rock mass specific gravity (t/m^3)
t	=	crown pillar thickness (m)
S_R	=	span ratio (crown pillar span / crown pillar strike length)

The Scaled Crown Span is then compared to the "Critical Span", S_C. This Critical Span is the widest stable span value for unsupported ground in the particular rock mass and is defined in terms of Q as follows:

$$S_C = 3.3 \times Q^{0.43} \times \sinh^{0.0016}(Q)$$

For the Ashton main headings, the Q value varies with the mining geometry and cover depth, according to the range summarised in **Table 1** (this is explained further in **Section 3.2)**. Note that the Q values relate to the unweathered material only and are considered to be typical, representative values for the whole of the unweathered overburden (ie they are not limited to, say, the bolted horizon).

Table 1: Representative Overburden Q Values for Ashton Headings and Intersections

Depth of Cover (m)	Portals	Headings	Intersections
<30m	4.5	9.0	3.0
≥30m	N/A	22.5	7.5

The Critical Span can then be compared to the Scaled Crown Span, for the range of depths relevant to the Ashton study area. In particular, a Factor of Safety (FoS) can be defined by the ratio of these two parameters (ie S_C / C_S). **Carter 2001** recommends a FoS of greater than two for permanent crown pillars to be of no public concern, with no monitoring required. The FoS trends are illustrated in **Figure 10** and the results are summarised in **Table 2** below.

Table 2: Scaled Span Factors of Safety for Headings and Intersections

Cover Depth (m)	Headings (width 5.4m)		Intersections							
			8m		10m		12m		14m	
	C_S	FoS	C_S	FoS	C_S	FoS	C_S	FoS	C_S	FoS
20	3.50	1.8	5.19	1.0	6.49	0.8	7.78	0.7	9.08	0.6
25	2.81	3.1	4.16	1.9	5.20	1.5	6.24	1.3	7.28	0.7
30	2.41	5.4	3.57	2.2	4.46	1.8	5.36	1.5	6.25	1.3
35	2.15	6.1	3.18	2.5	3.97	2.0	4.77	1.7	5.56	1.4
40	1.95	6.7	2.89	2.7	3.61	2.2	4.34	1.8	5.06	1.6
45			2.67	3.0	3.34	2.4	4.01	2.0	4.67	1.7
50			2.49	3.2	3.12	2.5	3.74	2.1	4.36	1.8
55			2.35	3.4	2.93	2.7	3.52	2.2	4.11	1.9
60			2.22	3.6	2.78	2.8	3.34	2.4	3.89	2.0
65			2.12	3.7	2.65	3.0	3.18	2.5	3.71	2.1

The following comments are made regarding the results:

i) For a typical 5.4m wide heading, a FoS of 2 is reached at a depth of 21m. The only areas of the three initial entries that fall into this category are the immediate portals and the return airway up to 1C/T (ie all areas outbye of the road reserve).

ii) Heading FoS reaches 2.95 at a depth of cover of 24m (ie at the outbye edge of the road reserve) and continues to increase substantially with depth (reaching 5.1 at 28m, the minimum drivage depth at the outbye edge of the road pavement).

iii) For a narrow, ≤8m wide, three-way intersection, the FoS reaches 2 at a depth of cover of 27m. As previously noted, the shallowest three-way intersection within the road reserve lies at a depth of 34m, see **Figure 9**.

iv) For a standard, ≤10m wide, three-way intersection, the FoS reaches 2 at a depth of cover of 35m. As noted above, there is one three-way intersection at a depth of 34m within the road reserve. At a width of 10m, this intersection, 12m from the edge of the reserve and 38m from the edge of the road pavement, would have a FoS of 1.96. Therefore, controlling the span is an important consideration for this particular intersection, which is planned to be 9-9.5m wide (FoS is 2.06 at 9.5m).

v) For a standard, ≤12m wide, four-way intersection, the FoS reaches 2 at a depth of cover of 46m. There are no four-way intersections at all within the road reserve in the current study area. Outside of the road reserve, there are a number of four-way intersections at a depth of <46m.

vi) For a large, ≤14m wide, four-way intersection the FoS only reaches 2 at a depth of cover of 58m.

vii) In general, the trends associated with these results correlate reasonably with the results of previous analysis. This is particularly true for headings, whereby the identical conclusion is reached, namely that there is negligible likelihood of crown failure / sinkhole formation at a depth of >21m (referring back to the results for headings in **Section 2.2**). This probably largely reflects the fact that stope spans of 4-5m are characteristic of the database underpinning the scaled span concept (ie the stope database is consistent with standard coal mine roadway widths of ≤5.5m).

viii) As the span increases, the results of the scaled span analysis are considered to become increasingly conservative, which may partly reflect a reduced number of case histories with spans of ≥10m. However, another major factor is considered to be the essential difference in geometry between the typically sub-vertical open stopes and an effectively horizontal coal mine, namely that in the case of a sub-vertical open stope there is far less potential for caving and bulking of the crown pillar material to arrest the upward propagation of any roof failure.

ix) **Carter** recognises this limitation of the methodology and recommends the use of caving and bulking analysis techniques analogous to those in **Section 2.1** to determine the likely long-term surface impact of progressive crown pillar failure.

In summary:

i) **Carter's** scaled span approach is considered to provide a useful indication of the likelihood of massive crown pillar (ie mine roof) failure and the potential for this to migrate through to surface, in the absence of sufficient caved and bulked material to arrest the upward propagation of failure.

ii) The methodology relates to the likelihood of failure in the absence of installed support (ie stability is totally reliant upon the natural spanning capability of the strata). The likelihood of failure would, of course, be greatly reduced given the installation of roof support. As it relates to the coal mine situation, therefore, the methodology is more relevant in terms of the potential for instability in the very long-term, assuming that ultimately (ie over a period of >20 years) any installed tendon support would ultimately corrode and eventually fail.

iii) The fact that the approach appears to generate credible predictions, particularly at spans of <10m, in part reflects the similarity between the characteristically 4-5m wide spans of the stopes in the underpinning database and the typical coal mine roadway width of ≤5.5m. It is also a function of the application of globally recognised, well established and broadly applicable characterisation schemes for the rock mass (ie the NGI Q and CSIR RMR systems).

iv) Caution should be exercised in using the scaled span approach for wider spans, noting particularly that the methodology does not address bulking of any failed crown pillar material, which has the potential, as previously discussed, to arrest the upward propagation of failure.

2.4 Conclusions Regarding Long-Term Roof Stability and Potential Surface Impact

The potential for sinkhole formation has been determined using well established analytical caving and bulking methodologies in **Section 2.1** and associated surface impacts have been assessed in **Section 2.2**. These analytical results have been compared to local and global experiences in **Section 2.3**. The result is considered to be a reasonably consistent set of outcomes defining the potential for sinkhole-type subsidence above the portal entries and main headings at Ashton. Having considered the sum of the analysis and the international experience, the guidelines provided in **Table 3** below are considered to represent rational limits defining the potential for long-term sinkhole formation at Ashton.

Table 3: Depth of Cover Limits to Potential Long-Term Sinkhole Formation

Excavation Type and Size	Limiting Depths of Cover for Sinkhole Formation			
	Practically Impossible		Permanently Precluded	
	Height = 3m	Height = 4.5m	Height = 3m	Height = 4.5m
5.5m Heading	20	25	22	28
8m Wide Intersection	25	34 (N/A)	29	39 (N/A)
10m Wide Intersection	28	40 (N/A)	34	47 (N/A)
12m Wide Intersection	30	43	37	52
14m Wide Intersection	31	45	39	55

Note: (N/A) = not applicable (ie no such geometry in the area of interest)

The following comments are made regarding the results in **Table 3**:

i) It is emphasised that the results summarised in **Table 3** define the limiting depths of cover for potential sinkhole formation in the Ashton geological / geotechnical environment in the **absence** of any controls, such as roof support or monitoring. The results therefore reflect the potential for sinkhole formation in the very long-term, due to gradual support decay, roof deterioration and progressive caving.

ii) During the life of the mine, roof support and monitoring are considered to further reduce the potential for a sinkhole to the point that such an occurrence is not considered credibly possible in any part of these initial entries / main headings.

iii) In **Table 3**, depth limits for long-term sinkhole formation are defined in terms of two probabilities of failure: "practically impossible" and "permanently precluded". In risk management terms "practically impossible" is considered to represent a likelihood of sinkhole formation of <1%, as defined in Strata Engineering's Risk Model, see **Appendix B**. The term "permanently precluded" is self-explanatory; it is considered to represent a situation of zero likelihood of sinkhole formation in perpetuity (at least in relevant human terms). Given the importance of the surface infrastructure above and in the vicinity of the portal entries and main headings, defining the likelihood of sinkhole formation at other depth limits (eg attempting to assess a depth range at which sinkhole formation might be considered "unlikely") would effectively be an academic exercise.

iv) For the standard, ≤5.5m wide headings, the only area in which the possibility of a sinkhole forming is not permanently precluded is between the immediate portals and 1C/T (ie at depths of <22m), well outbye of the edge of the road reserve, see **Figure 11**. It is also worth noting that in this outbye area, only the return airway is planned to be driven to a height of 3m; both the men and materials and conveyor roadways are planned to be driven to a height of 2.8m.

v) For reasonably narrow, ≤8m wide, three-way intersections, sinkhole formation is considered to be practically impossible at a depth of 25m and totally precluded at a depth of 29m. At this width, there are only two three-way intersections at which the likelihood of sinkhole formation is not considered to be precluded; both are at 1C/T, outbye of the road reserve. There are no three-way intersections planned to be >3m high in the area of interest.

vi) For regular, ≤10m wide, three-way intersections, sinkhole formation is considered to be practically impossible at a depth of 28m and totally precluded at a depth of 34m. At a width of 10m, there is one additional three-way intersection (ie apart from the two 1C/T intersections specified in point (iv) above) that only just falls into the "totally precluded" category, that being the previously mentioned case at 34m depth, as shown in **Figure 11**. As noted in the comments on the scaled span methodology, restricting the span to the planned 9-9.5m is an important factor for this particular intersection.

vii) For large, ≤12m wide, four-way intersections, the potential for sinkhole formation is highly dependent on the mining height. At the typical maximum height of 3m, the likelihood of sinkhole formation is considered to be practically impossible at a depth of 30m and totally precluded at a depth of 37m. At the increased mining height of 4.5m for key intersections (ie at conveyor drives and air-crossings), the limiting depths are 43m (practically impossible) and 52m (totally precluded). The intersections for which the likelihood of sinkhole formation is not considered to be precluded are shown in **Figure 11**. Note that there are no intersections in this category within the road reserve.

viii) For extremely large, ≤14m wide, four-way intersections, the potential for sinkhole formation is again highly dependent on the mining height. At the typical maximum height of 3m, the likelihood of sinkhole formation is considered to be practically impossible at a depth of 31m and totally precluded at a depth of 39m. At the increased mining height of 4.5m for key intersections, the limiting depths are 45m (practically impossible) and 55m (totally precluded). The intersections for which the likelihood of sinkhole formation is not considered to be precluded are again shown in **Figure 11**. Once more, note that no intersections in this category are within the road reserve.

It is concluded that the mine design permanently precludes sinkhole formation in the reserve for the New England Highway. It is also worth noting the tolerances to the planned roadway width (5.4m) and height (2.8-3.0m) criteria. Given that the minimum depth at the edge of the road reserve is 24m, versus the 22m limit stipulated in point (iv) above, sinkhole formation would be considered to remain permanently precluded at a width of ≤6m or height of ≤3.5m.

In spite of this positive finding, it is apparent from discussions with representatives of the DPI that additional controls, in the form of intensive roof support and additional monitoring will be required in the area of the road reserve. Furthermore, inbye of the road reserve, the limited areas in which sinkholes are not permanently precluded include three, 4.5m high, four-way intersections along the conveyor road, within 5m of the surface alignment of the Powertel fibre-optic cable. Depth at these three intersections ranges from 37m to 45m, such that the likelihood of long-term sinkhole formation is considered to range from unlikely to practically impossible. Although expected roof composition and behaviour are such that no instability is expected at conventional levels of roof support, these may also warrant additional support and monitoring controls, which for completeness are also addressed in **Section 3 (Support Design)** and **Section 4 (Strata Management)**.

3.0 SUPPORT DESIGN

Support design for those zones within the portal entries and initial area of the main headings identified as being of significance with regard to surface protection has been formulated with the aid of a range of appropriate design tools and methodologies, as commonly applied in civil engineering practice. These include:

- internationally recognised rock mass classification systems,
- linear / "voussoir" arch analytical techniques and
- finite element analysis.

The design outcomes have been compared to Strata Engineering's own database of industry experience, to arrive at support recommendations.

Firstly, it is important to define the areas considered to require specific support practices. On the basis of previous discussions with DPI and RTA representatives, it is suggested that the areas of interest are as shown in **Figure 12** and summarised as follows:

i) The most significant area pertains to all roadways beneath the road pavement or within a plan distance of 20m of the road pavement (to allow for the associated embankments).

ii) Next are the three 4.5m high intersections (labelled A to C) in the conveyor road, at which the potential for long-term sinkhole formation, although considered to be practically impossible for the life of the project (at conventional levels of support), cannot be permanently precluded from the outset, given the design geometry. Although outside the road reserve, the three intersections in question are within 5m of the surface alignment of the Powertel fibre-optic cable.

iii) Also included, essentially as contingency, is the three-way intersection within the road reserve, 38m inbye of the pavement. This travel road intersection (labelled D) does not require specific support measures, if the span is restricted to ≤10m (noting that the intersection is planned to be 9-9.5m wide). However, if in practice the span exceeds 10m, additional measures would be considered appropriate.

Note that for practical purposes the 20m stipulated in point (i) above effectively includes all of the road reserve on the outbye (NE) side of the pavement, see **Figure 12**. On the inbye (SW) side, the reserve extends for about 50m, well beyond the 20m limit.

In the following section the geotechnical environment is outlined, prior to deriving the specific parameters that are used as inputs for support design purposes.

3.1 Overview of the Geotechnical Environment

Along with the mine workings of interest to this report shown in **Figure 12** are the position of the New England Highway and the three cored boreholes in the vicinity of these workings (ie WML 002, 003 and 056), noting that WML 056 was drilled at the portal specifically to gather geotechnical data. An additional, non-cored, but geophysically-logged hole (WML 101) has since been drilled immediately adjacent to the road pavement, to the south-west of the return roadway (the shallowest of the three entries). This latest hole was drilled to check the depth of the seam, the weathering depth and trends in overburden composition (filling in the gap between WML 056 and WML 003). Based on the boreholes and also sections provided by the mine, the depth of the workings varies from a minimum of 17m at the portals to 41m at WML 003 and 60m at WML 002 in the NW. With particular regard to the entries beneath the road reserve, minimum depth is 24m in the SE (28m at the edge of the road pavement). The logged depth of weathering is typically 10-11m.

The Pikes Gully Seam has a total thickness of ~2.7m in the area of the main headings. The immediate roof is a layer of carbonaceous shale typically 0.2-0.3m in thickness, overlain by sandstones with minor siltstones, the latter thickening towards the portals. The sandstones are variable in nature, ranging from fine to coarse grained, bedded to massive, with zones of sub-vertical jointing.

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
- Weathering and the hydro-geological regime

The relevant issues are summarised below in turn, prior to discussing rock mass properties in detail in **Section 3.2**.

3.1.1 Rock Material and Mass Properties

Rock material and mass properties have been ascertained largely from:

- geotechnical logging and testing of the core from WML 002, 003 and 056,
- the geophysical (sonic) log from WML 101,
- a review of the Feasibility Study, including the more widely distributed sample of roof rock strength parameters provided therein (**White Mining Limited, October 2000**) and
- physical inspection of the highwall.

The main features of the overburden at the portal are as follows:

i) Depth to the PG Seam is only 17.2m at WML 56. The first 7.5m of cover is highly to completely weathered (ie clay / highly weathered mudstone to a depth of 4m, with highly weathered sandstone and siltstone units between 4m and 7.5m). The thickness of unweathered rock above the PG Seam is only 6.3m. Depth varies between 17 and 20m at the actual portal positions and given that the weathering depth is consistent at around 10-11m, a minimum of around 7m of unweathered rock is encountered at the return airway portal.

ii) As a whole, the first 7.5m of roof at the portal would be classified as weak.

iii) The carbonaceous mudstone forming the first 0.5-0.6m of the roof of the seam is highly moisture sensitive and very weak.

iv) From 0.6 to 3.3m above the seam, the roof is dominated by weak siltstone with minor sandstone units. Although more competent than the underlying mudstone, these siltstones remain moisture sensitive and would tend to weather if exposed.

v) Between 3.3 and 5.5m above the PG Seam, the roof is comprised of fine grained sandstone units, silty in parts. These sandstones are more durable and stronger, representing the most favourable anchorage horizon for any cables that might be required for underground roof support.

vi) Between 5.5 and 7.5m the roof is comprised very largely of weak siltstone units, becoming weathered above 6.3m.

vii) The siltstones are overlain in turn by a 0.9m bed of very weak, moisture sensitive, weathered carbonaceous mudstone, to a height of 8.4m above the seam. Other than a 0.3m thick sandstone band of moderate strength that caps this mudstone, the overlying strata then degrade rapidly from very to extremely weak. In general, the strata between 7.5 and 9.5m above the PG Seam would be classified as very weak, with the remaining strata to surface classified as extremely weak.

Roof composition changes markedly and rapidly, with a resulting improvement in quality, as the three entries are driven to the south-west. Comparison of borehole WML 56 with WML 003 (~250m to the SSW), see **Figure 13**, indicates that:

- the carbonaceous mudstone in the immediate roof thins from 0.6m to 0.2-0.3m,

- the remaining 9m of the immediate roof comprises bedded sandstones (although minor siltstone units are evident in other boreholes, such as WML 002 further to the west) and

- the carbonaceous mudstone seen ~8m above the seam at WML 56 is not evident at WML 003 (although appreciable shale / coal / mudstone units are encountered >17m above the PG Seam at this location).

Incorporating the information from the geophysical log for WML 101, the section shown in **Figure 14** has been derived. Key features of the overburden illustrated by this section are:

i) The weathering depth is confirmed at 11m at the road pavement. Although the weathering depth reaches 14.8m at WML 003, SSW of the area of interest, this is a local maximum. Weathering depth at WML 002, further to the west in the main headings, is only 10m.

ii) The thinning of the carbonaceous mudstone and siltstone units in the immediate roof is clearly evident. These units thin from a total of 3.3m at WML 056 to 2.4m at WML 101 and finally 0.2m at WML 003. In all cases, the lower 0.2-0.5m (ie the carbonaceous mudstone at the seam contact) is the weakest component.

iii) The thickening of the sandstone units in the main roof is equally evident, from 2.2m at WML 056 to 5.6m at WML 101 and finally 16.7m at WML 003.

The implications of the changes in overburden composition to roof competency as the portal entries advance to the SW are discussed in detail in **Section 3.2**.

3.1.2 Rock Mass Structure

Two prominent conjugate joint sets are 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.

High angle (80^o to vertical) joints are evident in the bore core and were also observed during a previous inspection of the Barrett Pit. Prior to excavation of the Arties Pit, the available data suggested that joints are typically widely spaced, but tend to occur in swarms.

Excavation of the Arties Pit highwall has exposed the overburden rock mass at the portals and largely confirms previous conclusions with regard to geological structure in the vicinity of the main headings.

Joint Set 2 is most prominent in the highwall, tending to form shallow, localised failure planes in the main sandstone unit, **Figure 15**. The strike varies from WNW to NW, within a 25^o angle of the face. Dip is to the NNE-NE and dip angle ranges from 80^o to vertical. The joint spacing varies from approximately 0.3m to 3m, but is typically in the range of 1 to 2m. Joint surfaces appear slightly rough and planar to undulating. This set is orientated at around 65^o (almost sub-perpendicular) to the drivage direction (ie Joint Set 2 is favourably orientated with regard to initial entry roof stability).

Joint Set 1 is less prominent in the highwall, being at a high angle to the face, see **Figure 15**. The strike is typically N-NNE, at a 50-60^o angle to the highwall. Dip angle ranges from 75^o to vertical, with joints dipping to both the ESE and WNW. Joint spacing varies from around 0.3 to 5m, but is typically in the range of 1 to 2m. Joint surfaces appear slightly rough and planar to undulating.

Set 1 tends to form wedges in the highwall face in combination with Set 2; given the high dip angle of both sets (ie ~75^o to vertical) and the 60-65^o slope angle, the wedges are shallow and of limited volume.

Set 1 is orientated at a moderate angle of 50-60^o to the drivage direction for the initial entries. Therefore, Set 1 is less favourably orientated with respect to initial entry roof stability than Set 2, but is not sub-parallel (ie <20^o, which would be considered unfavourable).

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.

3.1.3 *In-Situ* Stress

Given the depth of cover range for the area of interest of 17 to 50m, 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.

The Arties Seam highwall is orientated approximately NW-SE and therefore has the effect of relieving the NE-SW major horizontal stress (noting that this would in any event be very low in magnitude). Also, the location of the Arties Pit and the portal area close to the crest of the major Camberwell Anticline again suggests a low stress environment.

Horizontal stress relief due to the highwall "free face" is associated with particular zones of loosening in highwall entries. Experience indicates that two zones can generally be defined:

i) The first 6m of entry drivage from the lip tends to be characterised by pronounced dilation along any natural joints and also blasting damage (the latter particularly in the first 3m). Everything else being equal, this is the most problematical area.

ii) Up to 30m in from the highwall lip a secondary zone of joint dilation is common, which in this case would primarily affect the WNW-orientated Set 2 (ie the joint set closest to the highwall orientation).

Accordingly, specific roof support measures were previously specified and implemented for the first 30m of each portal entry, these areas being outside (outbye) the road reserve, the area of interest for this current study (see **Strata Engineering Report No. 04-001-ASH-2, Hill, August 2005**).

It is concluded that *in-situ* stresses are likely to be low, but not extremely so, in the area of interest, which is ≥70m inbye from the portals. This is generally positive, in that low 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 C**.

However, in a low stress environment there can be insufficient confinement to the rock mass and a roof fall is still possible, as illustrated schematically in **Figure 16**. The roof fall scenario is normally related to the presence of persistent mid to high angle structure (eg joints). This structure effectively forms pre-existing shear planes, along which a roof fall can occur due to the limited horizontal confinement across the planes. Such a roof fall can occur without little or no obvious change in the state of the roof measures as a plug-type fall, often with minimal visible or audible warning. In the absence of sub-vertical structure, a fall of this nature is still possible due to bending under self-weight of weak, incompetent strata (typically laminated to thinly bedded units). A conservative roof support strategy is therefore generally warranted in such shallow conditions.

3.1.4 Weathering and the Hydrogeological Regime

The logged depth of weathering at WML 56 is 10.8m and is typically 10 to 11m for the portal and main headings area as a whole. The feasibility study indicates that the seams are the major aquifers; no unusual sources of groundwater inflow to the main headings or seepage along the highwall are anticipated. There is no evidence in the highwall above the portal area of major structures (eg faults with associated zones of increased groundwater flow) or any form of 'channel' of deeper weathering, which again would locally weaken the overburden.

3.2 Rock Mass Characterisation and Implications for Roof Support Design

Minimum roof support for the initial entries could be simply and solely defined according to the CMRR-based process utilised in **Strata Engineering Report No. 04-001-ASH-1** for roof support in the main headings as a whole. However, given the criticality of these drivages and excavations, as well as the requirements of the DPI (as they are understood), a particularly conservative approach has been adopted, utilising a range of appropriate methodologies.

The quality of the roof rock masses has been assessed using three classification systems:

- The Norwegian Geotechnical Institute (NGI) Q System.
- The South African CSIR Rock Mass Rating System (RMR).
- The USBM (NIOSH) Coal Mine Roof Rating System (CMRR).

These empirical systems attach weightings to key rock mass geotechnical characteristics, for example, rock material strength and joint spacing. The sum of the weighted values provides a measure of the quality of the rock mass, the operational significance of which can then be determined from a database of industry experience. Correlation of the results for the three systems provides an opportunity to capitalise on the advantages of each, check the validity of the results and investigate any discrepancies.

3.2.1 NGI Q System

As noted previously, the Norwegian Geotechnical Institute's 'Q' system (**Barton et al, 1974**) has been widely applied in the field of civil engineering for over 30 years, with an associated experience base founded largely on long-life, critical excavations, which is directly relevant to these key initial entries and excavations. The main advantages of the NGI Q system are a link to the *in-situ* stress environment and an extensive published experience base linking Q values to support requirements. The system uses six parameters to classify the rock mass, mainly related to the joint properties, as follows:

i) Rock Quality Designation (RQD), a measure of fracture density,

ii) the number of joint sets (Jn),

iii) joint roughness (Jr),

iv) joint alteration (Ja),

v) groundwater flow (Jw)

vi) and *in situ* stress (SRF).

The parameters are related by the following equation:

$$\text{Tunnelling Quality, } Q = \frac{RQD*Jr*Jw}{Jn*Ja*SRF}$$

The inputs summarised in **Tables 4** and **5** have been derived from the exploration boreholes, observations of geological structure (jointing) at the highwall and additional data on the rock mass contained in the feasibility study. Note that these inputs and Q values are focussed on the roof support horizon and in particular the first 6m of the roof, rather than the overburden as a whole. It is also worth noting that Jn should be doubled at the immediate portal (defined as the first 15m) and tripled at intersections; the values in the tables relate to the situation for headings.

Table 4: NGI Q System Inputs and Values Outbye at the Portal (WML 056 Area)

Parameter	Carbonaceous Mudstone (0 to 0.5m into the Roof)	Siltstones and Sandstones (0.5 to 6.0m into the Roof)
RQD	84	74 – 100
Jn	6	6
Jr	1	1.5
Jw	1	1
SRF	10 (water sensitive)	2.5 (near surface)
Q Value	1.4	2.6 – 10.0
Classification	Poor	Poor to Fair

Table 5: NGI Q System Inputs and Values for the Main Headings (WML 002 / 003 Area)

Parameter	Carbonaceous Mudstone (0 to 0.3m into the Roof)	Siltstones and Sandstones (0.3 to 6.0m into the Roof)
RQD	18-32	89 - 100
Jn	6	6
Jr	1	1.5
Jw	1	1
SRF	10 (water sensitive)	1
Q Value	0.3 - 0.5	22.2 – 25.0
Classification	Very Poor	Good

It is clear from **Tables 4** and **5** that whilst the quality of the mudstone in the immediate 0.5m of roof remains poor to very poor throughout, the quality of the main roof improves markedly inbye from poor / fair at the portal to good at the main headings.

Having reviewed the log and geophysical data for WML 101, it is estimated that Q is of the order of 10 to 20 (ie fair to good) for the main roof in the area immediately outbye of the road pavement, see again the section provided in **Figure 14** and also the correlation of the sonic log to rock strength illustrated in **Figure 17** (ie UCS, after **McNally, 1988**).

Representative Q values for support design in the area of interest, recognising excavation geometry and the competency improvement with depth, are summarised in **Table 6**.

Table 6: Representative Q Values for Roof Support Design

Location	Q Value	Classification
Headings : Depth 20-30m	10.0	Fair / Good
Headings : Depth ≥30m	22.5	Good
Intersections: Depth ≥30m	7.5	Fair

An “Equivalent Dimension”, De, is also defined for the entries, as follows:

$$De = \frac{\text{Actual Span}}{\text{ESR}}$$

ESR is the Excavation Support Ratio, which is effectively an inverse Factor of Safety concept that recognises the relative importance of various excavation types. Appropriate selection of ESR is critical to the support design, particularly in this case.

Barton (1976) gives the following suggested values for ESR:

- Temporary mine openings : ESR = 3 -5
- Permanent mine openings : ESR = 1.6
- Minor road and railway tunnels : ESR = 1.3
- Major road / railway tunnels and intersections : ESR = 1.0
- Underground nuclear power stations / public facilities : ESR = 0.8

It is suggested that the following values be applied (refer also to **Figure 12**):

- Headings beneath / within 20m of the road pavement : ESR = 0.8
- The three key conveyor road intersections (A to C) : ESR = 1.0
- The travel road intersection (D, but only if span is >10m) : ESR = 1.0

Therefore, a range of De values can be defined, as summarised in **Table 7**.

Table 7: Equivalent Dimension (De) Values for the Entries

ESR	Excavation Geometry (De Values)				
	Heading (≤5.5m)	Intersections (Width)			
		(≤8m)	(≤10m)	(≤12m)	(≤14m)
0.8	6.9	N/A	N/A	N/A	N/A
1.0	N/A	8.0	10.0	12.0	14.0

Having determined the relevant Q and De values, support requirements can be determined from a nomogram presented in **Grimstad and Barton (1993)**, see **Figure 18**. The results for the headings are summarised in **Table 8**.

Table 8: Theoretical Support Requirements for Headings Beneath / Within 20m of the New England Highway Road Pavement

Heading Depth (m)	Roof Support
20 - 30	3m long bolts on a 2m grid
≥30	Spot bolts only

Depth in the area of interest is approximately equally divided between the 20-30m depth and the ≥30m category. The decrease in theoretical support requirements (ie to spot bolts only) at a depth of ≥30m reflects the finding that the roof is effectively self-supporting in the inbye area at the span, Q value and ESR involved. The spot bolts are merely for the control of the roof skin and localised minor structure.

In practice, it would be considered simpler and more appropriate to treat the whole area of interest for the three entries in accordance with the 20-30m depth category in **Table 8** (ie to take the worst-case Q value of 10). The theoretical support outcome would therefore be 3m bolts on a 2m grid in all areas beneath and within 20m of the road pavement. An equivalent or increased support outcome for the headings could be achieved using a combination of the support elements that are readily available to the local mining industry. A number of options are defined in **Table 9**, using Reinforcement Density Index ("RDI", a measure of the volume and capacity of the steel installed in the roof) as a gauge of the equivalence of these options.

Table 9: Roof Support Options for the Headings

Roof Support System	Reinforcement Density Index (RDI)	
	(RDI, MPa.m)	% of Base Case
Base Case: 3m long bolts on a 2m grid	0.22	100
Option 1: 4 x 1.8m bolts/1.5m	0.25	114
Option 2: 4 x 2.1m bolts/1.5m	0.29	132
Option 3: 4 x 2.1m bolts/1.5m; 2 x 6m Hi-Tens / 3m	0.71	323
Option 4: 6 x 2.1m bolts/m; 2 x 6m Hi-Tens / 3m	0.86	391

The following comments are made regarding the options presented in **Table 9**:

i) The only pattern that accords closely to the Q system outcome is Option 1: four 1.8m long 'T' grade roof bolts per 1.5m, with a RDI of 114% of the base case. This is the minimum roof support pattern previously recommended by Strata Engineering for the main headings as a whole (**Hill, August 2005**). In part, this gives an indication of the degree of conservatism associated with the original design, which is considered appropriate given the lack of site-specific experience at the design stage and the fact that these are life of mine roadways.

ii) It is also an indication of the limited scope to further rationalise the system, given firstly, an ongoing need to control the thin layer of mudstone in the roof with mesh and secondly, reservations regarding the use of bolts shorter than 1.8m in the absence of local experience. For example, Strata Engineering would propose the use of 1.5m bolts only given the availability of a significant body of consistent and good quality data confirming typically static roof behaviour.

iii) A support system based on 1.8m bolts would not, however, achieve anchorage in the main thickly bedded to massive sandstone above the mudstone and siltstone units in the immediate roof, as shown in the cross-section, **Figure 14** and the highwall, **Figure 15**. From the entry portals to the edge of the road reserve, the mudstone / siltstone unit thins from 3.3m to 2.4m; inbye of this point the unit thins to the typical 0.2 to 0.3m. Even recognising that around 0.2-0.5m of the weakest mudstone at the roof contact will be cut down on drivage, between 1.9 and 2.2m of interbedded siltstone and sandstone will continue to dominate the roof bolted horizon, at least in the outbye portion of the area of interest.

iv) Option 2 involves simply increasing the bolt length to 2.1m, which is the length previously recommended by Strata Engineering for areas shallower than 30m. The associated RDI of 0.29 MPa.m is 132% of the base case. However, a 2.1m bolt length would still not consistently achieve reliable anchorage in the main roof sandstone. It is probable that reasonable anchorage (≥0.3m) would be achieved in the sandstone at a depth of >30m, but not in the shallower areas.

v) Achieving the ideal of anchoring in the sandstone would therefore require either longer roof bolts (a minimum of 2.4m) or a combination of bolts and cables. At the initial entry height of 2.5-3m, a bolt length of ≥2.4m would be problematical and secondary cables are preferred.

vi) Option 3 combines four 2.1m bolts per 1.5m with a moderate cable density of two 6m Hi-TEN cables (580kN capacity) per 3m. It could be argued that shorter (4m) cables would suffice, but there are other benefits associated with the use of 6m cables, as will be discussed later. The result is a RDI of 0.71 MPa.m, 323% of the base case. At this point, the support design bears no real relation to the Q system outcome, other than that it can be shown that the theoretical average tendon length for Option 3 is 2.9m, close to the 3m specified in **Table 8**.

vii) Option 4 combines six 2.1m bolts per metre with two 6m Hi-TEN cables per 3m. The six bolts per metre pattern has been proposed by the Ashton management primarily as a means of maintaining a very high level of long-term skin control in the outbye area. It is worth noting that it is the experience of Strata Engineering that there are very few examples across the Australian coal mining industry of roof types (ignoring anomalies, such as structure zones) that cannot at least be developed at such a bolt density, without cables. The result for Option 4 is a RDI of 0.86 MPa.m, 391% of the base case. Again, the roof support design bears no relation to the Q system outcome.

viii) Of the various support alternatives presented in **Table 9**, Options 3 and 4 offer the advantage of anchorage in the main roof sandstone. This is advantageous in that, in terms of managing the residual risks associated with roof stability and support in this area of the mine, gradual time-dependent deterioration of the mudstones and siltstones in the immediate roof is regarded as the main long-term roof stability hazard.

ix) Although not specified in the Q system outcomes, the use of steel mesh for roof skin control is considered appropriate and has previously been recommended throughout all areas, given the weak mudstone band in the immediate roof.

The results of the equivalent Q system support analysis for the intersections are summarised in **Table 10**.

Table 10: Theoretical Support Requirements for the Three Key Intersections

Intersection Span (m)	Roof Support Requirements
8	2.7–3.0m long bolts on a 2.3m grid; 40mm of shotcrete
10	3.0m long bolts on a 2.3m grid; 40-50mm of shotcrete
12	4.0m long bolts on a 2.3m grid; 40-50mm of shotcrete
14	4.0–5.0m long bolts on a 2.3m grid; 50mm of shotcrete

With regard to the intersections, the theoretical support outcomes are heavily dependent on span and again, the civil-based Q system focuses on relatively long bolts and in this case also shotcrete. In practice, it is recommended that the roof support be rationalised along the lines shown in **Table 11**.

Table 11: Rationalised Support Requirements for Intersections

Intersection Span (m)	Roof Support Requirements	
	Q System	**Rationalised**
8 (3-Way)	2.7–3.0m long bolts on a 2.3m grid; 40mm of shotcrete	Four 1.8m ‘T’ grade bolts per 1.5m; mesh; two 6m Hi-TEN cables per 3m
10 (3-Way)	3.0m long bolts on a 2.3m grid; 40-50mm of shotcrete	Four 1.8m ‘T’ grade bolts per 1.5m; mesh; two 6m Hi-TEN cables per 3m
12 (4-Way)	4.0m long bolts on a 2.3m grid; 40-50mm of shotcrete	Four 2.1m ‘T’ grade bolts per 1.5m; mesh; two 6m Hi-TEN cables per 3m
14 (4-Way)	4.0–5.0m long bolts on a 2.3m grid; 50mm of shotcrete	Six 2.1m ‘T’ grade bolts per 1.5m; mesh; two 6m Hi-TEN cables per 3m

The following comments are made regarding the options presented in **Table 11**:

i) In all of the above cases, an equivalence between the tendons (expressed in terms of RDI) is achieved between the Q system results and the rationalised patterns with 1.8-2.1m bolts alone.

ii) The 6m Hi-TEN cables suggested for the rationalised patterns therefore cause the installed support capacity to significantly exceed the Q system specification (ie the RDIs increase to two to three times the Q system values).

iii) In the area of these intersections, the mudstone is expected to have thinned to around 0.3m and bolt anchorage will be almost entirely in sandstone. That being the case, the cables are not required to anchor the weak immediate roof to the strong main roof (as in the outbye area). Also, the shotcrete specified by the Q system is not regarded as critical in this inbye area and the rationalised support recommendations substitute the use of mesh.

iv) The only conceivable purpose of the cables would be to provide redundancy in the event that an unforeseen anomaly (eg a fault) is encountered. Given the level of knowledge with regard to the area and the fact that these excavations will be formed as conventional 5.5m headings prior to widening to form intersections, the likelihood of this occurring is considered practically impossible. In the absence of any other considerations, it would be regarded as appropriate to manage this potential hazard via the mine’s strata management process (eg ‘Level 1’ support would be 1.8m roof bolts plus mesh, with increased support densities specified for atypical roof behaviour).

v) With regard to three-way intersection ‘D’ on the main travel road, it is suggested that this be supported at six 2.1m bolts per 1.5m and a decision taken on the potential need for cables after the actual span is confirmed (none if ≤10m).

vi) Given the geometry of the three conveyor road intersections A to C, the simplest approach would be to assume that all of these will be 14m wide and support accordingly.

In conclusion, the NGI Q system results provide roof support designs according to a widely accepted and applied civil-based rock mass classification system. In the following section, the CSIR Rock Mass Rating System (RMR) is utilised to gain further appreciation of the roof competency and likely behaviour of the rock mass.

3.2.2 CSIR RMR System

The RMR system uses six parameters to classify a rock mass: the uniaxial compressive strength of the rock material, the RQD, the spacing, condition and orientation of the joints and the groundwater conditions. In this case the rating process is on a cumulative score basis. **Table 12** summarises the results for the Ashton roof.

Table 12: CSIR RMR Results

Area	Typical Rock Mass Rating (RMR)					
	Immediate Roof Mudstone (0.2-0.5m)		Interbedded Siltstone and Sandstone		Main Roof Sandstone	
	RMR	Class	RMR	Class	RMR	Class
Outbye (WML 056)	33-48	Poor / Fair	52 - 64	Fair / Good	70 - 79	Good
Inbye (WML 002 / 003)	33 - 38	Poor	N/A	N/A	79 - 82	Good / V. Good

Figure 19 summarises these results in the context of stand-up time for the unsupported roof at the heading span of ≤5.5m. It can be seen from the figure that the 0.2 to 0.5m of mudstone at the roof contact (RMR ~35) would tend to fail immediately; in practice a proportion of this material is cut down at the face.

The overlying interbedded siltstones and sandstones encountered in the outbye area (RMR ~55) have limited self-supporting ability (theoretically these units would stand unsupported for around one month at a span of 5.5m), but would tend towards immediate collapse at the wider spans associated with intersections.

The main roof sandstone, with an RMR range of 70 to 82, would tend to stand unsupported indefinitely, particularly in the inbye area and even at the wider intersection spans.

The RMR results are consistent with the previous Q system outcomes, which suggested that the main sandstone requires minimal support (ie spot bolts only) at spans of ≤5.5m and only moderate support at wider spans, even for key excavations (ie low ESR values).

Roof support in the inbye area of the main headings should therefore be focussed largely on managing:

- potential skin deterioration associated with any mudstone that remains in the immediate roof and
- any atypical conditions or anomalies (eg faults) that might be encountered.

In the following section the roof support outcomes derived from the NIOSH (formerly USBM) CMRR system are presented. CMRR was derived from the RMR system, specifically as a tool for assessing and managing coal mine ground control issues.

3.2.3 NIOSH CMRR System

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.

The CMRR results for WML 002, 003 and 056 are summarised in **Table 13**, highlighting the following:

- The variance associated with bolt lengths of 1.8m and 2.1m.

- The effect of cutting 0.2m of the weak carbonaceous mudstone down from the immediate roof.

Figures 20 to **22** illustrate the overall Unit Rating profiles and logs for the three bore holes, covering the first 8m of roof in the cases of WML 002 / 003 and 10m for WML 056 (the portal hole). The details of the CMRR unit ratings are contained in **Appendix D**.

Table 13: CMRR Results

Borehole Number	Depth (m)	CMRR			
		Roof 'as is'		0.2m of mudstone cut down	
		1.8m Bolts	2.1m Bolts	1.8m Bolts	2.1m Bolts
WML 002	60	46.5	47.4	51.8	50.1
WML 003	41	58.1	59.1	61.9	61.8
WML 056	17	36.6	37.2	38.7	39.0

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 implications of this categorisation to Ashton are:

i) The roof in the shallow portal area would be classified as 'weak' (ie CMRR is in the range of 36.6 to 39.0).

ii) Elsewhere, at depths of 40-60m, the CMRR values of 46.5 to 61.9 are consistent with 'moderate' roof (ie the roof quality improves markedly away from the shallow portal area).

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 classification is employed in the Unit Rating profiles shown in **Figures 20** to **22**. Note that in **Figure 22** (WML 056), the bottom 90mm of the mudstone at the portal is extremely weak (a Unit Rating of 14.4) and has been omitted from the profile, as this material is cut down on drivage.

The following additional comments are made regarding the results:

i) Cutting down at least a portion of the weak and moisture sensitive carbonaceous mudstone from the immediate roof has a positive impact on CMRR in all three cases. This mudstone varies from 0.22m to 0.56m across these three bore holes and ignoring practical (height), as well as economic / yield constraints, cutting down all of the mudstone would be the optimal approach from the geotechnical viewpoint. In any event, the presence of the carbonaceous mudstone (even at a reduced thickness) will necessitate the use of mesh on drivage.

ii) The mudstone thins away from the portal, which is a positive result (ie WML 056 recorded the maximum thickness of 0.56m).

iii) No obvious trend between CMRR and bolt length is apparent, chiefly because weaker, siltier units tend to be incorporated at the top of the bolted interval with the 2.1m bolts, offsetting the benefit of a reduced proportion of weak mudstone in the immediate roof. Note: this does not necessarily imply that there is no benefit in adopting longer bolts.

iv) The first 3m of roof at WML 056 is dominated by relatively weak siltstone units, whereas, further down dip, the same horizon at both WML 002 and WML 003 is composed largely of more competent sandstones.

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).

As previously noted in **Section 3.1.3**, this effectively self-supporting condition is defined as 'static' (see **Appendix C**); essentially the *in situ* horizontal stresses are insufficient to cause the roof beam to break down (buckle). The roof beam remains intact and typically exhibits ≤3mm of displacement, consistent with elastic movement. The roof tends to be characterised by a dry, flat profile with CM pick marks visible. The implication is that if static roof behaviour is predicted, 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.

Figure 23 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 23** 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 23** that the Ashton data for the deeper holes lies above the line, whereas the WML 056 (portal hole) lies below. It may therefore be concluded that the Pikes Gully Seam roof will generally be self-supporting on development, with the exception of the very shallow portal area. However, 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). Therefore, unless it were economic to remove all of this mudstone continually, place changing is not considered a viable option in this area of the mains. Furthermore, it is our experience that CMRRs of ≥50 are required for consistently productive place changing at shallow / moderate depths (ie to achieve an effectively "always stable" situation at depths of ≤300m). This is again consistent with the need to cut down the mudstone.

In the context of main headings roof support at Ashton, the following may be concluded:

i) Roof competency improves rapidly inbye from the shallow portal area and with the exception of the carbonaceous mudstone that forms the skin of the roof (ie an average of 0.3m), the roof is largely self-supporting. Ignoring other considerations and interests, it follows that a light to moderate primary bolting density would be appropriate in typical roof conditions, largely for the purpose of suspending the mudstone from the more competent overlying units.

ii) The presence of the mudstone, as well as the long-term nature of these main headings, dictates the use of mesh. The currently available mesh products have a maximum panel width of 1.7m, enabling a bolt row spacing of ~1.5m, allowing for mesh overlap. Four bolts per 1.5m would therefore be a practicable minimum roof support density and would be consistent with experience in other static roof environments.

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 (ie the element chiefly requiring support) that relatively short bolts would be generally appropriate. With regard to mining industry experiences of roof bolt length, 1.8 to 2.1m is the typical range applied in Australia, with limited applications of shorter (ie 1.2 and 1.5m), as well as longer (ie 2.4 and 2.7m) bolts. Cumnock Colliery successfully utilised 1.8m roof bolts in the Lower Pikes Gully Seam for several years, with some application of 1.5m long bolts over the last three years of operation; longer cables were applied in atypical, poor 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 24** 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 24** 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 is the CMRR range that is considered generally applicable in the case of the Ashton main headings 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 if at least some of the mudstone is cut down (as would be likely in practice). It is therefore concluded that the 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 current lack of site-specific experience and the long-term importance of the roadways, a conservative approach is warranted. In the original Strata Engineering report on main headings roof support for Ashton, the following bolt lengths were recommended:

- 2.1m long bolts at depths of ≤30m, where the CMRR is expected to be <45.
- 1.8m long bolts at depths of >30m, where the CMRR is expected to be ≥45.

However, given that the 30m contour runs directly through the middle of the road reserve, it is suggested that this provision be tightened, such that 2.1m bolts are applied throughout the reserve area.

Having determined the roof bolt length, a minimum support density can be derived using the following equation between PRSUP and CMRR (**Mark, 2000**):

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 absence of site-specific experience and the long-term importance of the main headings.

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 (m)
Nb	=	number of bolts per row
C	=	bolt yield capacity (kN)
Sb	=	bolt row spacing (m)
We	=	roadway width (m)

Given:

- a roadway width of ≤5.5m,
- a minimum 'T' grade bolt (180KN yield and 300kN uniaxial tensile strength),
- a practical minimum of four bolts per row and
- a maximum practicable row spacing of 1.5m,

it can be shown that the minimum Ashton PRSUP would be 11.8kN/m for 2.1m roof bolts. As indicated previously, the PRSUP value suggested by NIOSH varies with CMRR, which for the relevant Ashton bore core data is summarised in **Table 14**.

Table 14: Suggested PRSUP Values for Intersections versus CMRR

Borehole Number	Depth (m)	Roof 'as is'		0.2m of mudstone cut down	
		CMRR	Suggested PRSUP	CMRR	Suggested PRSUP
WML 002	60	47.4	4.6	50.1	4.0
WML 003	41	59.1	1.9	61.8	1.3
WML 056	17	37.2	6.9	39.0	6.5

Given that 0.2-0.5m of the immediate roof mudstone is being cut down throughout the area of interest, the CMRR varies in practice from a minimum of 39 outbye to 62 inbye. A pattern of four bolts per 1.5m would therefore result in a PRSUP value 81 to 808% higher than that suggested by NIOSH.

This outcome reflects that obtained using the Q system; the generally intended / practicable minimum levels of support are significantly higher than the theoretically required values.

3.2.4 Concluding Remarks Regarding the Rock Mass Classification Results

The results of the analysis for the NGI Q, CSIR RMR and NIOSH CMRR systems are highly consistent and indicate in particular:

i) The rapid improvement in roof competency in the inbye direction.

ii) The weak nature of the thin layer of mudstone in the immediate roof.

iii) The generally self-supporting nature of the main roof sandstone.

iv) The potential, ignoring specific stakeholder concerns with regard to the area of interest, to apply relatively low densities of roof support in the main headings.

v) Even applying factors associated with a very high degree of design conservatism, in particular, a Q system ESR value of 0.8 for the area of the road reserve under consideration, the support outcomes remain moderate.

The databases underpinning these long established, internationally recognised rock mass classification systems cover a broad range of civil and mining tunnelling applications and in combination are considered to address every relevant aspect of the Ashton main headings geotechnical environment.

3.3 Further Analysis

Given the criticality of the workings in question, roof stability has also been assessed using three alternative techniques, namely:

- a “voussoir” or linear-arch analytical model,
- a finite element analysis and
- a “dead-weight” load model.

The voussoir arch model assesses the natural stability of a jointed rock mass by resolving forces through the beam; in this case it has been used as a check of the spanning ability of the main sandstone unit in the roof.

The following inputs were applied:

- Span: 5.5m
- UCS: 40MPa
- Young’s Modulus: 6GPa
- Angle of friction: 35^{o}
- Joint angle from the horizontal: 80^{o}
- Tensile strength: zero
- Cohesion: 0.5MPa
- Vertical / horizontal stress ratio: 2

The results indicate that a bed 1m thick will span the 5.5m roadway and is stable in terms of abutment crushing, shear and buckling, even when subject to the full overburden load. This again suggests that the main roof sandstone would span the roadway unsupported, even in the presence of sub-vertical joints and at low horizontal stress values associated with shallow conditions.

A simple finite element model has been developed for the shallow entries, assuming that the only horizontal stress is due to self-weight (ie Poisson’s ratio effect). As illustrated in **Figure 25**, the results suggest that the zone of tension in the immediate roof (the zone most likely to fail in this case) is limited to a height of approximately 1m, within the interbedded siltstone and sandstone, below the main sandstone. Anchoring the immediate mudstone / siltstone to the overlying stable strata is once more indicated to be an effective support strategy.

One of the simplest and generally most conservative forms of roof support analysis involves assessing the maximum likely height of potential unstable material and then designing the support system to suspend the potential associated dead-weight load from the overlying strata, which is assumed to be stable. In the Ashton case, the maximum potential fall height in the area of interest is considered to be to the base of the main sandstone, which is in turn a maximum of 2.4m into the roof. Such a fall height would also be consistent with experience of typical roadway falls.

Applying the roof fall model shown originally in **Figure 1**, together with a caving angle of 20^o from the vertical, it can be shown that the associated dead-weight load for a 2.4m fall height is 27.8 tonnes per metre of roadway. Two 580kN Hi-TEN cables per 3m of drivage would be sufficient to carry such a load, with a Factor of Safety of 1.4, which is considered adequate. Given that the intent is to design for a worst-case dead-load, the cables would be positioned closer to the rib lines than would be considered conventional for a reinforcement design and the cables would also be angled out over the rib lines at an angle of 75^o (to further increase the likelihood of anchoring outside any potential failure envelope). The result would be as shown in **Figure 26**. The general desirability of anchoring above / outside any likely failure plane is the main reason why shorter cables would not be considered adequate in this case, given the design considerations.

Finally, it is worth noting that as the interbedded siltstone and sandstone unit thins, this roof support regime will become extremely conservative towards the inbye limit of the area under the road reserve. However, there is little scope to rationalise the system, recognising also the logistics involved (ie the area is of limited extent).

3.4 Roof Support Design for Headings Beneath / Within 20m of the Road Pavement

Having considered all of the preceding sources of information and methods of analysis, the recommended roof support regime for the entries under and within a horizontal distance of 20m of the road pavement is summarised as follows.

Primary Support:

- Six 2.1m long, T Grade bolts per metre, installed through mesh
- Two-speed chemical resin anchored, sufficient to generate ≥90% encapsulation
- Molybond-coated low friction nuts, dome balls and 150mm x 150mm x 10mm plates
- Pre-tensioned to a minimum of 8 tonnes
- Installed within 10^o of vertical

The bolts should be installed within 5m of the face.

Secondary Support:

- Two 6m long, 580kN capacity cables per 3m
- ≥1.5m of resin anchor
- Post-grouted with Stratabinder HS or equivalent, to generate full encapsulation
- 200mm x 200mm x 12mm plates
- Pre-tensioned to a minimum jack load of 20 tonnes
- Installed 1.5m either side of centre and at 15^o from vertical over the rib lines

The cables should be installed and tensioned within 5m of the face, as well as post-grouted within 30m of the face.

This support density amounts to 391% of the recommended NGI Q system value, derived using conservative inputs with regard to rock mass quality Q and Excavation Support Ratio (ESR).

The support density also exceeds the recommended values derived using the NIOSH CMRR system (ie the PRSUP results) by a factor of ≥8.

In arriving at the roof support design, consideration has been given to the properties of the rock mass and likely modes of deformation, including a highly conservative, "dead-weight" load scenario. The only mode of deterioration that is not considered to be fully addressed using the proposed bolts, mesh and cables system is the potential for weathering of moisture sensitive units (ie mudstone / siltstone) in the immediate roof to result in frittering and skin deterioration.

Therefore, it is also recommended that the roof be sealed in the headings beneath and within 20m of the road pavement. In the intake roadways (ie the men and materials and conveyor roadways), which will be deeper and driven in more competent rock, it would be considered appropriate to apply a thin layer (2-3mm) of a proprietary sealant, such as Tekflex. However, in the return airway, given the increased humidity and the slightly shallower and weaker rock mass, it is recommended that a minimum of 25mm of fibre-reinforced shotcrete be applied.

3.5 Roof Support Design for Intersections A to D (refer to Figure 12)

Recognising that:

i) these intersections are less critical than the area defined above,

ii) overall rock mass quality results suggest that these excavations should be self-supporting,

iii) that the weak mudstone in the immediate roof is expected to be <0.5m in the area (and that a portion of this material will almost certainly be removed on drivage),

iv) the potential to monitor these areas prior to widening and

v) the only apparent justification for the use of cables would relate to managing the extremely low likelihood of widening these excavations in the presence of an undetected major geological structure,

the recommended roof support regime for all four intersections is outlined as follows.

Primary Support:

- Six 2.1m long, T Grade bolts per 1.5m, installed through mesh
- Two-speed chemical resin anchored, sufficient to generate ≥90% encapsulation
- Molybond-coated low friction nuts, dome balls and 150mm x 150mm x 10mm plates
- Pre-tensioned to a minimum of 8 tonnes
- Installed within 10^{o} of vertical

The bolts should be installed within 5m of the face.

Secondary Support:

- Two 6m long, 580kN capacity cables per 3m
- ≥1.5m of resin anchor
- Post-grouted with Stratabinder HS or equivalent, to generate full encapsulation
- 200mm x 200mm x 12mm plates
- Pre-tensioned to a minimum jack load of 20 tonnes
- Installed 1m either side of centre and vertically

The cables should be installed and tensioned prior to intersection formation.

This support density amounts to 200 to 400% of the recommended NGI Q system values.

In the case of intersection ‘D’, some 38m inbye of the edge of the road pavement, it would be considered appropriate to omit the cables if the excavation is driven to plan (ie to a span of ≤10m).

3.6 Rib Stability

The preceding assessment does not directly address issues related to rib stability. Given:

- the shallow depths in the area of interest (ie <50m).
- the generally limited drivage heights (ie typically ≤3m).
- the absence of appreciable bands of weakness within the seam (ie significant clay bands).
- the finite element modelling results, which suggest that yielding will be confined to the first 200mm of the rib and then largely within the mudstone band cut down at the top of the seam, see **Figure 27** and
- Strata Engineering’s experience elsewhere of coal rib behaviour in similar mining environments,

no systematic bolting of the ribs is recommended for the headings beneath and within 20m of the road pavement. It is, however, recommended that sealing of the roof extend down the rib lines for a minimum of 0.5m, to protect the weak mudstone exposed at the top of the rib.

The main residual hazard with regard to rib stability for the headings is considered to relate to the limited possibility of encountering a major geological structure (eg a fault or dyke). If a structure of this nature is encountered, it is recommended that a site-specific system of rib support be developed to address any localised deterioration that might result.

Also, the identified 4.5m high intersections inbye of the road reserve (A to C) are considered to require systematic rib support. Accordingly, it is recommended that 1.2m ‘T’ grade bolts be installed on a 1.5m square grid (ie three rows of bolts, with the top row 0.75m from the roof line) and with mesh for 2.5m either side of each corner. Bolts should be fully encapsulated and fitted with 150mm x 150mm x 10mm plates.

4.0 STRATA MANAGEMENT

The preceding support recommendations should be applied in combination with a formalised 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,

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

v) catering for geometrical anomalies, such as roadways wider than 5.5m,

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

Although negligible roof movement is anticipated, monitoring should be conducted to confirm the maintenance of adequate stability from initial drivage through to the end of the life of the project. Accordingly, it is recommended that:

i) 8m Tell-Tales be installed at 10m intervals throughout the area of interest in the headings.

ii) 8m Tell-Tales be installed at intersections A to D.

iii) Sonic extensometers be installed in the roof of all three entries, immediately below the road pavement (these provide more accurate and detailed information on the location in the roof of any strain zones, as well as rates of movement).

iv) All instruments should be installed within 5m of the face.

v) Borescope holes should be drilled adjacent to all monitoring stations.

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

5.0 CONCLUDING REMARKS

This report has addressed two specific issues, namely:

- Long-term roof stability for the this area of the Pikes Gully main headings and the potential for measurable surface impacts, primarily, through the development of sinkhole-type subsidence.

- Roof support design for areas of significant interest to other stakeholders (chiefly the RTA and DPI).

The issue of long-term roof stability and possible subsidence relates primarily to surface protection and particularly the drivage of the three initial entries from the portal beneath the New England Highway, noting that this has caused the main headings to be moved inbye from their originally planned position. Given the significance of the highway, the likelihood of significant surface impacts must be precluded.

In the context of support design, there is no universally acceptable design methodology that would preclude roof failure indefinitely. Therefore, this study has concentrated on defining the consequences of any localised instability (ie roof fall), including the potential for measurable surface impact.

It has been shown, using a variety of analytical approaches coupled to global experience, that the latest mine design and planned geometry precludes sinkhole development within the road reserve. Outside of the road reserve there are very limited areas in which a sinkhole is not also precluded and even in these areas the likelihood of such an event is still generally considered to be practically impossible, in the long-term.

Nevertheless, a series of specific ground support and strata management measures have been proposed for key areas, primarily under the road pavement and within a horizontal distance of 20m thereof. The support measures address the various possible modes of roof deformation and deterioration in the area of interest. Conservative inputs have been used to a number of alternative methodologies widely applied in the civil tunnelling and coal mining fields. The outcomes of the analyses are consistent across the different methodologies.

The support design outcomes are also highly conservative, reflecting what are considered to be worst-case scenarios for the areas concerned. In general, the recommended roof support designs far exceed the outcomes determined using widely accepted and applied techniques for analysing rock mass competency and stability.

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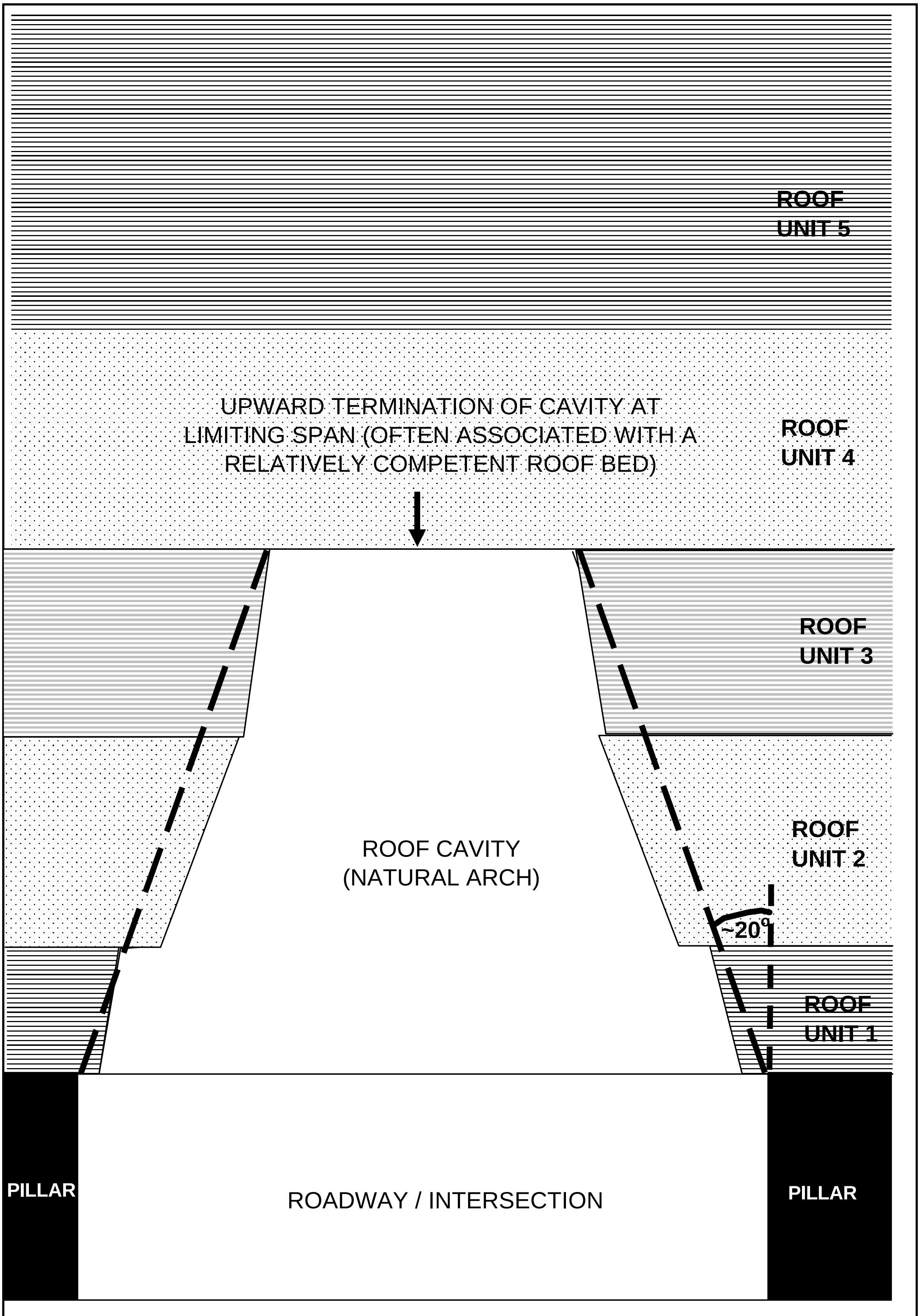
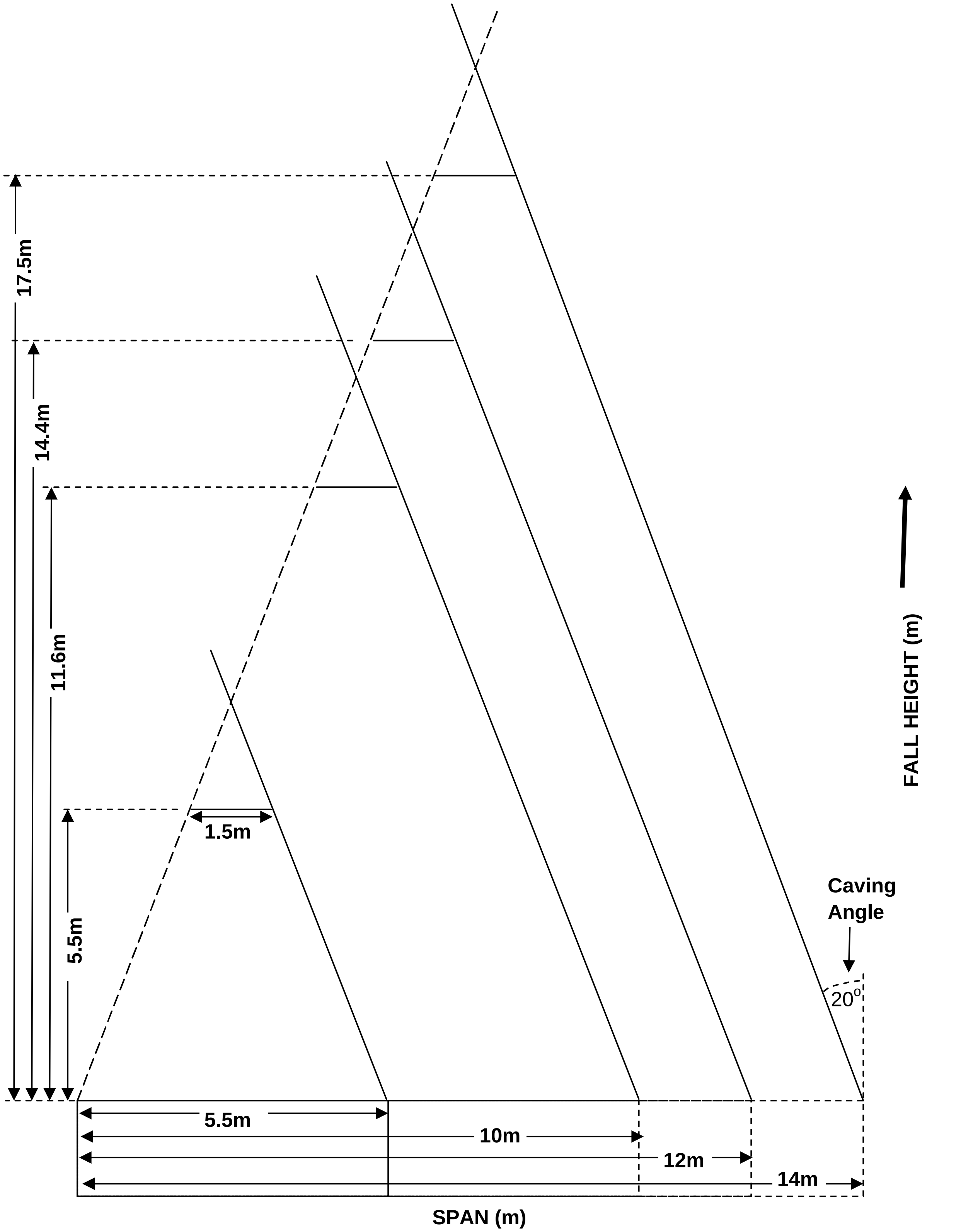


ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**	
DATE:	23.03.05	TITLE:	Schematic Section Through Typical Coal Mine		FIGURE
SCALE:	NTS		Roadway Roof Fall		1



ENGINEER:	D. Hill	CLIENT:	Ashton Mine R04-001-ASH-4	**STRATA ENGINEERING (Australia) Pty Ltd**	
DRAWN:	D. Hill				
DATE:	23.03.05	TITLE:	Theoretical Roof Fall Geometries at Practical Underground Colliery Roadway Spans		FIGURE
SCALE:	NTS				2

Figure 3a: Displacement Graph Showing Progressive Upward Extension of Roof Displacement

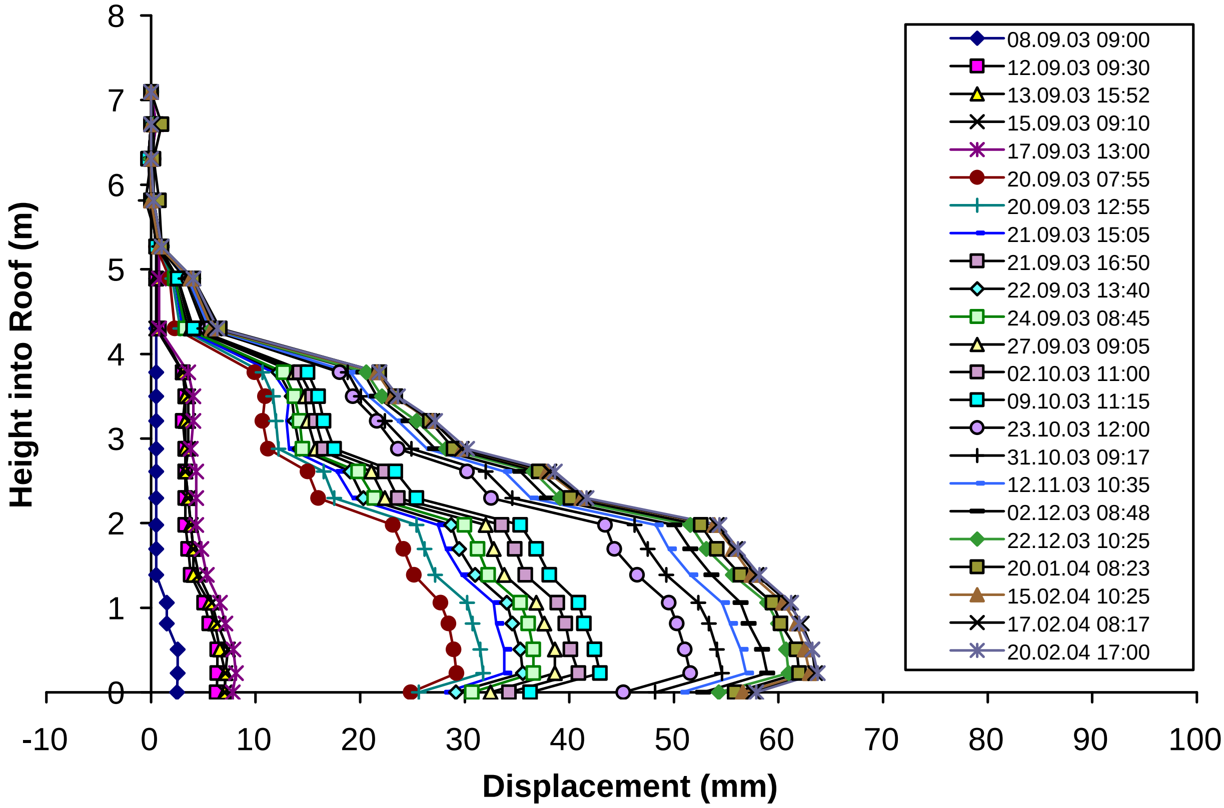
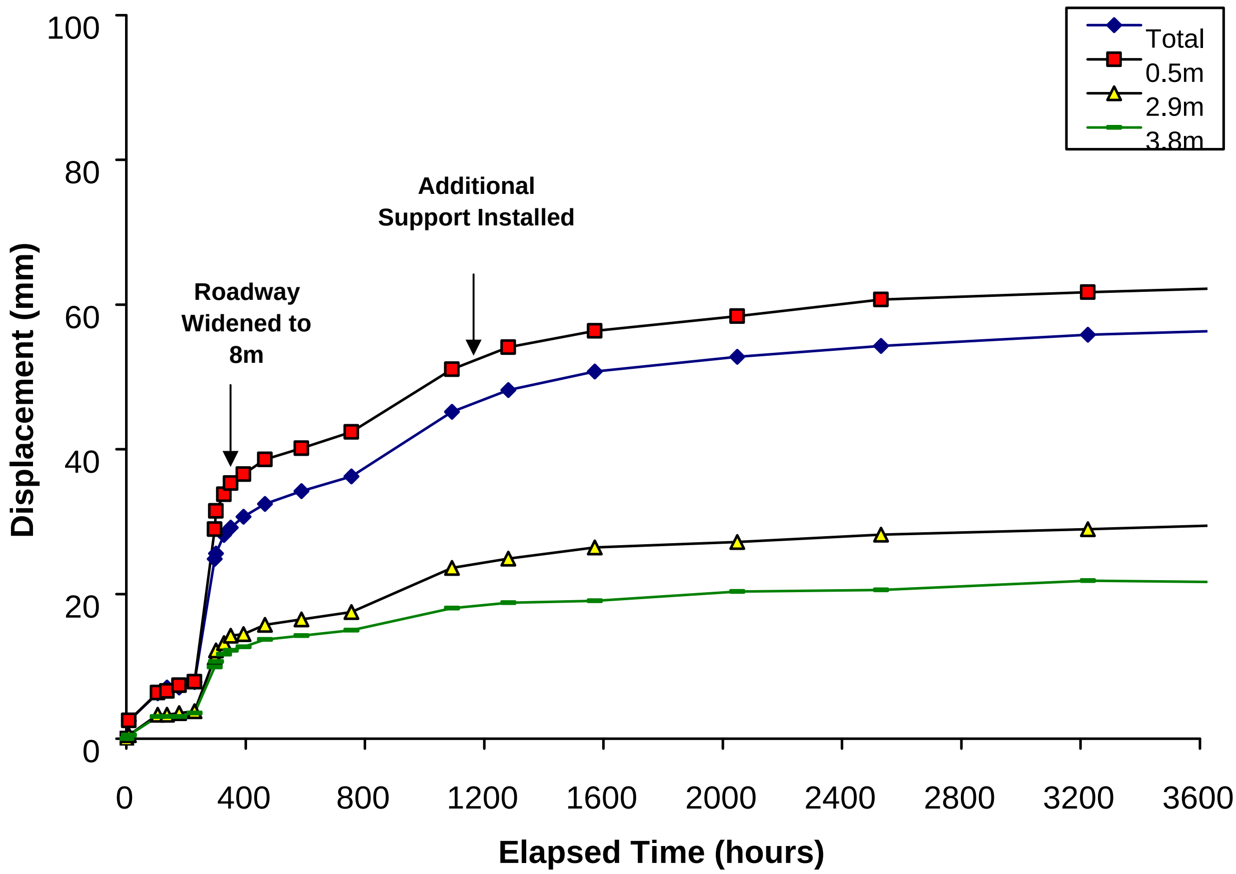
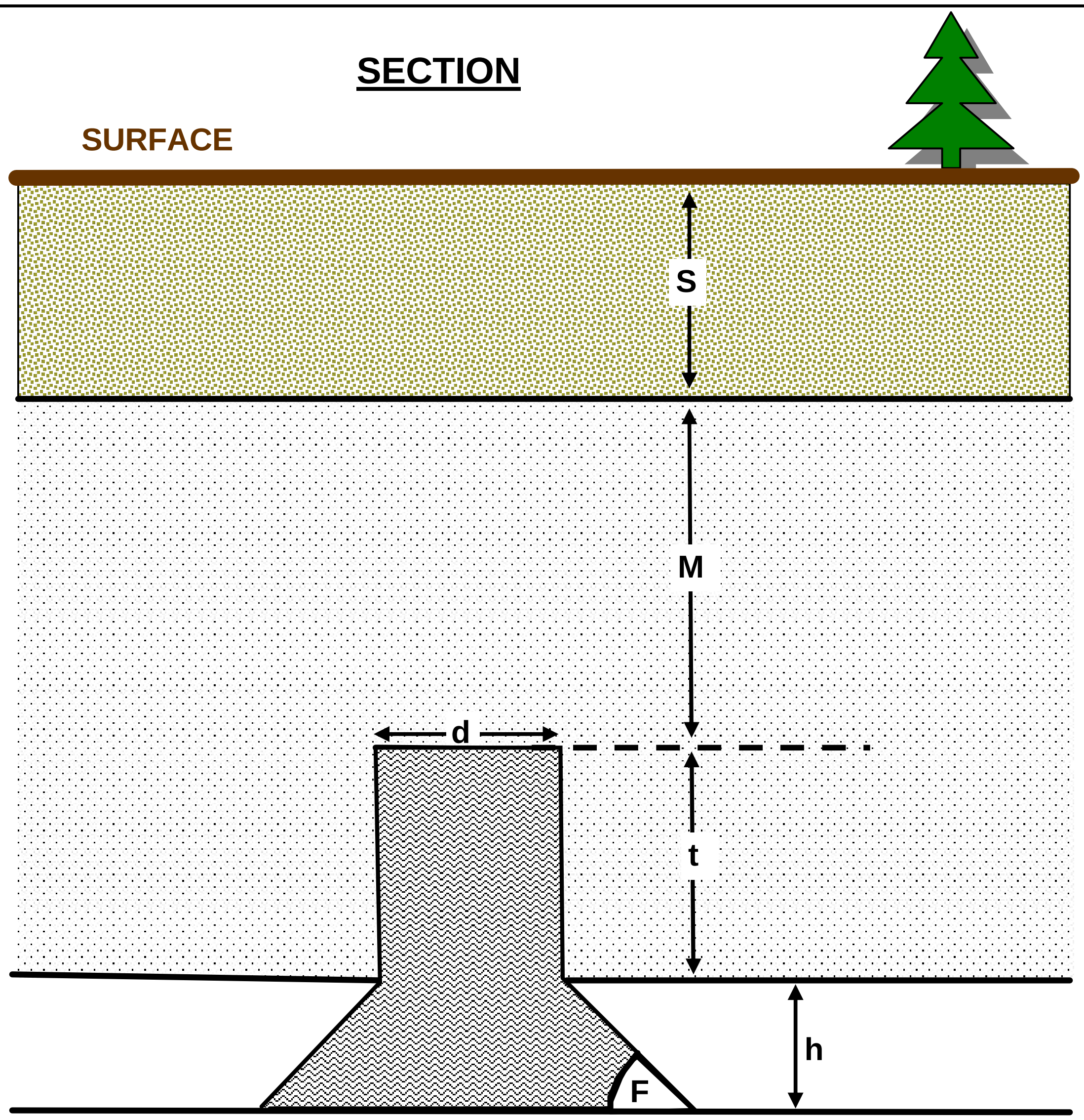
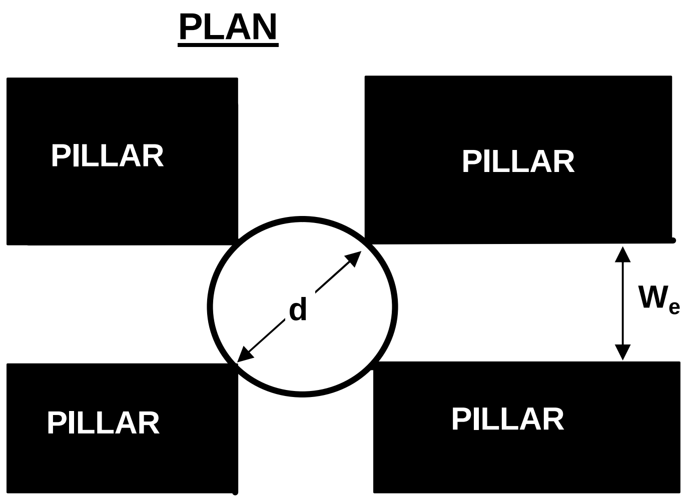


Figure 3b: Time-Dependent Graph Showing Displacement Trends



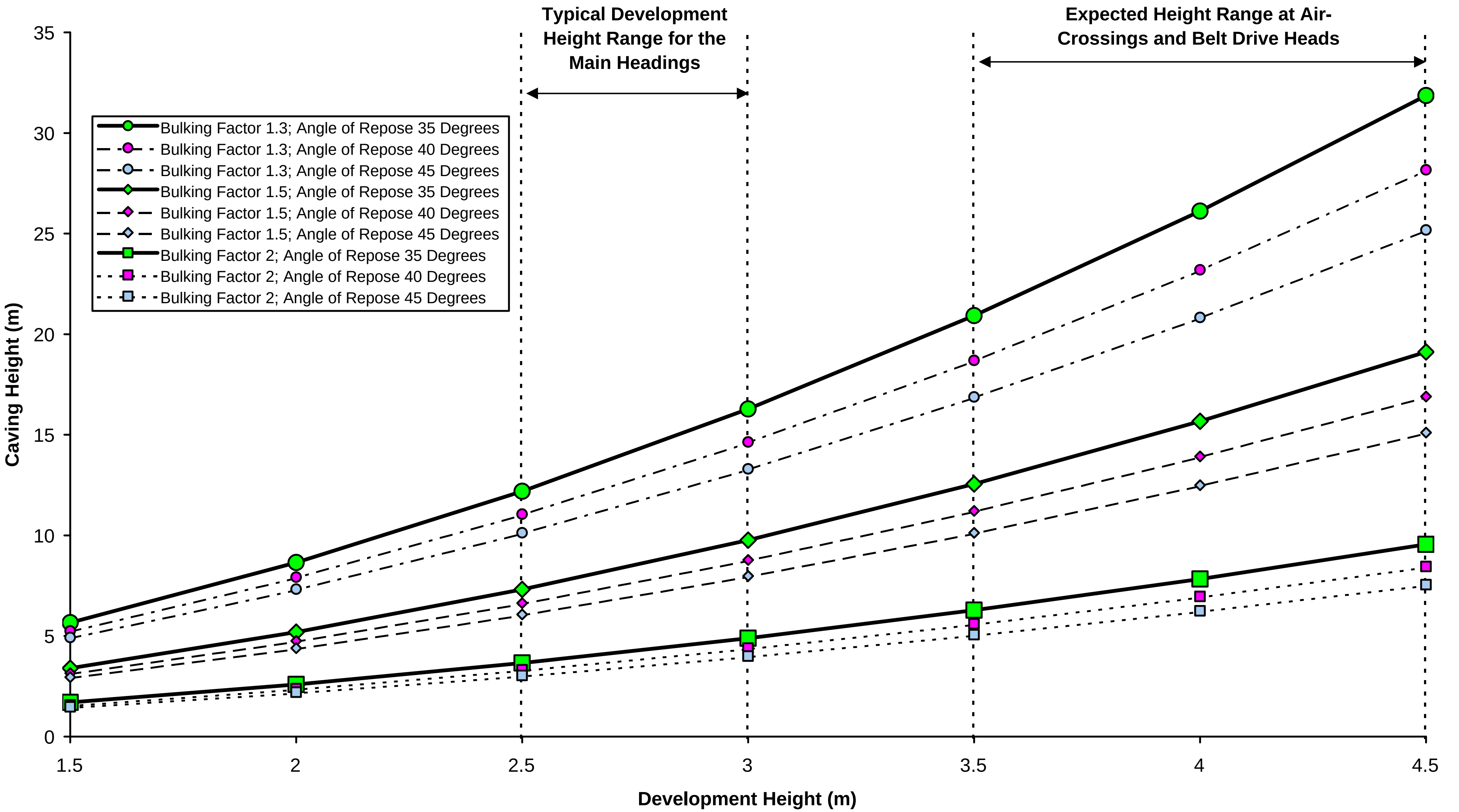
ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING (Australia) Pty Ltd**	
DRAWN:	D. Hill		R04-001-ASH-4		
DATE:	22.12.05	TITLE:	Example of Roof Monitoring Results for a Major Underground Coal Mine Excavation (8m Wide Roadway)		FIGURE 3
SCALE:	NTS				





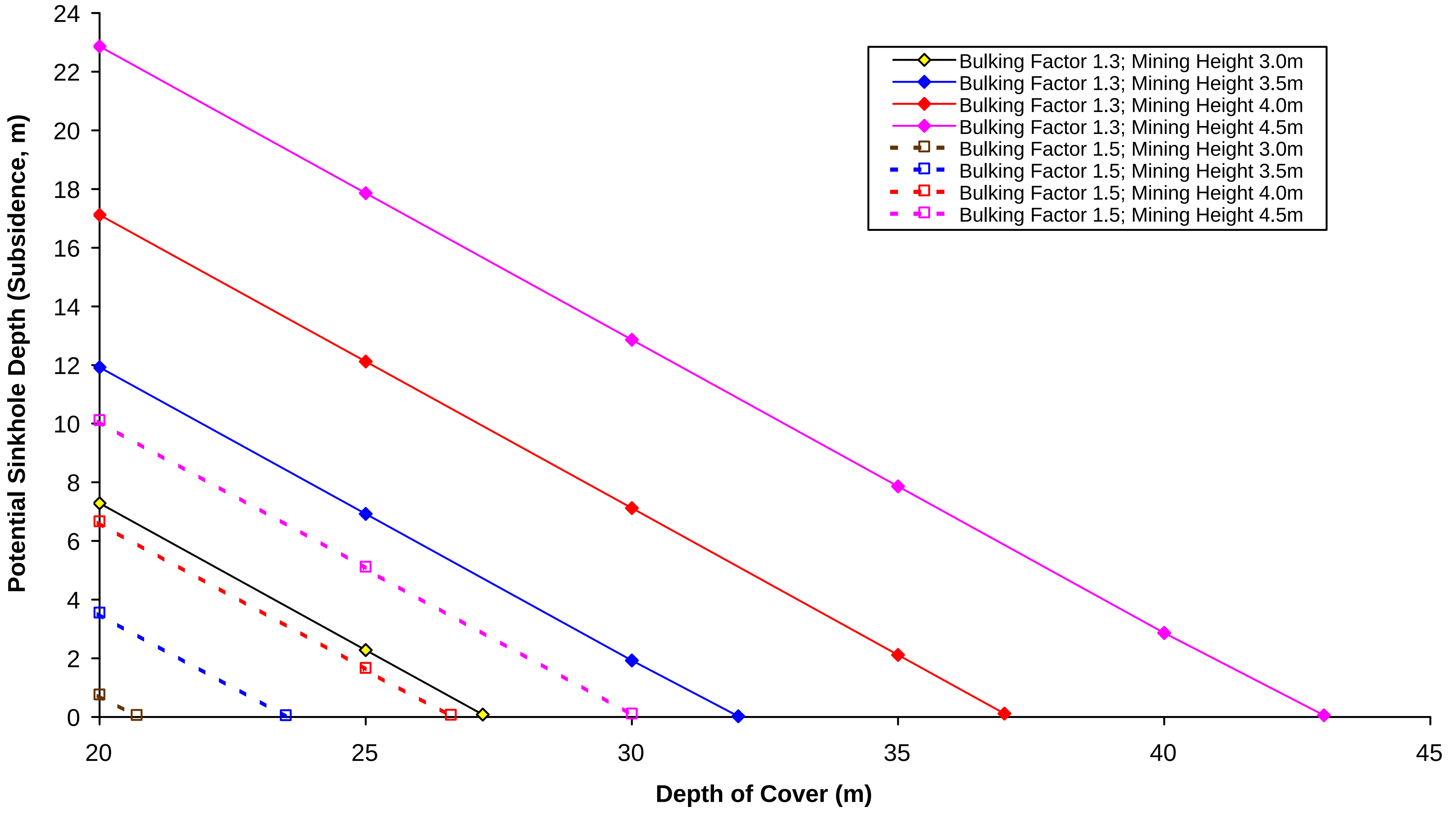
d = (We x 1.414)

ENGINEER:	D. Hill	CLIENT:	Ashton Mine R04-001-ASH-4	**STRATA ENGINEERING (Australia) Pty Ltd**	
DRAWN:	D. Hill				
DATE:	23.03.05	TITLE:	Mechanism of Sink Hole Development above Bord and Pillar Workings (after **Whittaker and Reddish, 1989**)		FIGURE 4
SCALE:	NTS				

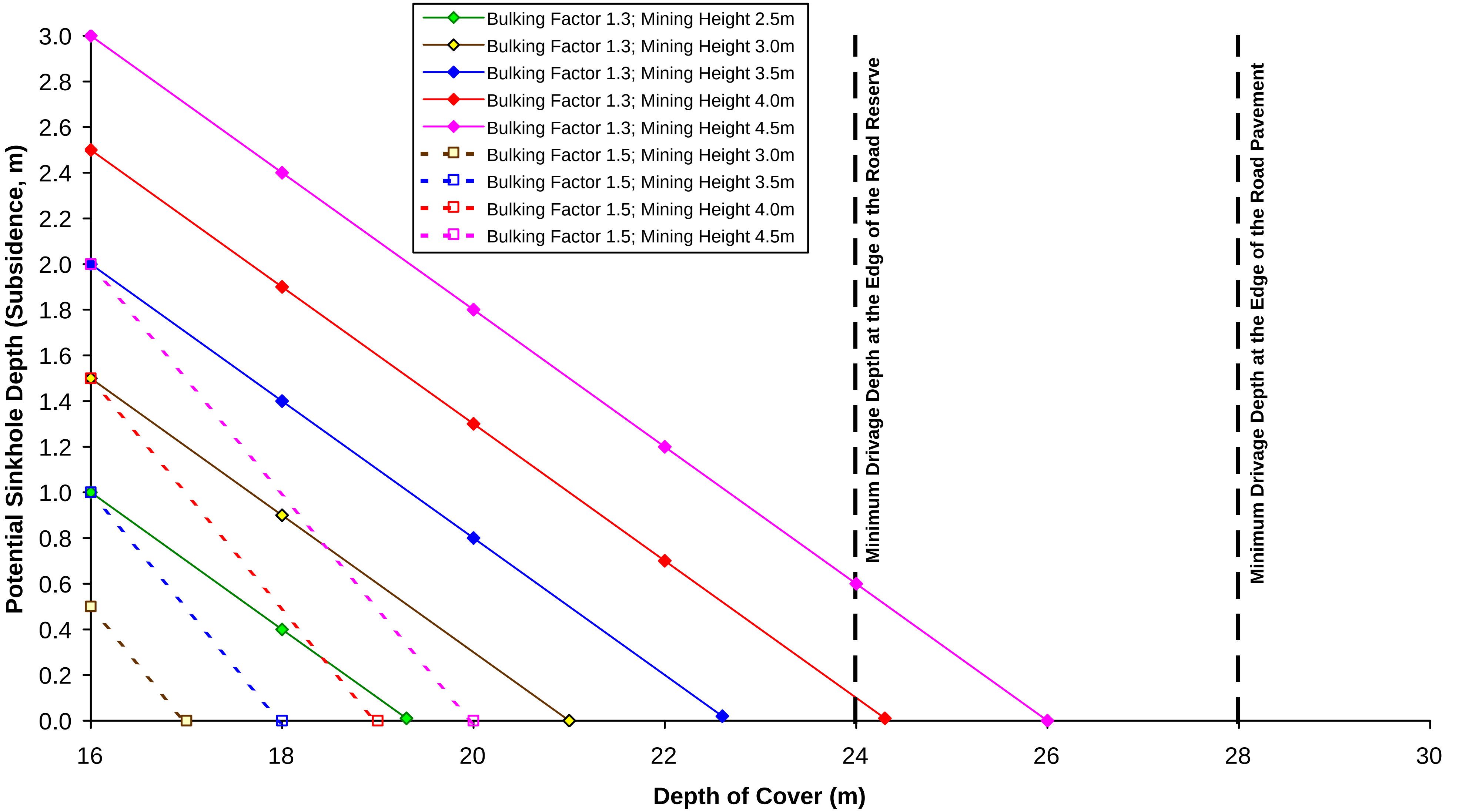


ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**	
DATE:	09.12.05	TITLE:	Maximum Potential Caving Heights above		FIGURE
SCALE:	NTS		Intersections in the Main Headings at Ashton		5

ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**	
DATE:	22.12.05	TITLE:	Arties Pit at Ashton, showing the Highwall		FIGURE
SCALE:	NTS		and Portal Preparation Works		6

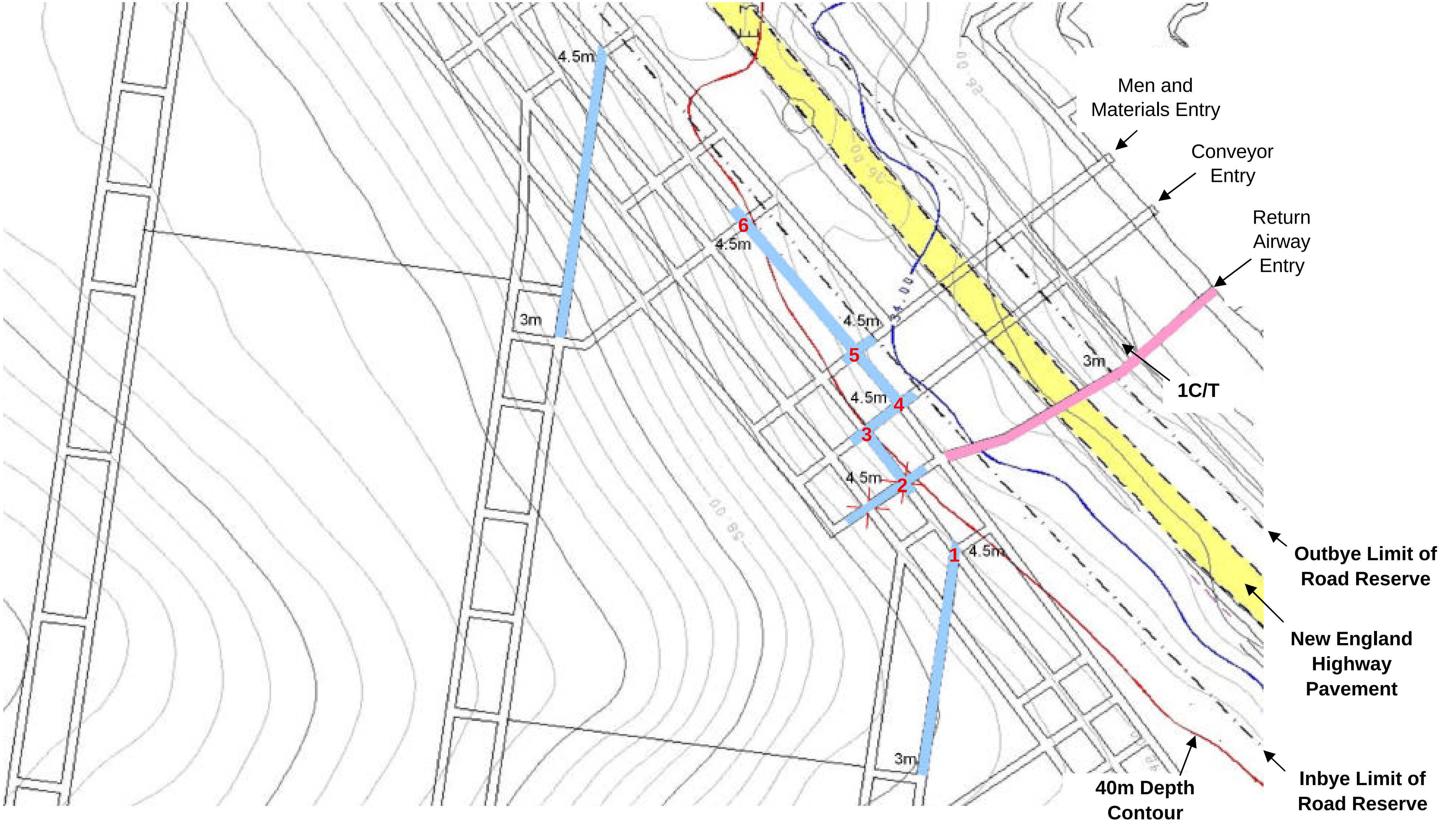


ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**	
DATE:	22.12.05	TITLE:	Potential Sinkhole Depths for Four-Way Intersections		FIGURE
SCALE:	NTS		(Derived from **Whittaker and Reddish, 1989**)		7



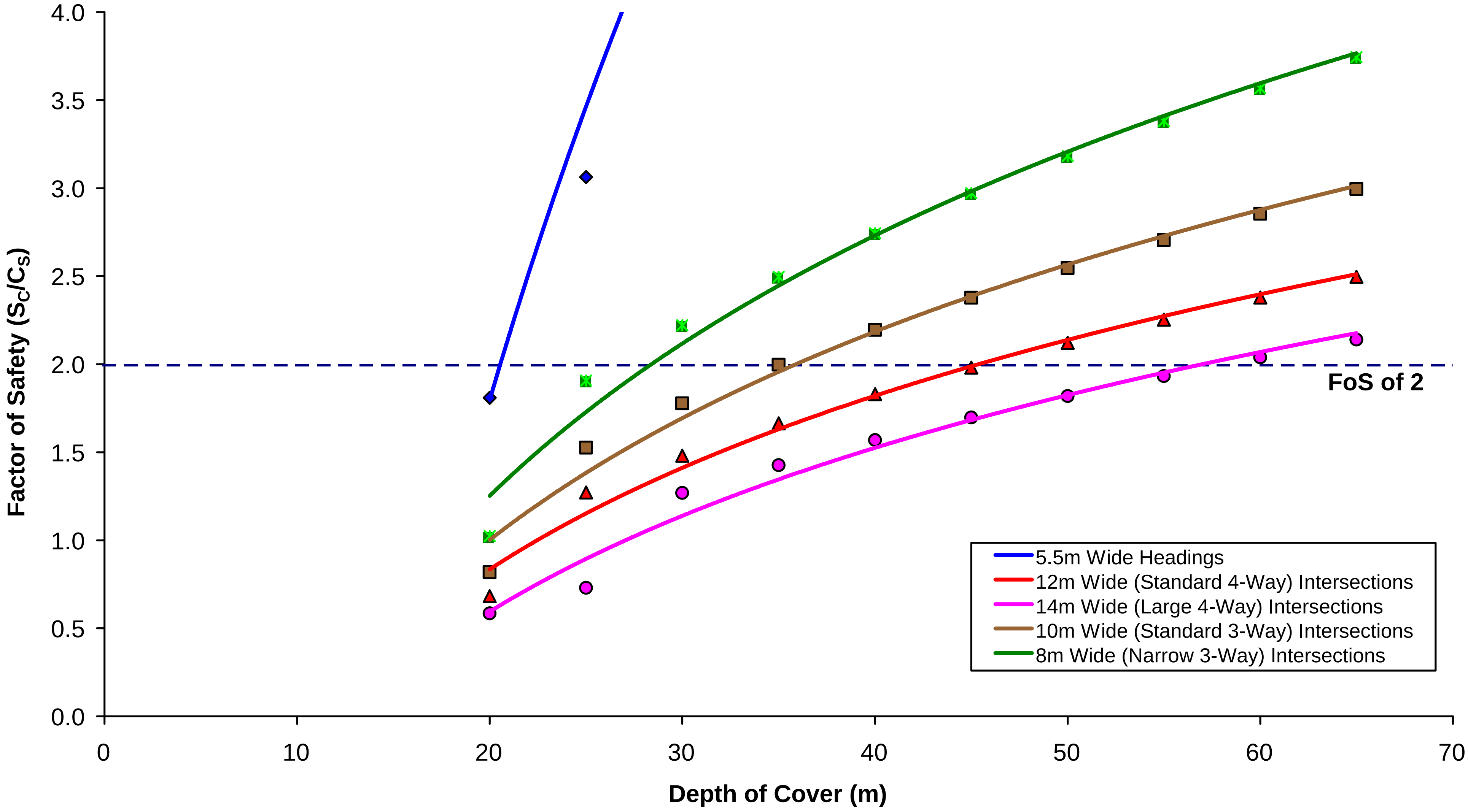
Assumption: Roadway width = 5.5m

ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**	
DATE:	22.12.05	TITLE:	Potential Sinkhole Depths above Underground Headings		FIGURE
SCALE:	NTS		based on a Two-Dimensional Bulking Factor Model		8

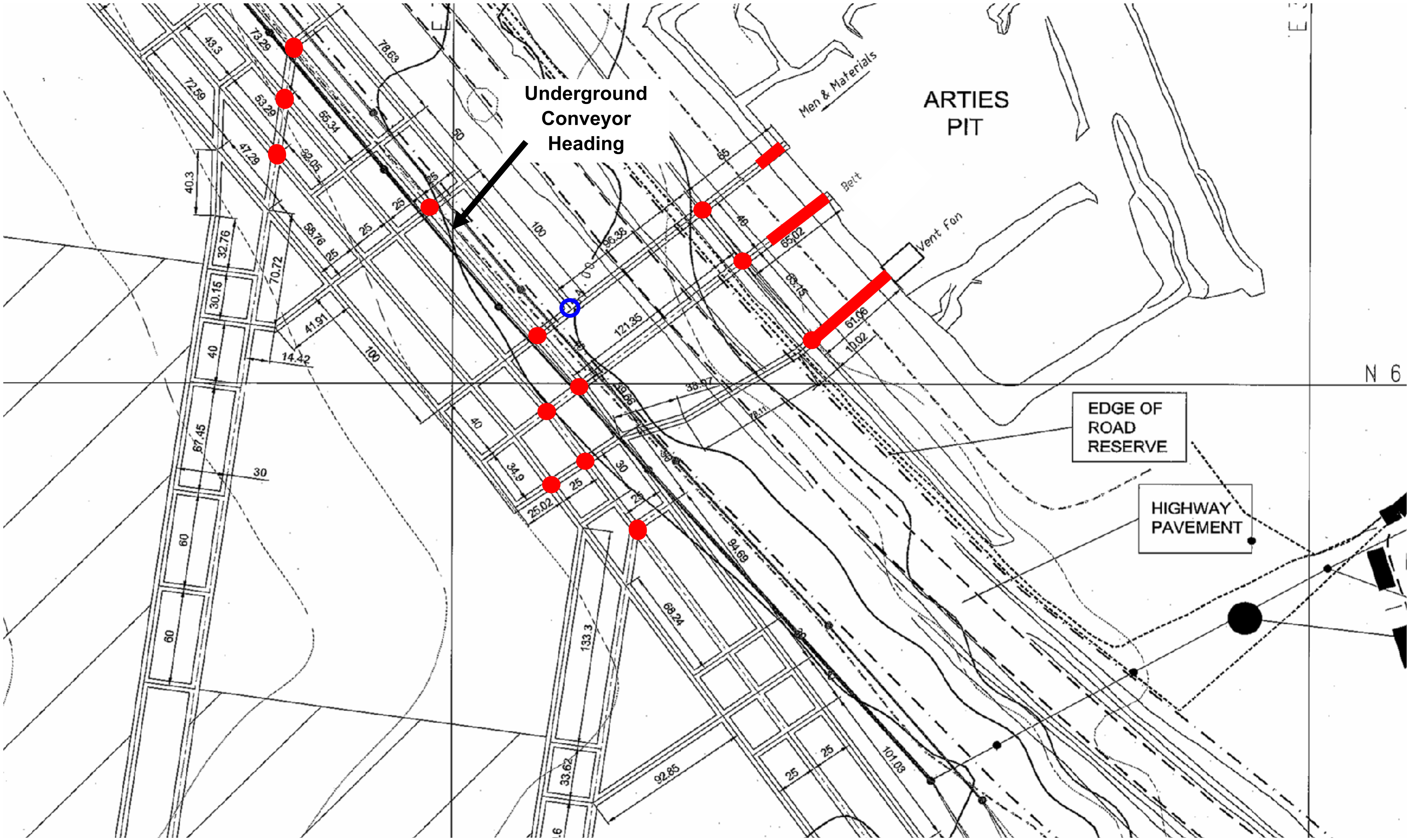


Note: Pink : 3m Drivage Height
Blue : 4.5m Drivage Height
Other : 2.5 to 3m Drivage Height
1-6 : 4.5m High Intersections at <43m Depth of Cover

ENGINEER:	D. Hill	CLIENT:	Ashton Mine	STRATA ENGINEERING (Australia) Pty Ltd
DRAWN:	D. Hill		R04-001-ASH-4	
DATE:	28.12.05	TITLE:	Planned Development Heights for the Portal Entries and Initial Area of the Main Headings at Ashton	FIGURE 9
SCALE:	NTS			



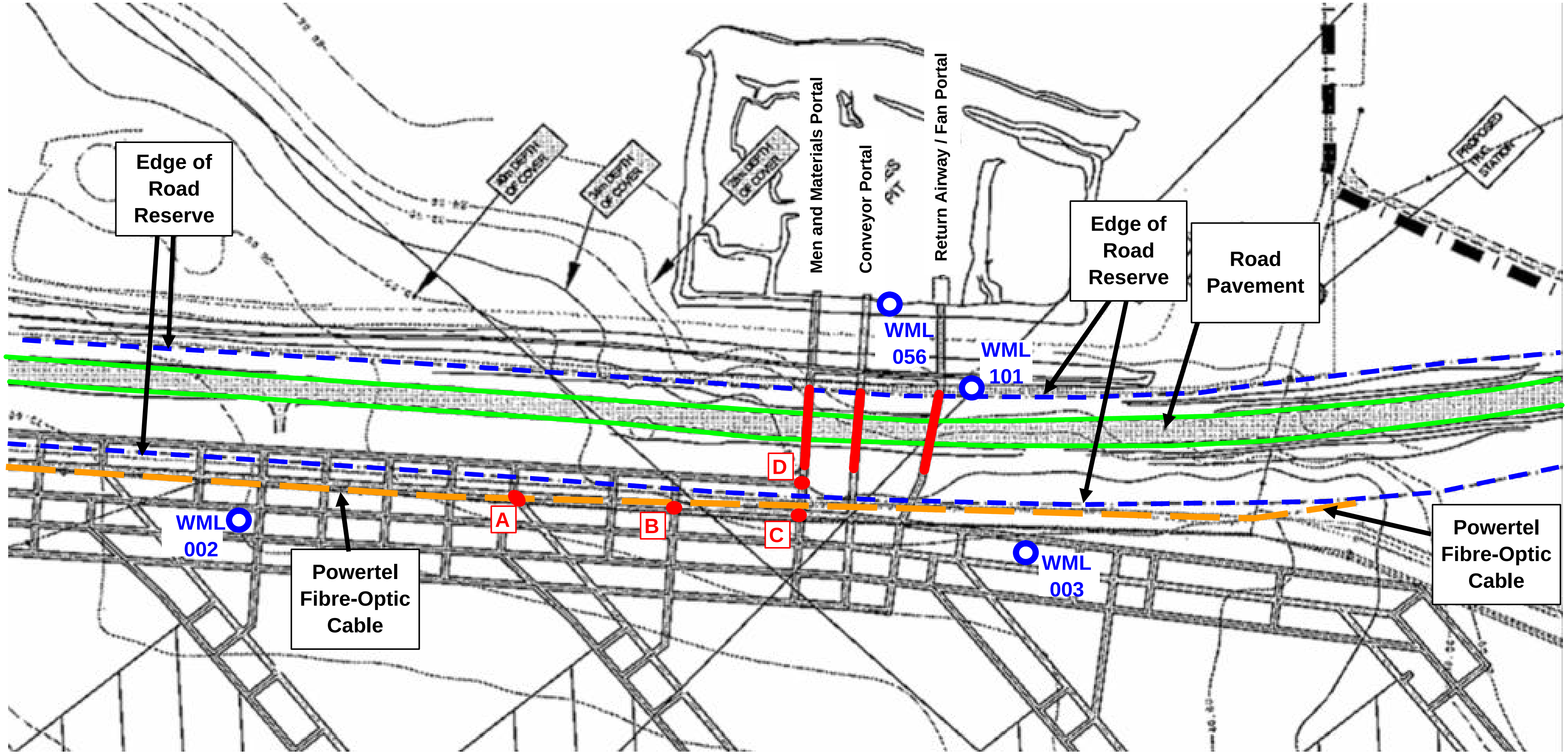
ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**	
DATE:	24.12.05	TITLE:	Scaled Span Factors of Safety for the Roof of the Initial		FIGURE
SCALE:	NTS		Entries at Ashton		10



Notes:

- ● Sinkhole formation not Totally Precluded
- ○ Sinkhole formation only Precluded at a Span of ≤10m

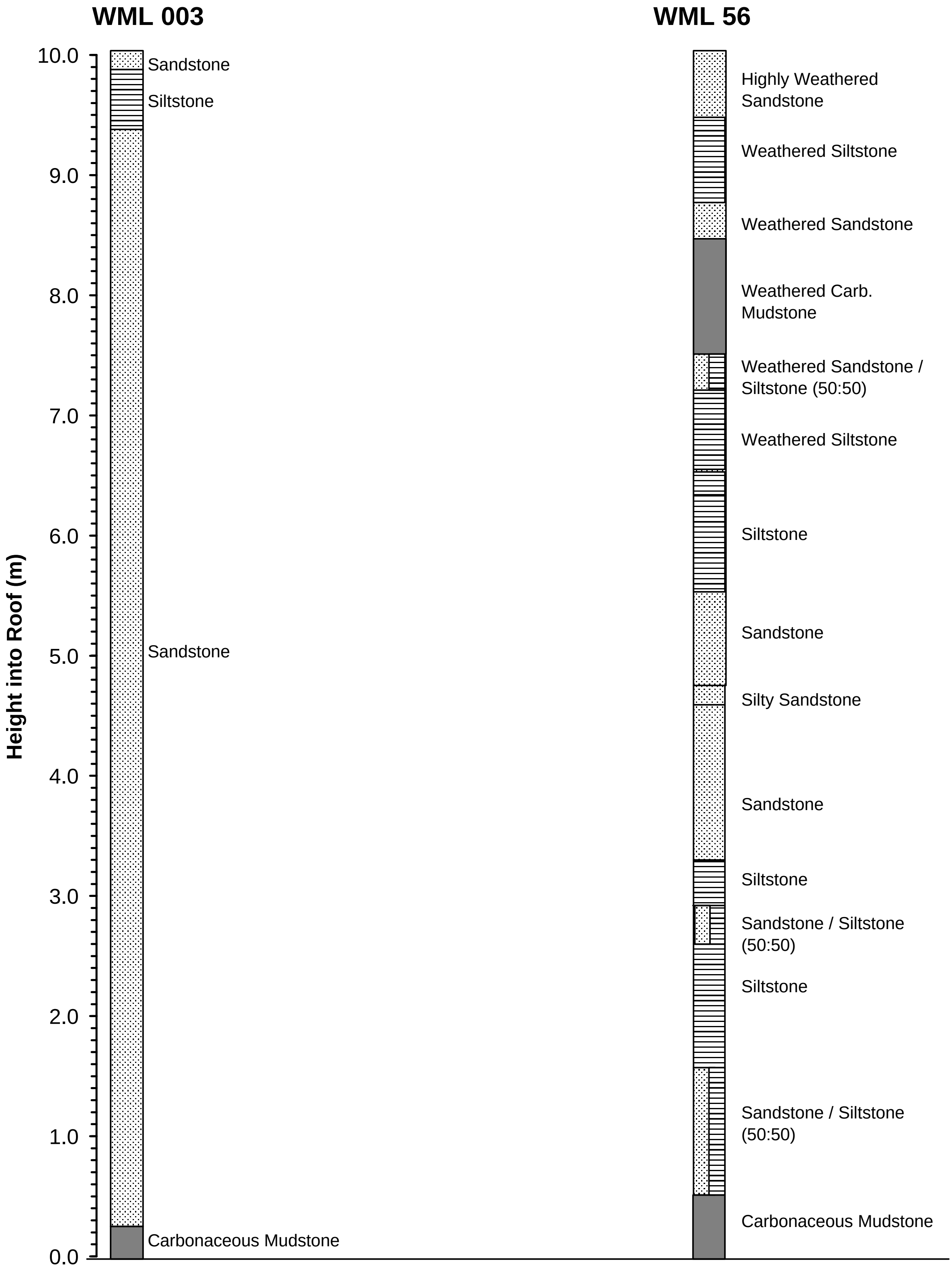
ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING (Australia) Pty Ltd**	
DRAWN:	D. Hill		R04-001-ASH-4		
DATE:	24.12.05	TITLE:	Locations Requiring Additional Controls to Preclude the Potential for a Sinkhole Forming in the Long-Term		FIGURE
SCALE:	NTS				11



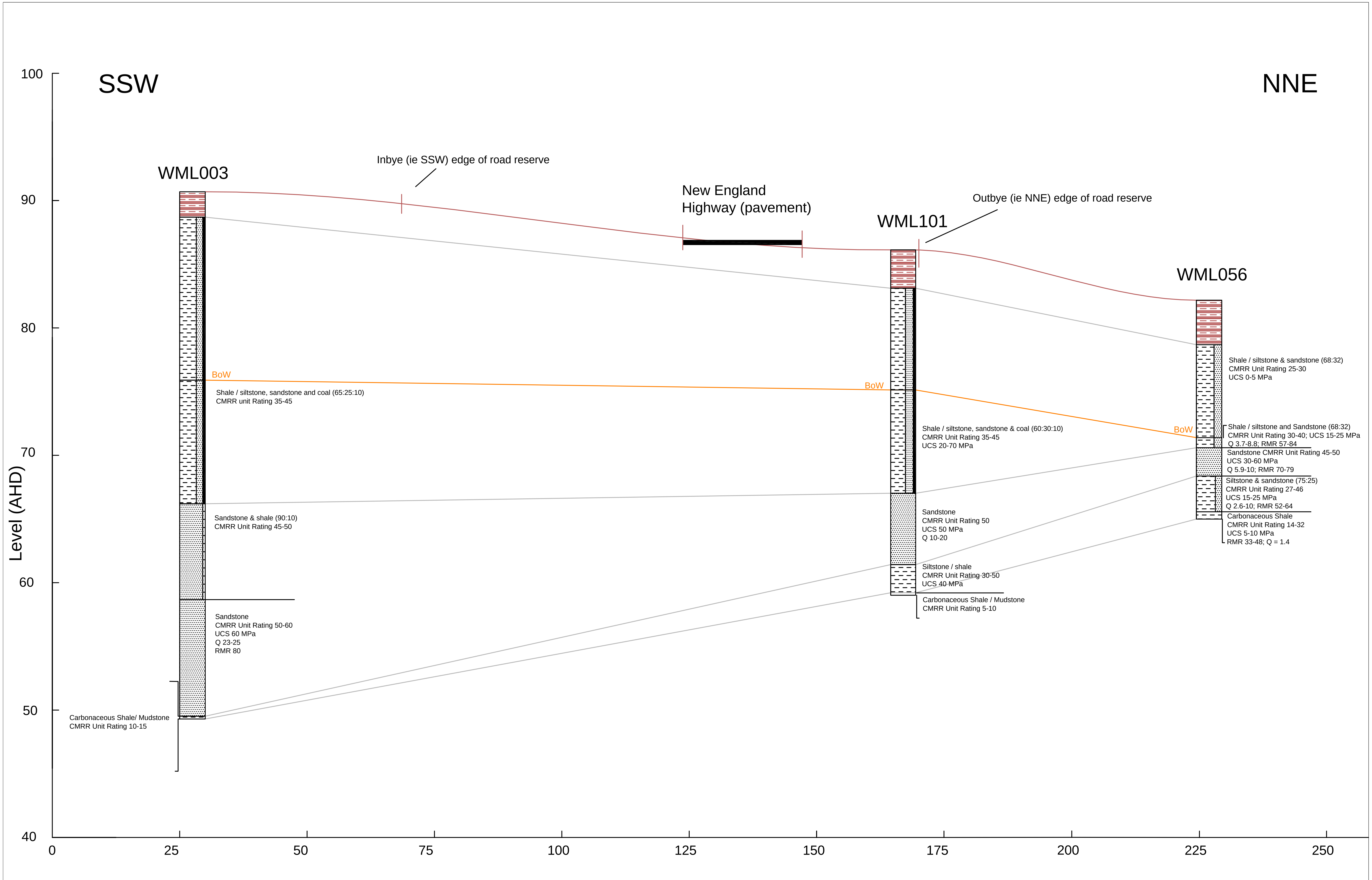
Note:

underground area of increased roof support density

ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**	
DATE:	24.12.05	TITLE:	Main Headings showing Road Pavement and Reserve,		FIGURE
SCALE:	NTS		Areas of Increased Support Density and Boreholes		12



ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**
DATE:	05.04.05	TITLE:	Comparison of Roof Lithologies for WML 003 and 56	FIGURE
SCALE:	NTS		(Pikes Gully Seam)	13



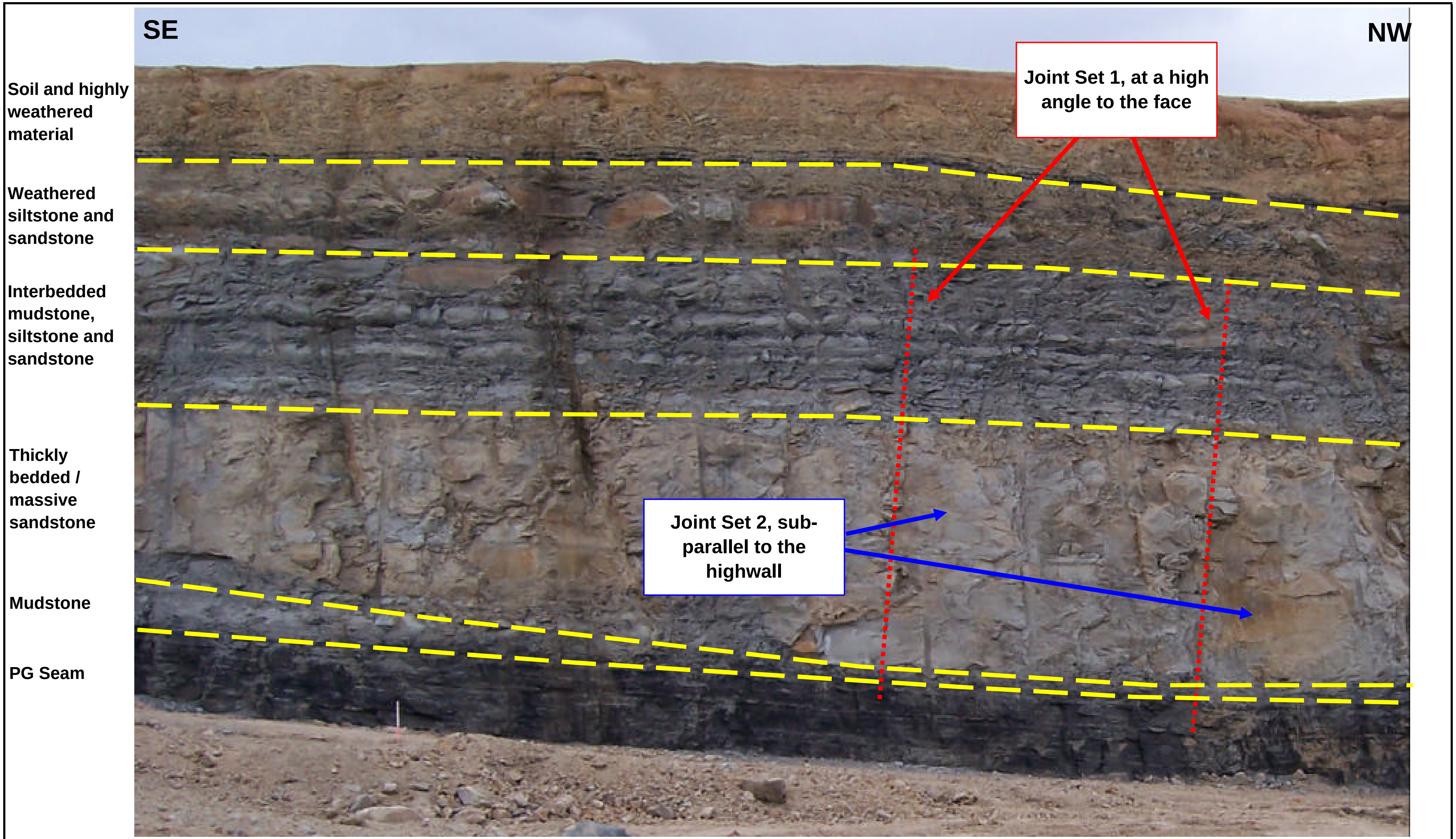
Key:

Shale / Siltstone   Sandstone

Soil / Clay   Coal

Note:
Strength and competency estimates derived from geophysical logs for WML101

Engineer:	D.Hill	CLIENT:	ASHTON MINE	**STRATA ENGINEERING (Australia Pty Ltd)**
Drawn:	K.McTyer		R04-001-ASH-4	
Date:	30.12.05	TITLE:	SSW - NNE SECTION ABOVE THE PIKES GULLY SEAM THROUGH THE ASHTON PORTAL ENTRIES AREA	FIGURE 14
Scale:	NTS			



ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**
DATE:	31.12.05	TITLE:	Arties Pit at Ashton, showing the Jointing in the Highwall	FIGURE
SCALE:	NTS		and the Roof Profile above the Pikes Gully Seam	15

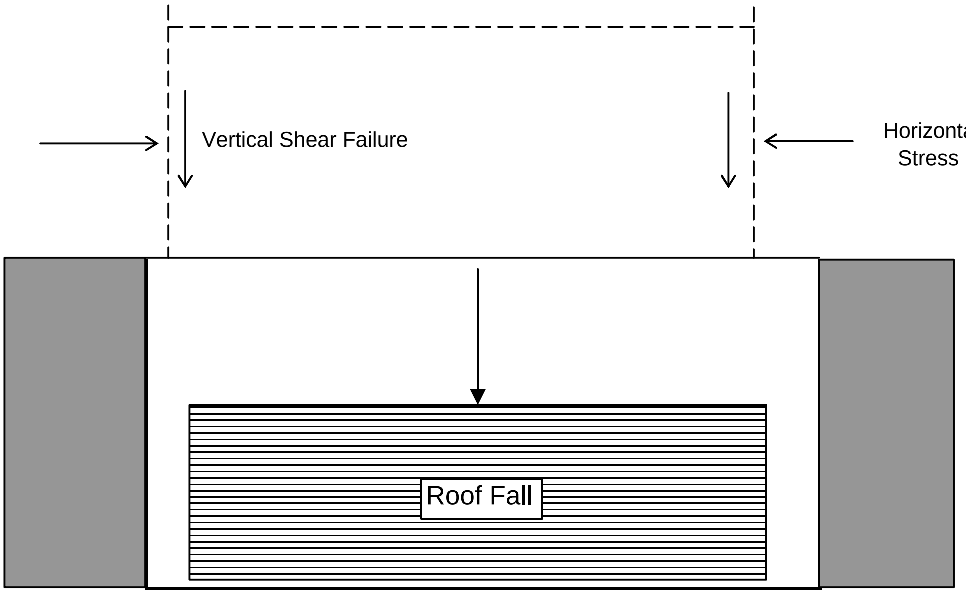


FIGURE 16a: Schematic Illustration of General Condition Under which a Roof Fall will Occur

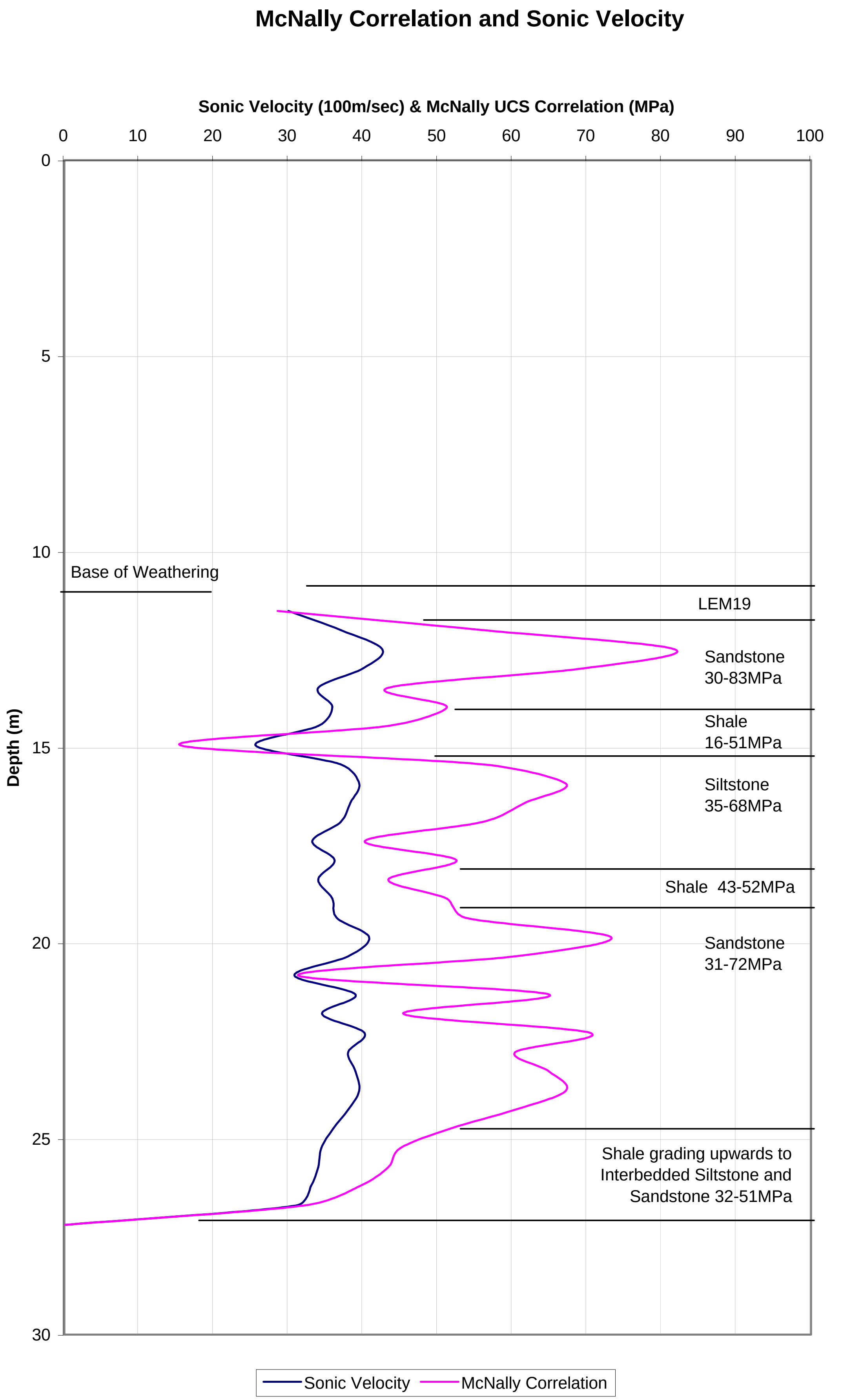
Centre brick will become more liable to fall out as :

1. Horizontal restraint is reduced
2. Shear properties (ie cohesion and friction) of contact surface reduces
3. Weight of centre brick increases
4. Inclination of shear surfaces reduces
5. Centre brick is too thin and undergoes uncontrolled buckling and subsequent failure under the action of the horizontal restraint

If centre brick becomes free it can only then be held by a vertical force equal to the weight of the brick

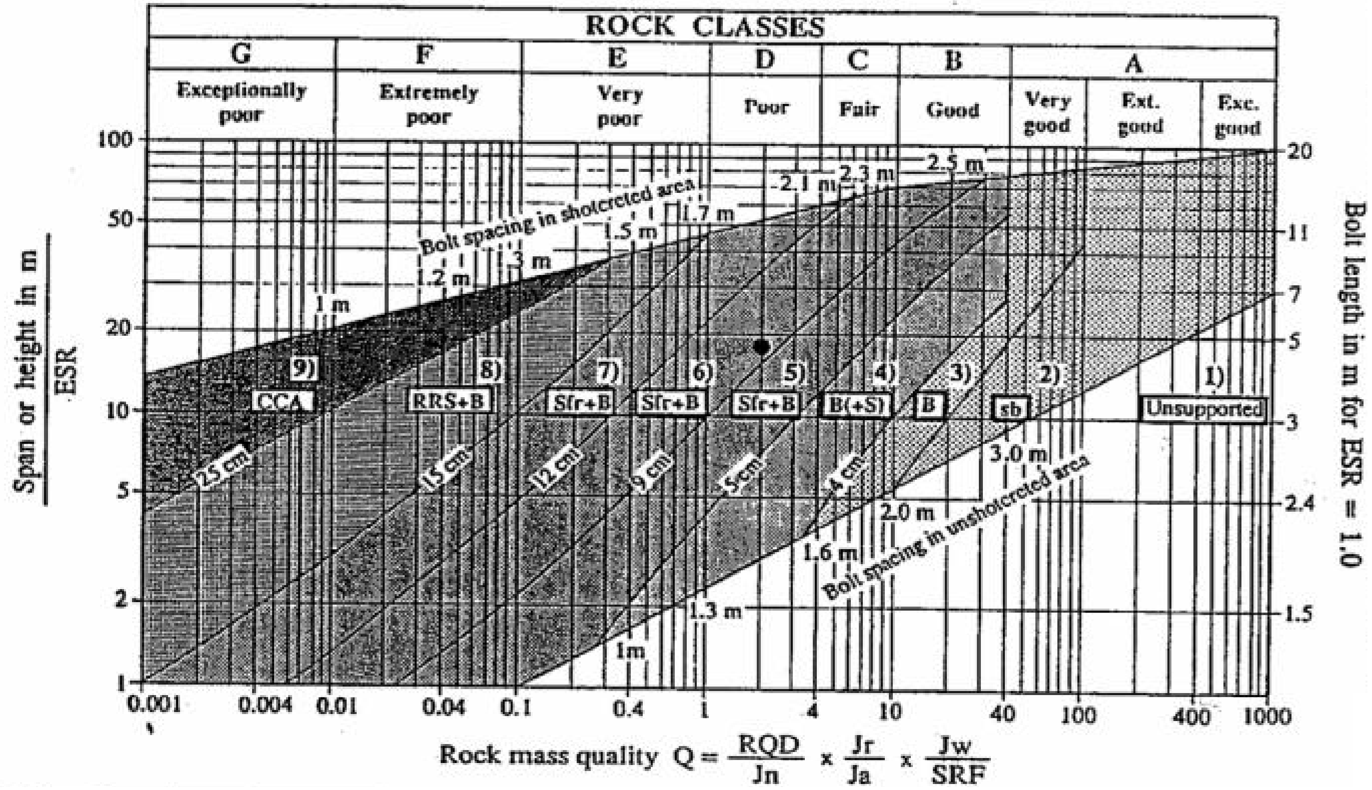
FIGURE 16b: Roof Fall Analogy with "3 Bricks"

ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING (Australia) Pty Ltd**
DRAWN:	D. Hill		R04-001-ASH-4	
DATE:	31.12.05	TITLE:	Roof Fall Model	FIGURE: 16a/b
SCALE:	NTS			



ENGINEER:	D. Hill	CLIENT:	ASHTON MINE	STRATA ENGINEERING	
DRAWN:	K. McTyer		R04-001-ASH-4	(Australia) Pty Ltd	
DATE:	03.01.06	TITLE:	Sonic to Strength Correlation		FIGURE
SCALE:	NTS		for WML 101		17

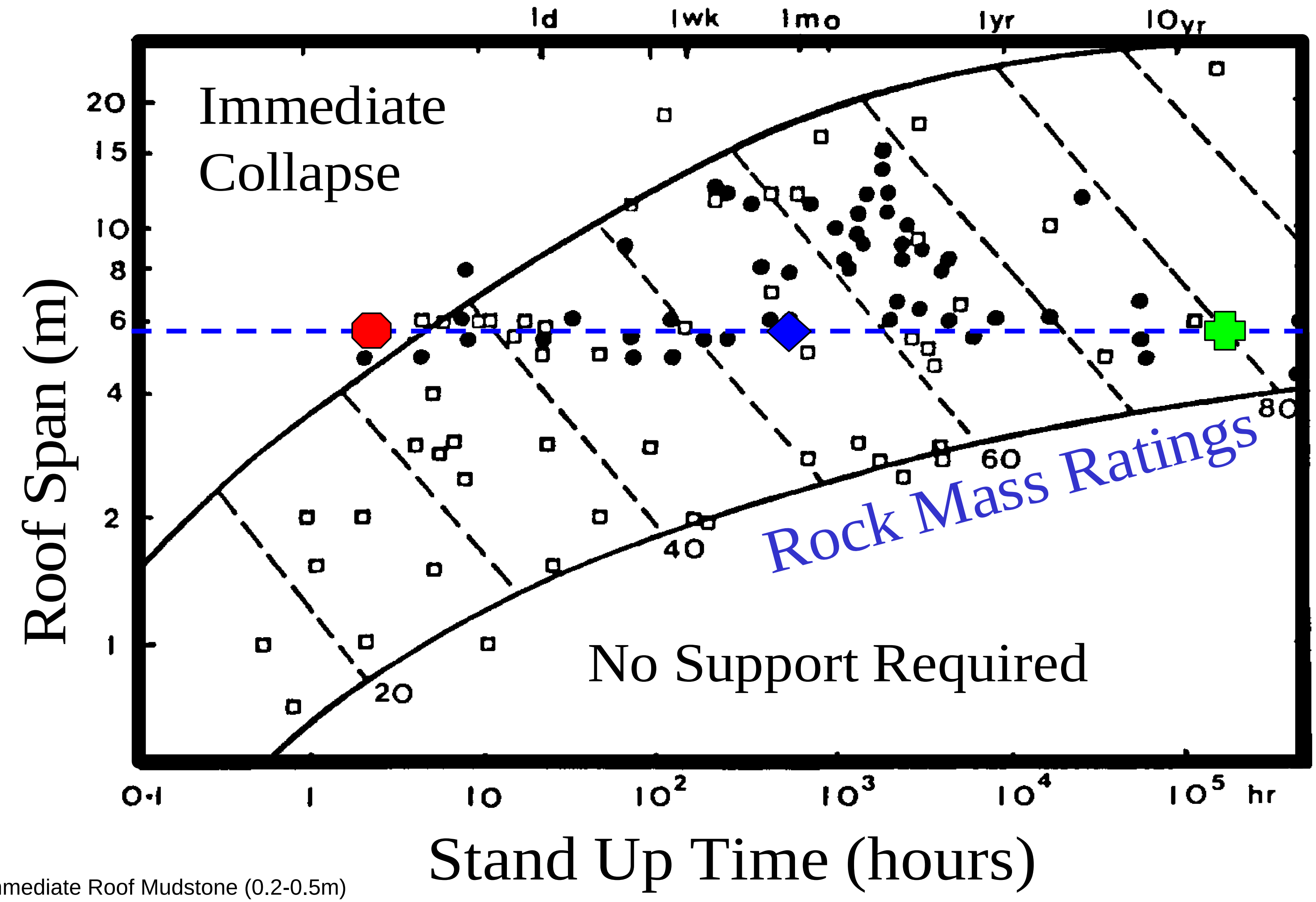
ROCK MASS CLASSIFICATION



REINFORCEMENT CATEGORIES:

1) Unsupported
2) Spot bolting, sb
3) Systematic bolting, B
4) Systematic bolting, (and unreinforced shotcrete, 4-10 cm), B(+S)
5) Fibre reinforced shotcrete and bolting, 5-9 cm, Sfr+B
6) Fibre reinforced shotcrete and bolting, 9-12 cm, Sfr+B
7) Fibre reinforced shotcrete and bolting, 12-15 cm, Sfr+B
8) Fibre reinforced shotcrete >15 cm, reinforced ribs of shotcrete and bolting, Sfr,RRS+B
9) Cast concrete lining, CCA

ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**	
DATE:	04.04.05	TITLE:	NGI Q System Support Design Nomogram		FIGURE
SCALE:	NTS		(**Grimstad and Barton, 1993**)		18



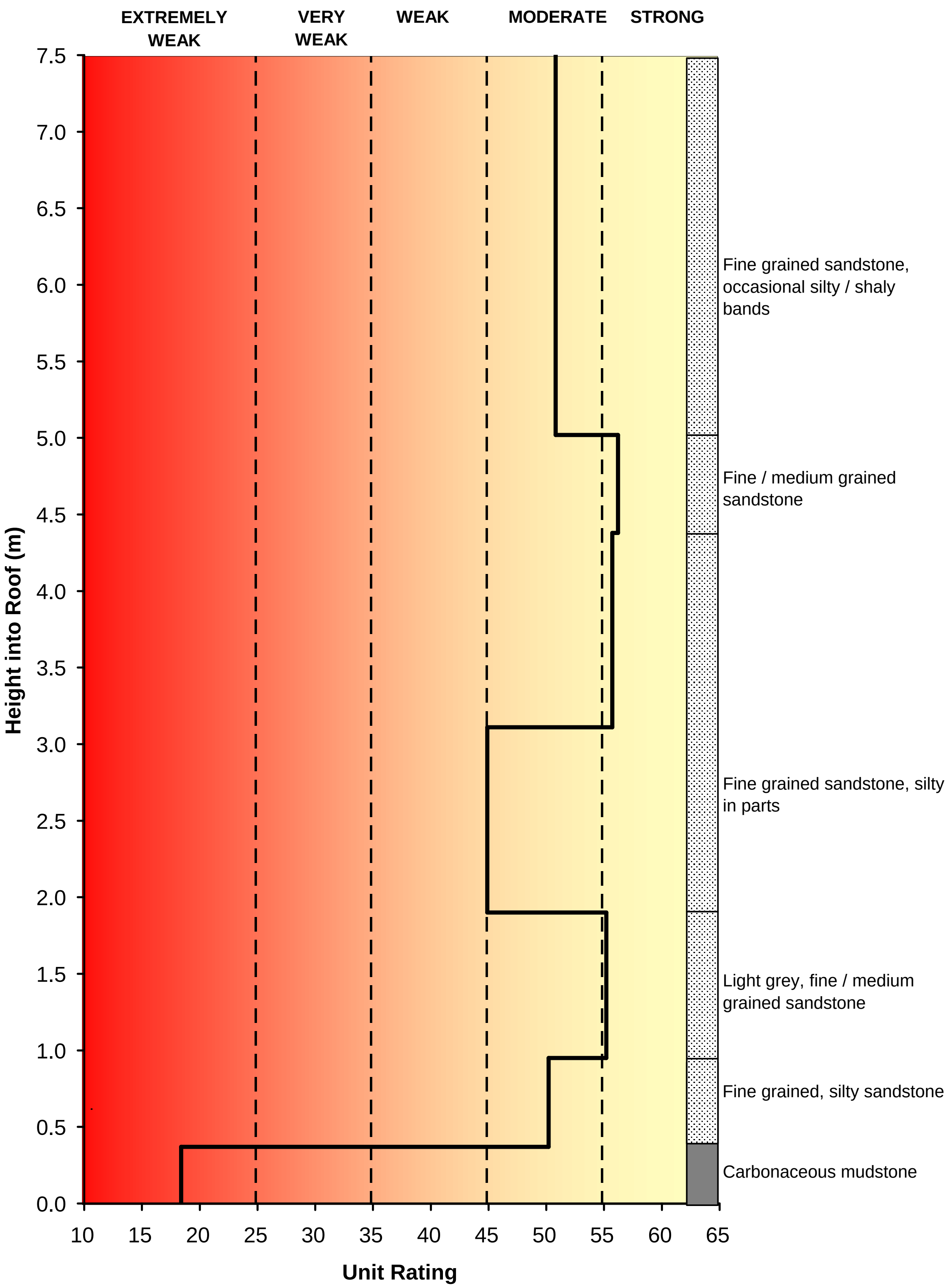
Key:

Immediate Roof Mudstone (0.2-0.5m)

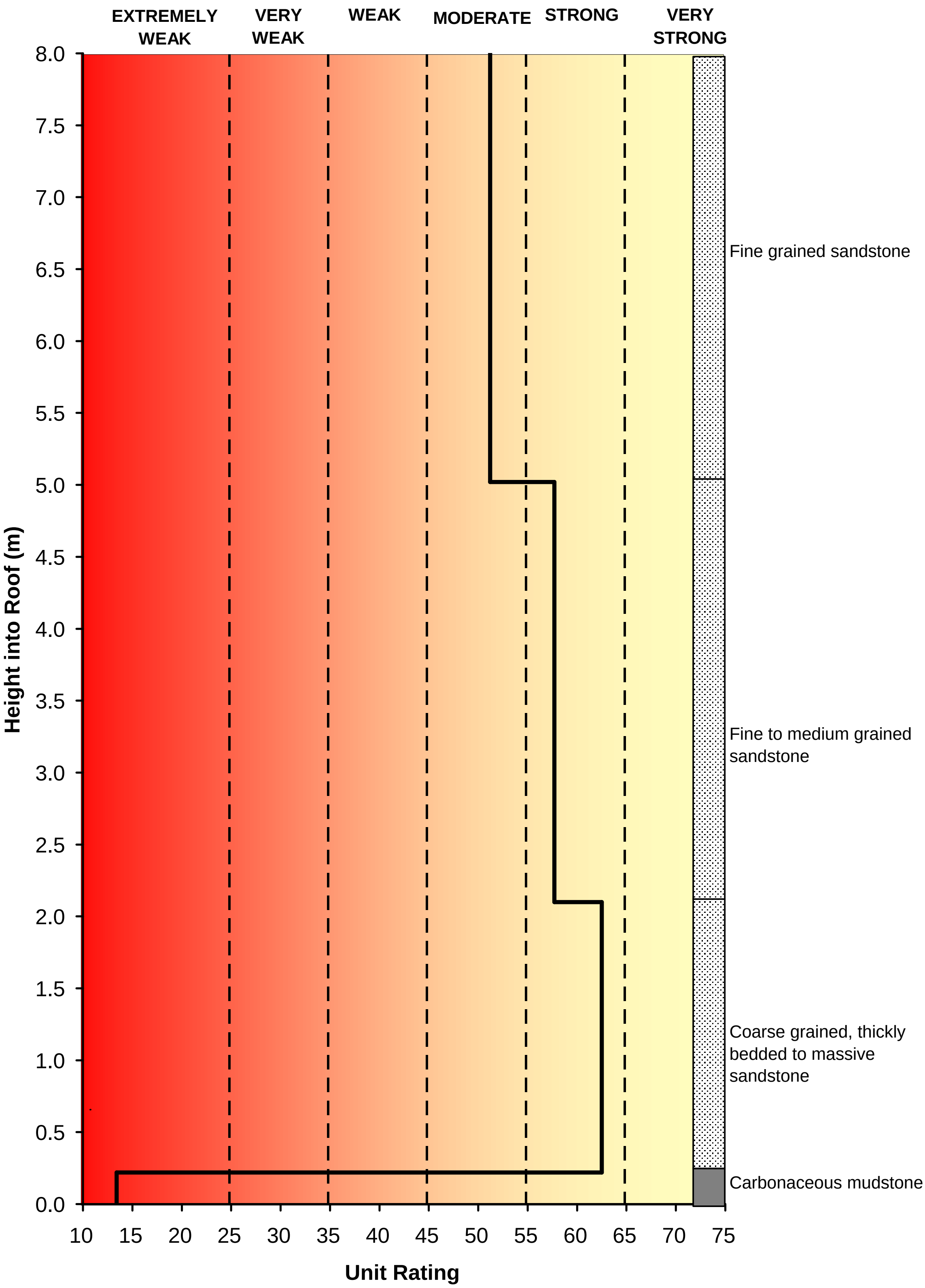
Interbedded Siltstone and Sandstone (Outbye)

Main Roof Sandstone

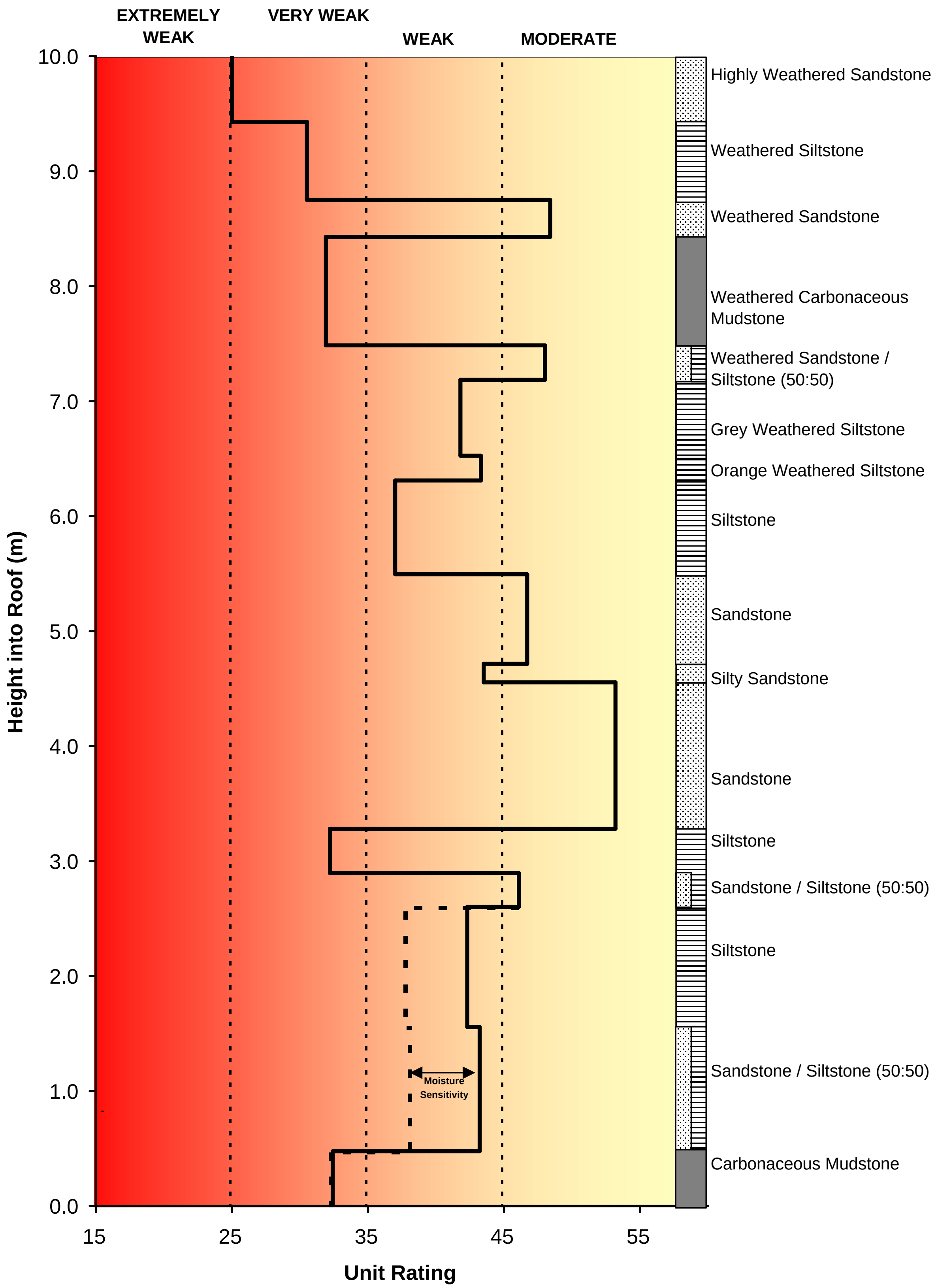
ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**	
DATE:	03.01.06	TITLE:	CSIR RMR Nomogram showing Ashton Results		FIGURE
SCALE:	NTS		for a Span of 5.5m		19



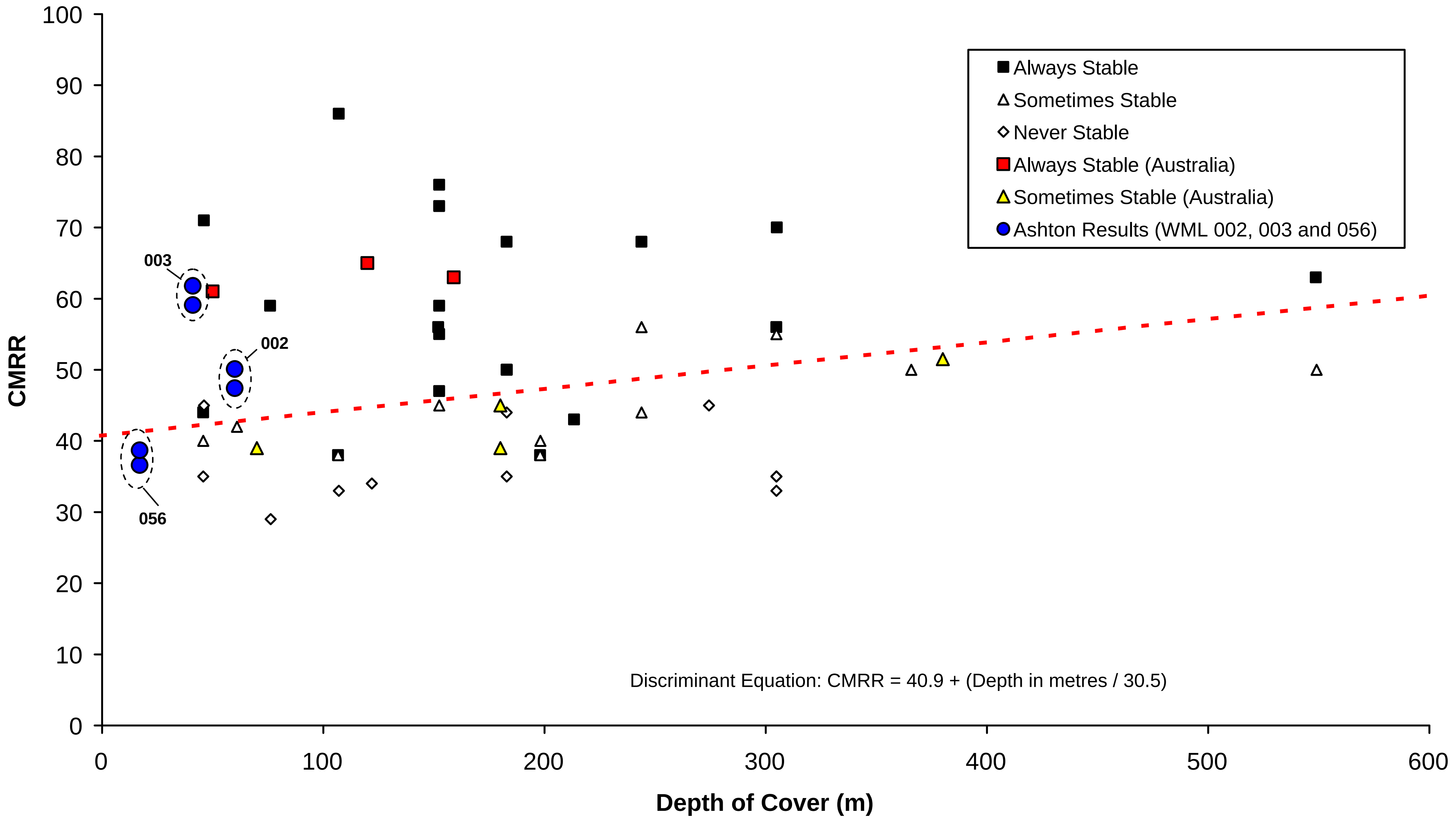
ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**
DRAWN:	D. Hill		R04-001-ASH-1	**(Australia) Pty Ltd**
DATE:	3.01.06	TITLE:	WML 002 Roof Unit Rating Profile	FIGURE
SCALE:	NTS		(Pikes Gully Seam)	20



ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**
DRAWN:	D. Hill		R04-001-ASH-1	**(Australia) Pty Ltd**
DATE:	3.01.06	TITLE:	WML 003 Roof Unit Rating Profile	FIGURE
SCALE:	NTS		(Pikes Gully Seam)	21

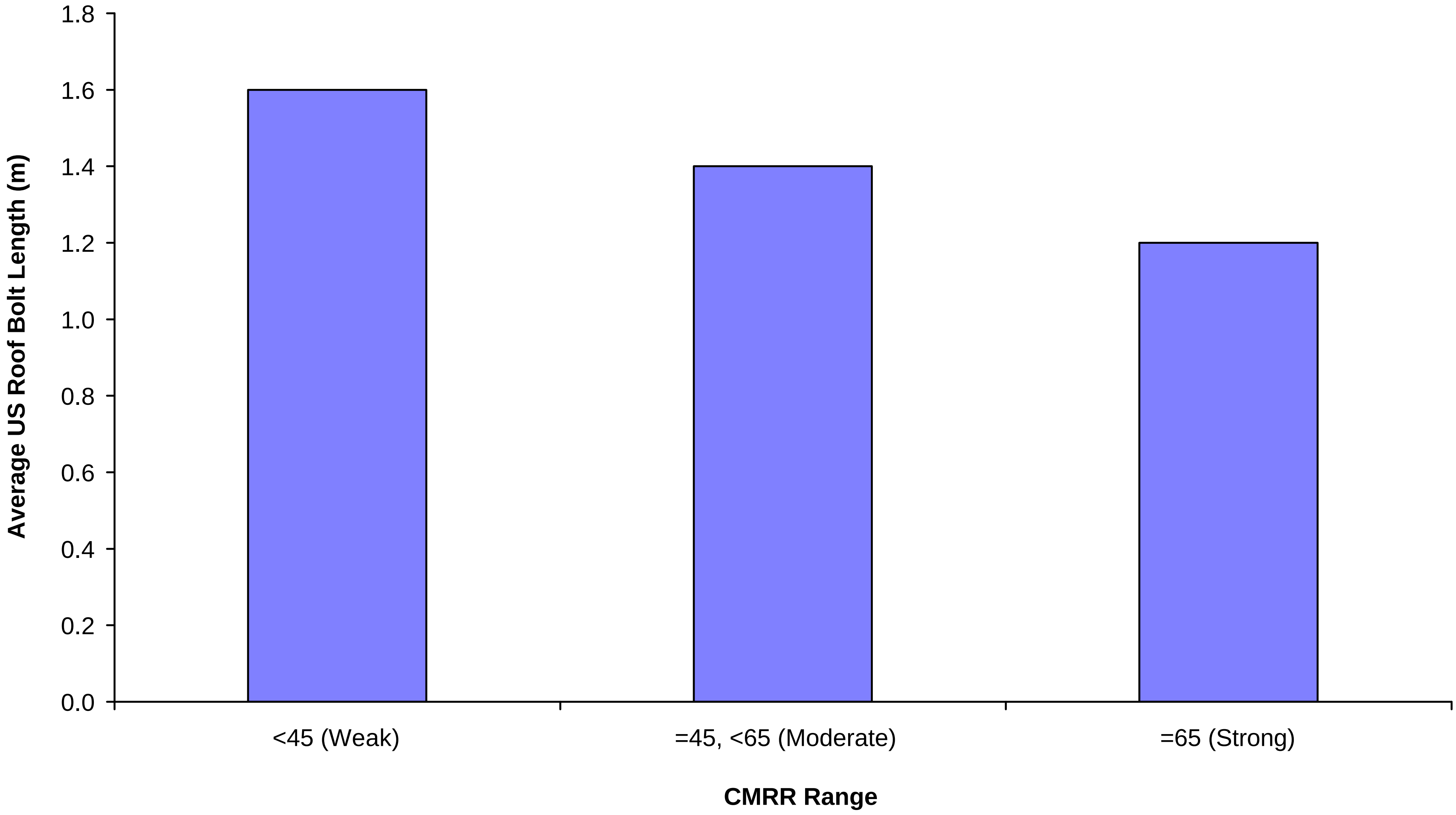


ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**
DRAWN:	D. Hill		R04-001-ASH-1	**(Australia) Pty Ltd**
DATE:	21.03.05	TITLE:	WML 056 Roof Unit Rating Profile	FIGURE
SCALE:	NTS		(Pikes Gully Seam)	22

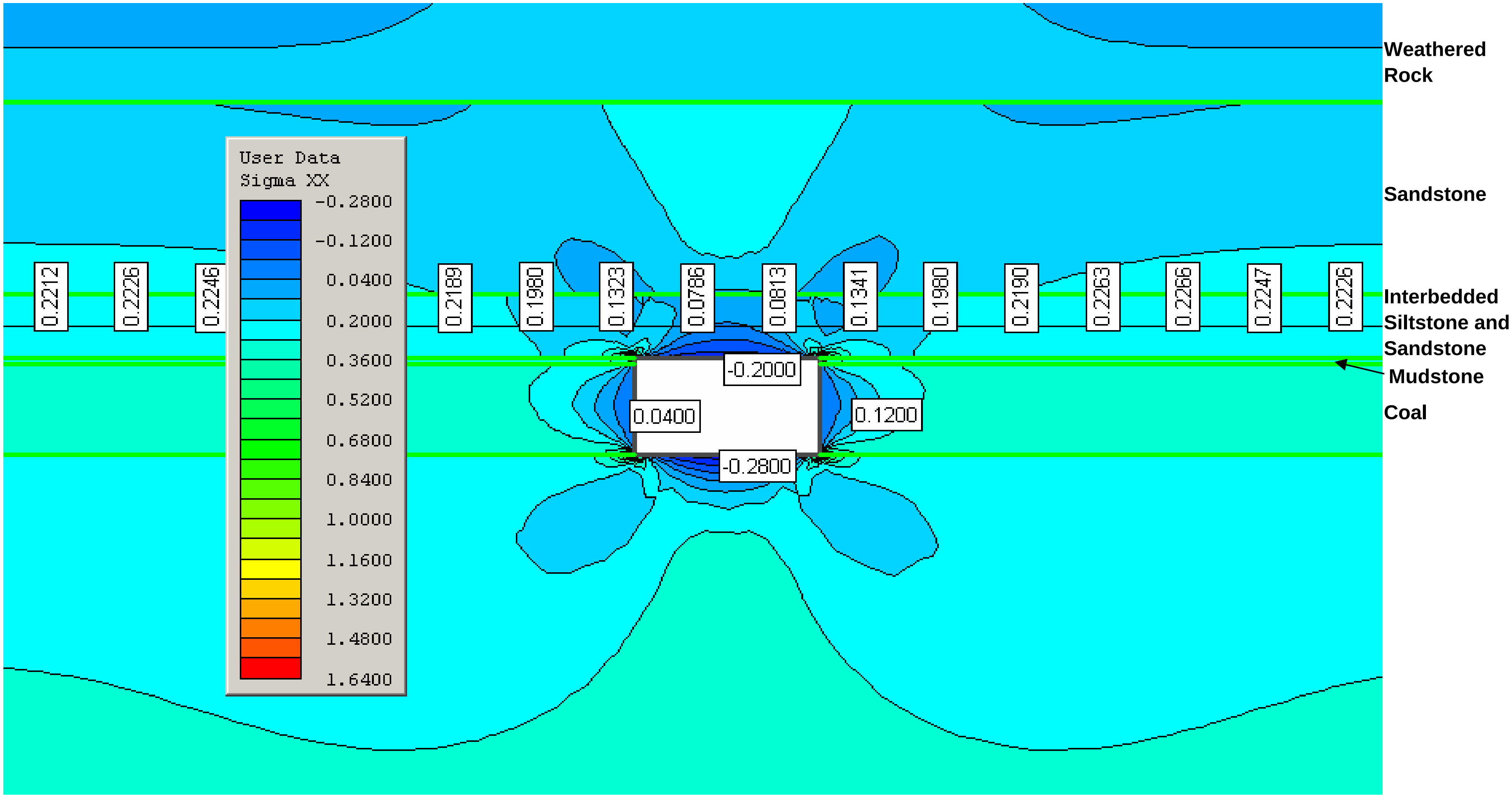


Note: Ashton paired results relate to "roof as is " versus
0.2m of mudstone cut down, for 2.1m long bolts

ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**
DRAWN:	D. Hill		R04-001-ASH-1	**(Australia) Pty Ltd**
DATE:	3.01.06	TITLE:	CMRR vs Extended Cut Database showing Ashton	FIGURE
SCALE:	NTS		Bore Core Results for WML 002, 003 and 056	23

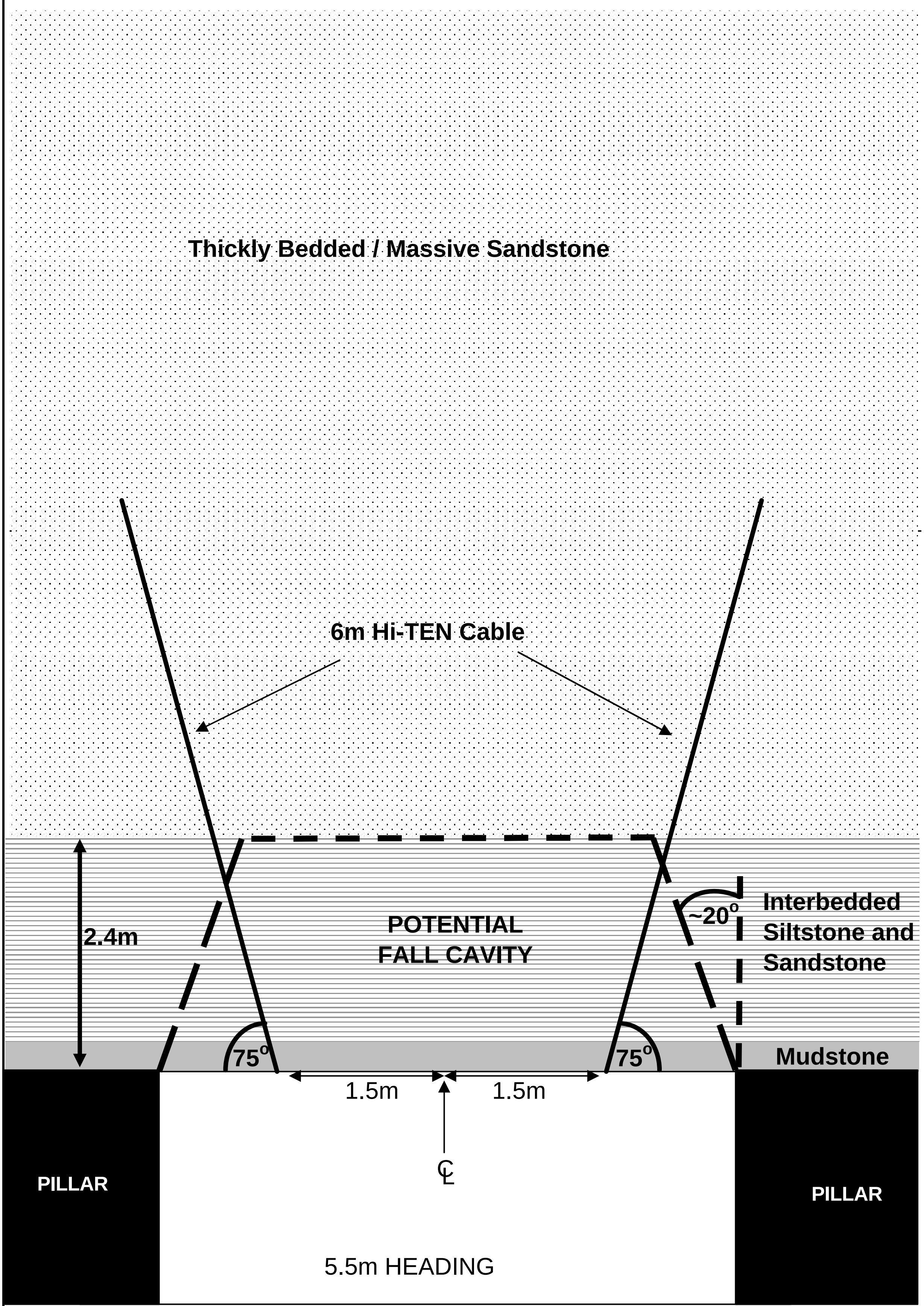


ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-1	**(Australia) Pty Ltd**	
DATE:	3.01.06	TITLE:	Average US Roof Bolt Length versus CMRR derived		FIGURE
SCALE:	NTS		from the NIOSH Database (**Molinda *et al* , 2000**)		24

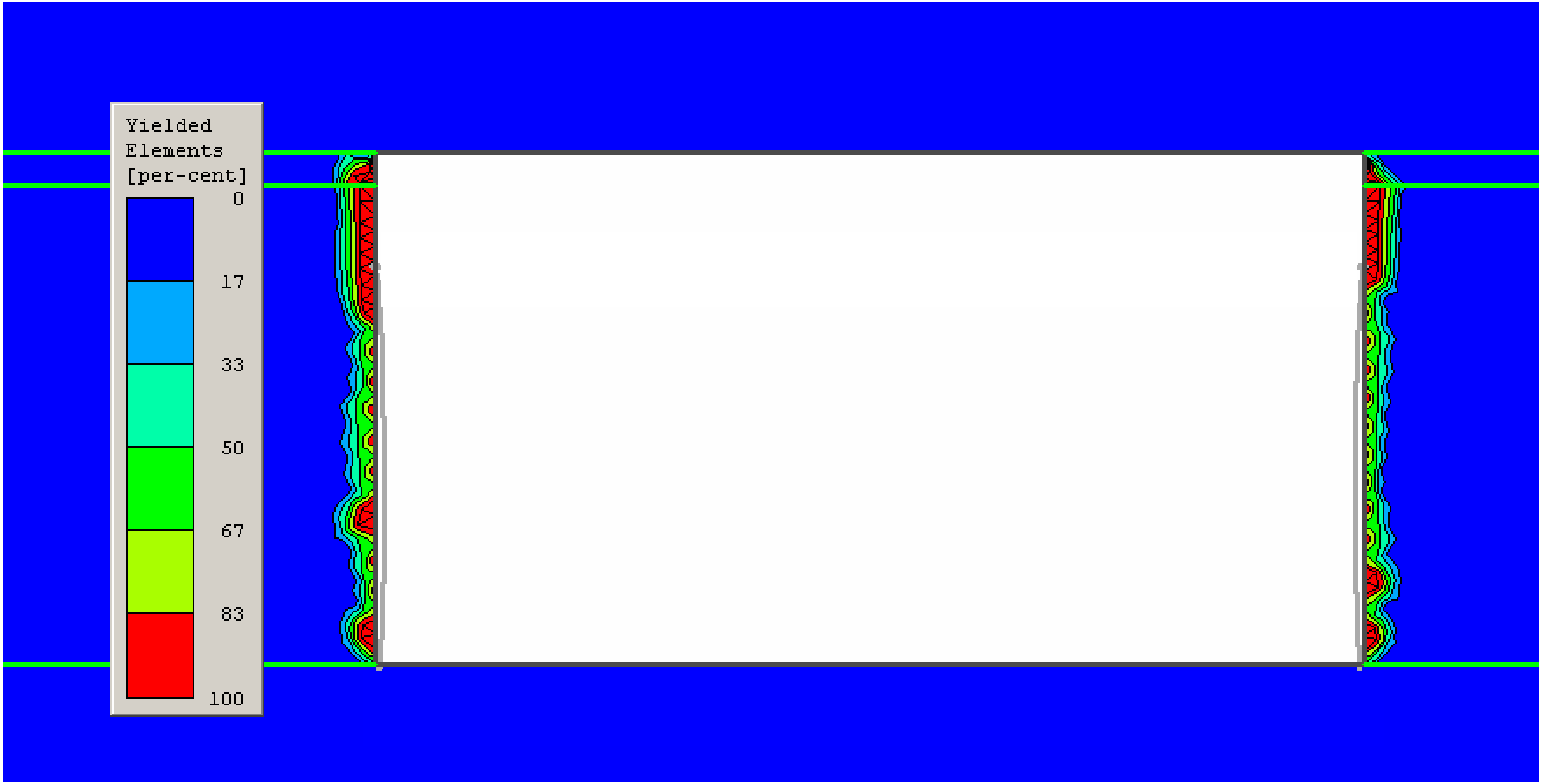


Note: Negative = tension zone

ENGINEER:	J. Jiang	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**	
DATE:	03.01.06	TITLE:	Finite Element Model Showing Horizontal Stress		FIGURE
SCALE:	NTS		Contours for the Ashton Entries at Shallow (27m) Depth		25

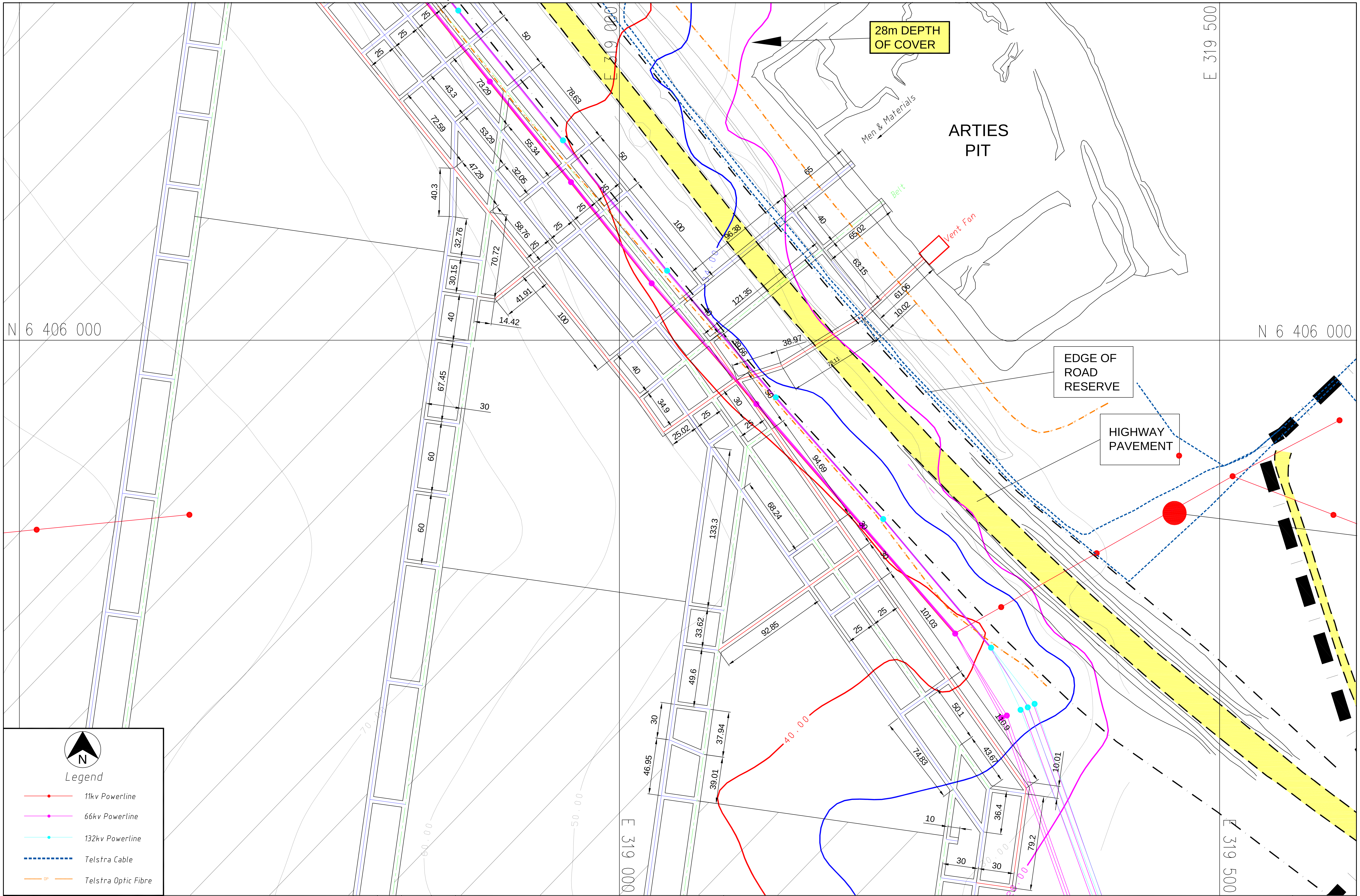


ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**
DATE:	3.01.06	TITLE:	Potential Roof Fall Geometry in Mudstone and	FIGURE
SCALE:	NTS		Interbedded Siltstone / Sandstone Units	26



ENGINEER:	J. Jiang	CLIENT:	Ashton Mine R04-001-ASH-4	**STRATA ENGINEERING (Australia) Pty Ltd**
DRAWN:	D. Hill			
DATE:	03.01.06	TITLE:	Finite Element Model Showing Limited Extent of Yielded Rib Elements	FIGURE 27
SCALE:	NTS			

APPENDIX A: MINE PLAN



REV.	DATE	BY	DESCRIPTION	CHK.	REV.	DATE	BY	DESCRIPTION	CHK.

REVISIONS

AshtonCoal

PO Box 699 Singleton NSW 2330
Phone 61+ 02 6576 1111 Fax 61+ 02 6576 1122

Ashton Underground Mine
Proposed Layout Main Headings - Option 1
Sheet 2 of 3 - Dimensions

Drawing No.	Revision No.	Date	Drawn	Checked	Approved	Sheet Size
01064	A	17.11.05	JR	BW	BW	A2

APPENDIX B: STRATA ENGINEERING'S RISK MODEL

Strata Engineering utilise a basic risk model in order to assess risk levels associated with strata control type hazards. It is based on standard risk assessment practices and is described herein for reference purposes.

Risk = Probability x Consequence

Therefore, the basic requirement of the risk model is for a value to be assigned for both the probability and consequence of a given event occurring and for these two values to be combined in such a way to give a measure of the risk level.

Probability Ranking

In most engineering design, it is difficult to give absolute probabilities for certain events occurring, primarily due to the relatively small database (*assuming one exists at all*) associated with the many possible hazards under consideration. Hence, the assignment of a probability value is more a qualitative type assessment, although each category is assigned what is assessed to be an equivalent probability range.

Probability Ranking	Qualitative Assessment that a Certain Event will Occur	Equivalent Probability Range
1	Almost certain	> 90%
2	Likely	50 to 90 %
3	Possible	10 to 50 %
4	Unlikely	1 to 10 %
5	Practically impossible	< 1%

Note: whenever the probability of an event occurring is considered, it is vital to define whether it is being considered in general terms or at a specific location whereby anomalous conditions are known to exist. Similarly, it is also important to clearly define whether any controls were assumed to be in place or not.

Consequence Ranking

In the same way that a ranking is assigned to the probability of a certain event occurring, the consequence of that event must also be ranked. It is usual to consider both the business consequence and safety consequence separately and then utilise the higher ranked of the two in defining an overall risk level. As with probability, the rankings are essentially qualitative in nature, although in the case of business consequence, it is categorised according to the period of lost longwall face production.

Consequence Ranking	Business Loss (Lost Face Production)	Safety
1	> 1 month	Fatality
2	1 week to 1 month	Serious LTI
3	1 day to 1 week	LTI
4	1 shift to 1 day	First Aid
5	< 1 shift	Near-miss

It is also possible to consider other business loss parameters (eg capital equipment loss, additional operating cost etc.) according to the hazard being considered.

When ranking a consequence, the general approach should be taken that when the event has occurred, with the existing controls in place, what is the worst case scenario?

Combined Risk Ranking

The probability and consequence rankings are combined according to the following table :

Probability→ Consequence ↓	**1**	**2**	**3**	**4**	**5**
1	**1**	**2**	**4**	**7**	**11**
2	**3**	**5**	**8**	**12**	**16**
3	**6**	**9**	**13**	**17**	**20**
4	**10**	**14**	**18**	**21**	**23**
5	**15**	**19**	**22**	**24**	**25**

Note:
1 to 10 – unacceptable residual risk
11 to 15 – acceptable residual risk

It is these risk rankings that are used to define residual risk levels (*with specified controls in place*) and the need for further controls (ie type and level) that must be put in place (*over and above existing controls which may be none in some instances*) to ensure that residual business risks are maintained at acceptably low levels.

APPENDIX C: 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.

C.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.

C.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 C1**.

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.

C.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 C2**.

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.

C.2.2 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 C1a

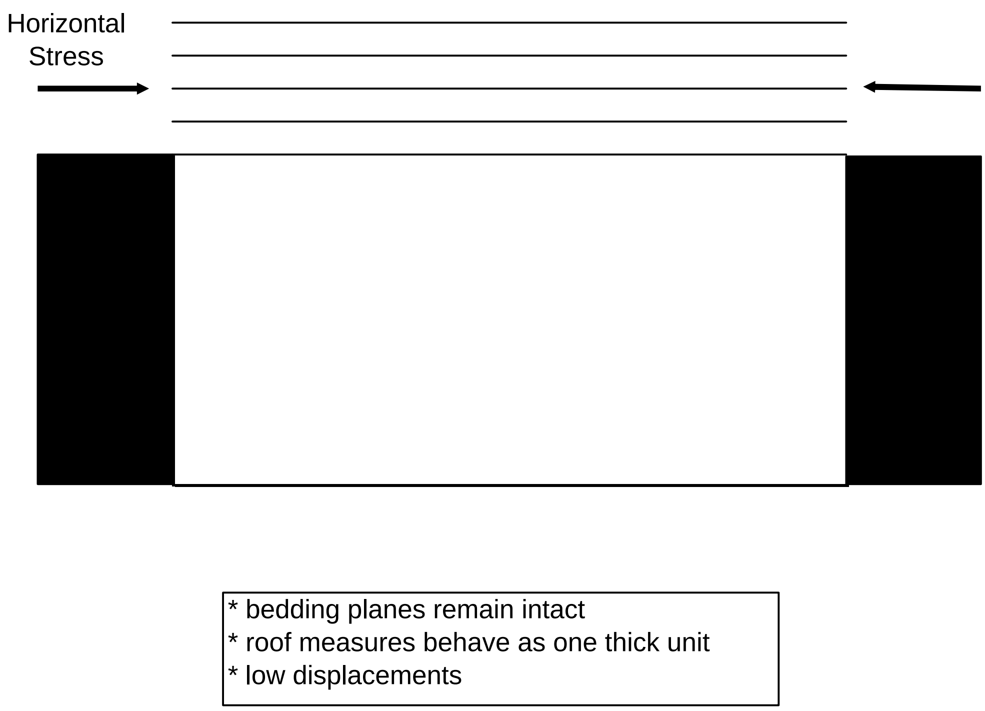
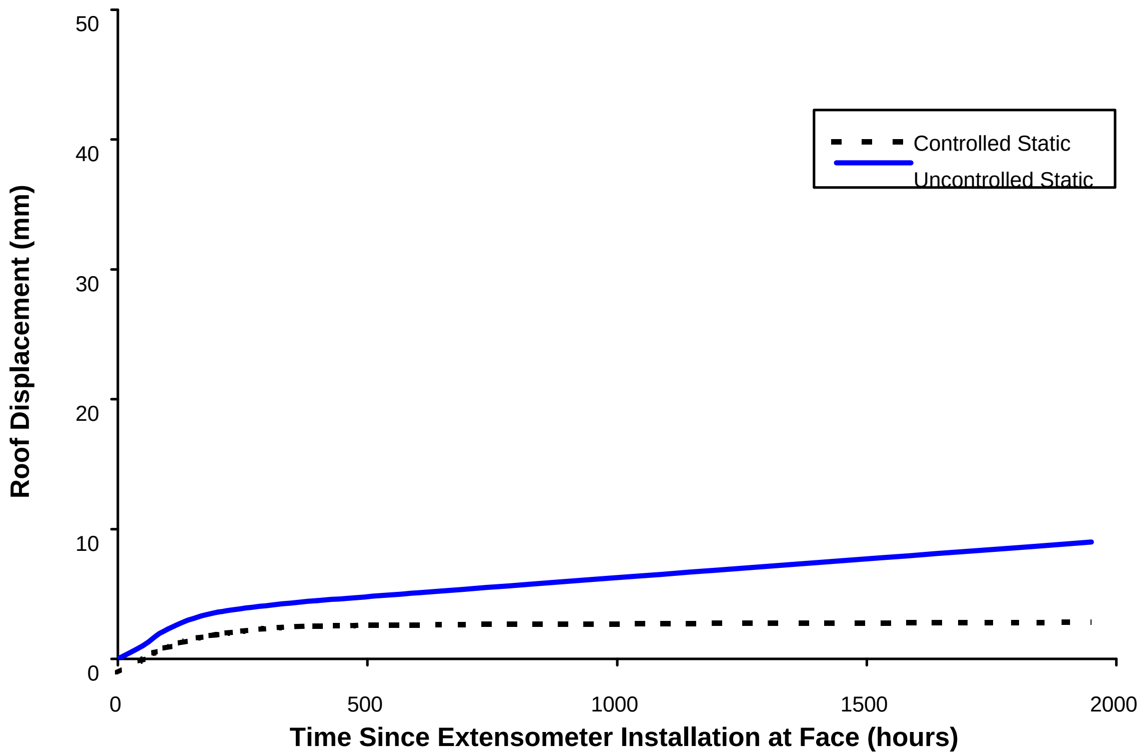
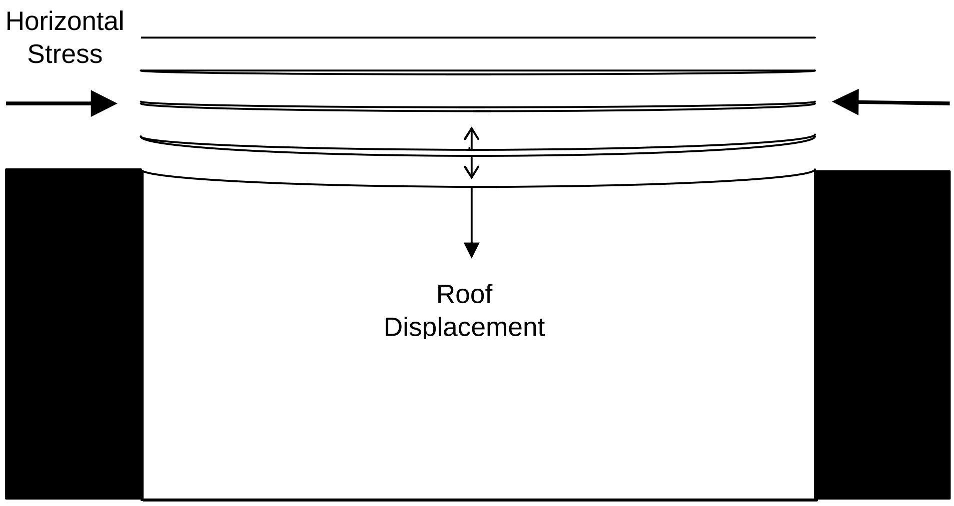


Figure C1b



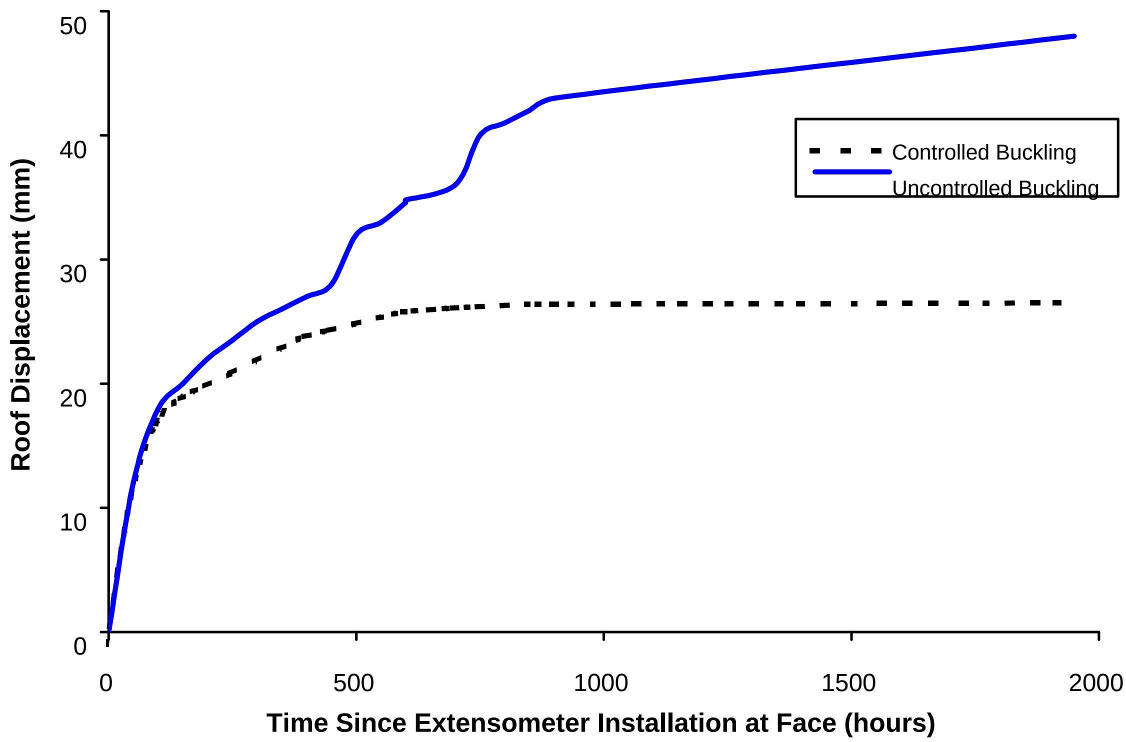
ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING (Australia) Pty Ltd**	
DRAWN:	D. Hill		R04-001-ASH-4		
DATE:	04.11.03	TITLE:	Schematic Illustration of Static Roof and Typical Time Dependent Displacement Trends		FIGURE: C1a/b
SCALE:	NTS				

Figure C2a



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

Figure C2b



ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**
DRAWN:	D. Hill		R04-001-ASH-4	**(Australia) Pty Ltd**
DATE:	04.11.03	TITLE:	Schematic Illustration of Buckling Roof and Typical	FIGURE:
SCALE:	NTS		Time Dependent Displacement Trends	C2a/b

APPENDIX D: CMRR UNIT RATING RESULTS

COAL MINE ROOF RATING

UNIT RATING SHEET

Colliery: Ashton
Location: WML 02
Date: 29.12.05
Engineer: DH
Details:

ENTER UNIT No: 1

Enter Unit Description: 0.37m of carbonaceous mudstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	32
Enter Fracture Spacing (mm)	93
Discontinuity Spacing Rating	24.9

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0
Diametral Strength Rating	25

3. Minimum of DSR and Diametral PLT Strength 24.9

4. Calculate UCS Rating

Enter First Axial IS_{50}	0.32
UCS (MPa)	6.72

UCS Rating 6.5

5. Unit Moisture Sensitivity -13

UNIT RATING (UR) 18.4

ENTER UNIT No: 2

Enter Unit Description: 0.58m of sandstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	193
Discontinuity Spacing Rating	35.5

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	2.4
Diametral Strength Rating	70.16

3. Minimum of DSR and Diametral PLT Strength 35.5

4. Calculate UCS Rating

Enter First Axial IS_{50}	2.5
UCS (MPa)	52.5

UCS Rating 14.7

5. Unit Moisture Sensitivity 0

UNIT RATING (UR) 50.2

ENTER UNIT No: 3

Enter Unit Description: 0.95m of sandstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	190
Discontinuity Spacing Rating	35.4

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	3.80
Diametral Strength Rating	99.42

3. Minimum of DSR and Diametral PLT Strength 35.4

4. Calculate UCS Rating

Enter First Axial IS_{50}	4.4
UCS (MPa)	92.4

UCS Rating 19.8

5. Unit Moisture Sensitivity 0

UNIT RATING (UR) 55.2

ENTER UNIT No: 4

Enter Unit Description: 1.21m of silty sandstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	134
Discontinuity Spacing Rating	33.4

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.65
Diametral Strength Rating	33.59

3. Minimum of DSR and Diametral PLT Strength 33.4

4. Calculate UCS Rating

Enter First Axial IS_{50}	1.4
Enter Second Axial IS_{50}	1.44
UCS (MPa)	29.82

UCS Rating 11.5

UNIT RATING (UR) 44.9

ENTER UNIT No: 5

Enter Unit Description: 1.27m of silty sandstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	89
Enter Fracture Spacing (mm)	180
Discontinuity Spacing Rating	35.1

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	1.5
Diametral Strength Rating	51.35

3. Minimum of DSR and Diametral PLT Strength **35.1**

4. Calculate UCS Rating

Enter First Axial IS_{50}	4.7
UCS (MPa)	98.7

UCS Rating **20.6**

UNIT RATING (UR) 55.7

ENTER UNIT No: 6

Enter Unit Description: 0.64m of fine / medium grained sandstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	210
Discontinuity Spacing Rating	36.0

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	4.6
Diametral Strength Rating	116.14

3. Minimum of DSR and Diametral PLT Strength **36.0**

4. Calculate UCS Rating

Enter First Axial IS_{50}	4.1
Enter Second Axial IS_{50}	5
UCS (MPa)	95.55

UCS Rating **20.2**

UNIT RATING (UR) 56.2

ENTER UNIT No: 7

Enter Unit Description: 2.75m of fine grained sandstone; occasional silty / shaly bands

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	95
Enter Fracture Spacing (mm)	183
Intermediate Rating	35.2
Discontinuity Spacing Rating	35.2

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	1.45
Diametral Strength Rating	50.31

3. Minimum of DSR and Diametral PLT Strength 35.2

4. Calculate UCS Rating

Enter First Axial IS_{50}	3.2
Enter Second Axial IS_{50}	2.5
UCS (MPa)	59.85

UCS Rating 15.7

UNIT RATING (UR) 50.8

COAL MINE ROOF RATING

UNIT RATING SHEET

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

COAL MINE ROOF RATING

UNIT RATING SHEET WITH ALL MOISTURE SENSITIVITY RESULTS

Colliery:	Ashton
Location:	WML 056
Date:	18.03.05
Engineer:	DH
Details:	Portal Geotechnical Hole - Pikes Gully Seam Roof (First 5 Units) (note immediate roof mudstone broken into Units 1a and 1b)

ENTER UNIT No: 1a

Enter Unit Description: 0.09m of Dessicating Carbonaceous Mudstone (Immediate PG Roof)

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	1
Enter Fracture Spacing (mm)	90
Intermediate Rating	-11.6
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}	0.31
UCS (MPa)	6.51

UCS Rating 6.4

5. Unit Moisture Sensitivity -12

UNIT RATING (UR) 14.4

ENTER UNIT No: 1b

Enter Unit Description: 0.475m of Carb. Mudstone (balance of immediate PG roof unit)

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	86
Enter Fracture Spacing (mm)	220
Intermediate Rating	36.2
Discontinuity Spacing Rating	36.2

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.4
Diametral Strength Rating	28.36

3. Minimum of DSR and Diametral PLT Strength 28.4

4. Calculate UCS Rating

Enter First Axial IS_{50}	1.1
UCS (MPa)	23.1

UCS Rating 10.0

5. Unit Moisture Sensitivity -6

UNIT RATING (UR) 32.4

ENTER UNIT No: 2

Enter Unit Description: 1.08m of interbedded sandstone and siltstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	74
Enter Fracture Spacing (mm)	250
Intermediate Rating	33.6
Discontinuity Spacing Rating	33.6

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.67
Diametral Strength Rating	33.93333

3. Minimum of DSR and Diametral PLT Strength 33.6

4. Calculate UCS Rating

Enter First Axial IS_{50}	1.3
Enter Second Axial IS_{50}	0.7
Average of PL Tests (UCS)	21
UCS (MPa)	21

UCS Rating 9.6

5. Unit Moisture Sensitivity -5

UNIT RATING (UR) 38.2

ENTER UNIT No: 3

Enter Unit Description: 1.045m of grey siltstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	90
Enter Fracture Spacing (mm)	348
Intermediate Rating	38.8
Discontinuity Spacing Rating	38.8

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.6
Diametral Strength Rating	32.54

3. Minimum of DSR and Diametral PLT Strength 32.5

4. Calculate UCS Rating

Enter First Axial IS_{50}	1
Enter Second Axial IS_{50}	0.9
Average of PL Tests (UCS)	19.95
Enter Laboratory UCS	24.1
UCS (MPa)	22.025

UCS Rating 9.8

5. Unit Moisture Sensitivity -15

UNIT RATING (UR) 27.3

ENTER UNIT No: 4

Enter Unit Description: 0.295m of sandstone / siltstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	295
Intermediate Rating	37.9
Discontinuity Spacing Rating	37.9

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	1.4
Diametral Strength Rating	49.26

3. Minimum of DSR and Diametral PLT Strength 37.9

4. Calculate UCS Rating

Enter First Axial IS_{50}	0.7
UCS (MPa)	14.7

UCS Rating 8.2

UNIT RATING (UR) 46.1

ENTER UNIT No: 5

Enter Unit Description: 0.385m of slightly weathered siltstone / fine grained sandstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	26
Enter Fracture Spacing (mm)	64
Intermediate Rating	22.6
Discontinuity Spacing Rating	22.6

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0
Diametral Strength Rating	25

3. Minimum of DSR and Diametral PLT Strength 22.6

4. Calculate UCS Rating

Enter First Axial IS_{50}	1
UCS (MPa)	21

UCS Rating 9.6

UNIT RATING (UR) 32.2

COAL MINE ROOF RATING

UNIT RATING SHEET

CMRR = **#DIV/0!**

Colliery:	Ashton
Location:	WML 056
Date:	18.03.05
Engineer:	DH
Details:	Portal Geotechnical Hole - Pikes Gully Seam Roof (Units 6 to 11)

ENTER UNIT No: 6

Enter Unit Description: 1.275m of sandstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	425
Intermediate Rating	39.9
Discontinuity Spacing Rating	39.9

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	1
Diametral Strength Rating	40.9

3. Minimum of DSR and Diametral PLT Strength 39.9

4. Calculate UCS Rating

Enter First Axial IS_{50}	1.8
Enter Second Axial IS_{50}	2.1
Average of PL Tests (UCS)	40.95
UCS (MPa)	40.95

UCS Rating 13.2

UNIT RATING (UR) 53.2

ENTER UNIT No: 7

Enter Unit Description: 0.16m of dark grey sandstone; stained surfaces

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	160
Intermediate Rating	34.4
Discontinuity Spacing Rating	34.4

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.6
Diametral Strength Rating	32.54

3. Minimum of DSR and Diametral PLT Strength 32.5

4. Calculate UCS Rating

Enter First Axial IS_{50}	1.3
UCS (MPa)	27.3

UCS Rating 11.0

UNIT RATING (UR) 43.5

ENTER UNIT No: 8

Enter Unit Description: 0.78m of grey sandstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	59
Enter Fracture Spacing (mm)	195
Intermediate Rating	31.2
Discontinuity Spacing Rating	31.2

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	1.8
Diametral Strength Rating	57.62

3. Minimum of DSR and Diametral PLT Strength 31.2

4. Calculate UCS Rating

Enter First Axial IS_{50}	3
Average of PL Tests (UCS)	63
Enter Laboratory UCS	54.5
UCS (MPa)	58.75

UCS Rating 15.5

UNIT RATING (UR) 46.7

ENTER UNIT No: 9

Enter Unit Description: 0.815m of grey siltstone; stained surfaces

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	88
Enter Fracture Spacing (mm)	272
Intermediate Rating	37.4
Discontinuity Spacing Rating	37.4

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.4
Diametral Strength Rating	28.36

3. Minimum of DSR and Diametral PLT Strength 28.4

4. Calculate UCS Rating

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

UCS Rating 8.7

UNIT RATING (UR) 37.0

ENTER UNIT No: 10

Enter Unit Description: 0.215m of weathered orange siltstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	215
Intermediate Rating	36.1
Discontinuity Spacing Rating	36.1

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.7
Diametral Strength Rating	34.63

3. Minimum of DSR and Diametral PLT Strength 34.6

4. Calculate UCS Rating

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

UCS Rating 8.7

UNIT RATING (UR) 43.3

ENTER UNIT No: 11

Enter Unit Description: 0.66m of grey weathered siltstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	74
Enter Fracture Spacing (mm)	94
Intermediate Rating	33.6
Discontinuity Spacing Rating	33.6

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.9
Diametral Strength Rating	38.81

3. Minimum of DSR and Diametral PLT Strength 33.6

4. Calculate UCS Rating

Enter First Axial IS_{50}	0.7
UCS (MPa)	14.7

UCS Rating 8.2

UNIT RATING (UR) 41.8

COAL MINE ROOF RATING

UNIT RATING SHEET

Colliery:	Ashton
Location:	WML 056
Date:	18.03.05
Engineer:	DH
Details:	Portal Geotechnical Hole - Pikes Gully Seam Roof (Units 12 to 16)

ENTER UNIT No: 12

Enter Unit Description: 0.3m of grey fine grained sandstone / siltstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	100
Enter Fracture Spacing (mm)	300
Intermediate Rating	38.0
Discontinuity Spacing Rating	38.0

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.9
Diametral Strength Rating	38.81

3. Minimum of DSR and Diametral PLT Strength — 38.0

4. Calculate UCS Rating

Enter First Axial IS_{50}	1.1
UCS (MPa)	23.1

UCS Rating 10.0

UNIT RATING (UR) 48.0

ENTER UNIT No: 13

Enter Unit Description: 0.945m of carb. mudstone, sandy in parts

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	25
Enter Fracture Spacing (mm)	118
Intermediate Rating	22.4
Discontinuity Spacing Rating	22.4

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0.75
Diametral Strength Rating	35.675

3. Minimum of DSR and Diametral PLT Strength 22.4

4. Calculate UCS Rating

Enter First Axial IS_{50}	0.9	
Enter Second Axial IS_{50}	1.1	
Average of PL Tests (UCS)	21	
Enter Laboratory UCS	20.7	GTO2
UCS (MPa)	20.85	

UCS Rating 9.5

UNIT RATING (UR) 31.9

ENTER UNIT No: 14

Enter Unit Description: 0.32m of grey sandstone

1. Calculate the Discontinuity Spacing Rating

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

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	1.1
Diametral Strength Rating	42.99

3. Minimum of DSR and Diametral PLT Strength 38.3

4. Calculate UCS Rating

Enter First Axial IS_{50}	1.1
UCS (MPa)	23.1

UCS Rating 10.0

UNIT RATING (UR) 48.4

ENTER UNIT No: 15

Enter Unit Description: 0.68m of carbonaceous mudstone / siltstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	35
Enter Fracture Spacing (mm)	49
Intermediate Rating	25.8
Discontinuity Spacing Rating	25.8

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0
Diametral Strength Rating	25

3. Minimum of DSR and Diametral PLT Strength **25.0**

4. Calculate UCS Rating

Enter First Axial IS_{50}	0.1
UCS (MPa)	2.1

UCS Rating **5.5**

UNIT RATING (UR) 30.5

ENTER UNIT No: 16

Enter Unit Description: Highly weathered siltstone

1. Calculate the Discontinuity Spacing Rating

Enter RQD (%)	20	estimate
Enter Fracture Spacing (mm)	71	estimate
Intermediate Rating	19.9	
Discontinuity Spacing Rating	20.0	

2. Calculate Diametral PLT Strength Rating

Enter Diametral IS_{50}	0	estimate
Diametral Strength Rating	25	

3. Minimum of DSR and Diametral PLT Strength **20.0**

4. Calculate UCS Rating

Enter First Axial IS_{50}	0	estimate
UCS (MPa)	0	

UCS Rating **5.0**

UNIT RATING (UR) 25.0

STRATA ENGINEERING (Australia) Pty Ltd
Consulting and Research Engineering

A.B.N. 26 074 096 263

FACSIMILE

TO:	**Brian Wesley**
COMPANY:	**Ashton Coal**
FAX No:	**02 6576 1122**
DATE:	**9th September 2005**
PAGES:	**5**
FROM:	**David Hill**

Report No. 04-001-ASH-2A

Brian,

Re: Highwall Stability and Portal Support

This brief report constitutes an addendum to the previous **Strata Engineering Report No. 04-001-ASH-2: Ground Stabilisation and Support Requirements for the Portals in the Arties Pit, Ashton Underground Mine** (August 2005).

The original report addressed:

- the proposed configuration of the highwall above the portals,
- support measures for the portals and
- support measures for the initial entries.

This addendum report has been prepared at the request of Mr Brian Wesley, Mine Manager, Ashton Underground Mine, largely to address issues potentially arising from the variances between the originally proposed highwall configuration and the as-built slope. The originally suggested concept is illustrated in **Figure 1**, whereas a representative surveyed profile of the actual slope is shown in **Figure 2**. The main variances are:

i) The as-built slope has an average angle of 61° from the horizontal, versus the envisaged 75° for the lower slope and 60° for the upper slope.

ii) The as-built slope is 17 to 21m high, versus an envisaged 13m lower slope and 6-7m high upper slope, separated by a 7m catch bench.

The as-built slope was inspected briefly by D. Hill of Strata Engineering on the 2nd of August 2005. No signs of significant instability were noted at that time. It is understood that grading of weathered material in the top 2 to 3m of the slope at an angle of ~26° (2:1) will reduce the crest height accordingly.

The following comments are made with regard to the impact of these variances on highwall stability and portal support requirements:

i) The reduced lower slope angle has generally positive implications, in that it would tend to reduce both the likelihood and size of any wedge failures, given that the slope angle is now significantly lower than the typical angle of dip (ie sub-vertical) of the main joint sets.

ii) However, the reduction in slope angle and the presence of weathered material in the upper portion of the slope could lead to greater erosion in the crest area. This is not likely to involve the loosening of large blocks of material, provided that measures are adopted to control the development of overhangs in successive hard / soft layers. The major disadvantages are likely to be increased ongoing maintenance costs associated with the cleaning of drains, as well as a gradual deterioration in the appearance of the highwall face. Consideration should be given to sealing (shotcreting) the top section of the face above the portals, to reduce the rate of deterioration. Some periodic remedial treatment is also likely to be required.

iii) The revised slope configuration changes the materials in the anchorage horizon for the mesh bolts close to the crest. According to the closest exploration borehole, WML 56, the first 3.5m is clay, followed by interbedded mudstone, siltstone and sandstone units ranging in material strength from weak to strong. Removal due to grading of the first 3m of clay at the crest of the slope would leave the anchorage of the planned 4m long bolts in mixed, but predominantly mudstone and sandstone strata. Reference to the log and rock strength data from borehole WML 56 suggests that this is likely to result in adequate bolt bond strengths, assuming that a conventional "soil anchors" approach is adopted (ie 24mm rebar in ~90mm diameter boreholes with a cementitious grout). A detailed examination of the highwall should be made prior to meshing, to confirm that the expected profile is consistent (and that there are no critical areas of deeper clay cover, for example). This is consistent with the recommendation made in **Section 2.4** of the original report. The anchors should be installed a minimum horizontal distance of 3m from the crest.

iv) The seam rises SE towards the planned return / fan portal site, with the result that the thickness of unweathered strata above the portals is a minimum at that location (ie a total slope height of 17-18m, with 7-8m of unweathered strata, some 1-2m less than that regarded as typical in the original report). The depth and thickness of weathered strata at both the men and materials and the conveyor portals is generally consistent with that originally envisaged (ie 19-21m depth to the roof of the Pikes Gully Seam). However, the roof support provisions previously proposed for the entries (ie in **Section 6.3** of the original report) in part anticipate and reflect this reduction in cover depth at the fan entry, specifically through the application of shotcrete. The entry roof support provisions are therefore considered to remain adequate, given the implementation of a formalised strata management process (including initial entry monitoring).

Finally, consideration is being given by the mine to using tensioned cables as opposed to 40mm rebar for forepoling immediately above the planned entries. It is envisaged that 55t capacity (UTS) cables would be point anchored with resin, tensioned to a jack load of ~25t and then post-grouted using a high strength grout, such as thixotropic Stratabinder HS, partly with the aim of filling and providing consolidation (ie frictional restraint) across potential pre-existing fractures and dilated joints. This alternative approach is considered acceptable and would probably offer some advantages, including enhanced stiffness and consolidation of the rock mass. The author has previously been involved in a similar portal roof grouting exercise (involving PUR on that occasion), which is considered to have contributed materially to enhanced lip conditions (**Hill, 2000**). However, the application of cables should be subject to a risk assessment and the development of a safe operating / working procedure.

Concluding Remark

It should be apparent from the comments herein that the main outcome remains the need for the detailed inspection and mapping of the highwall above the portals, as part of the ground stabilisation and portal support process. This is consistent with the recommendations of the original report.

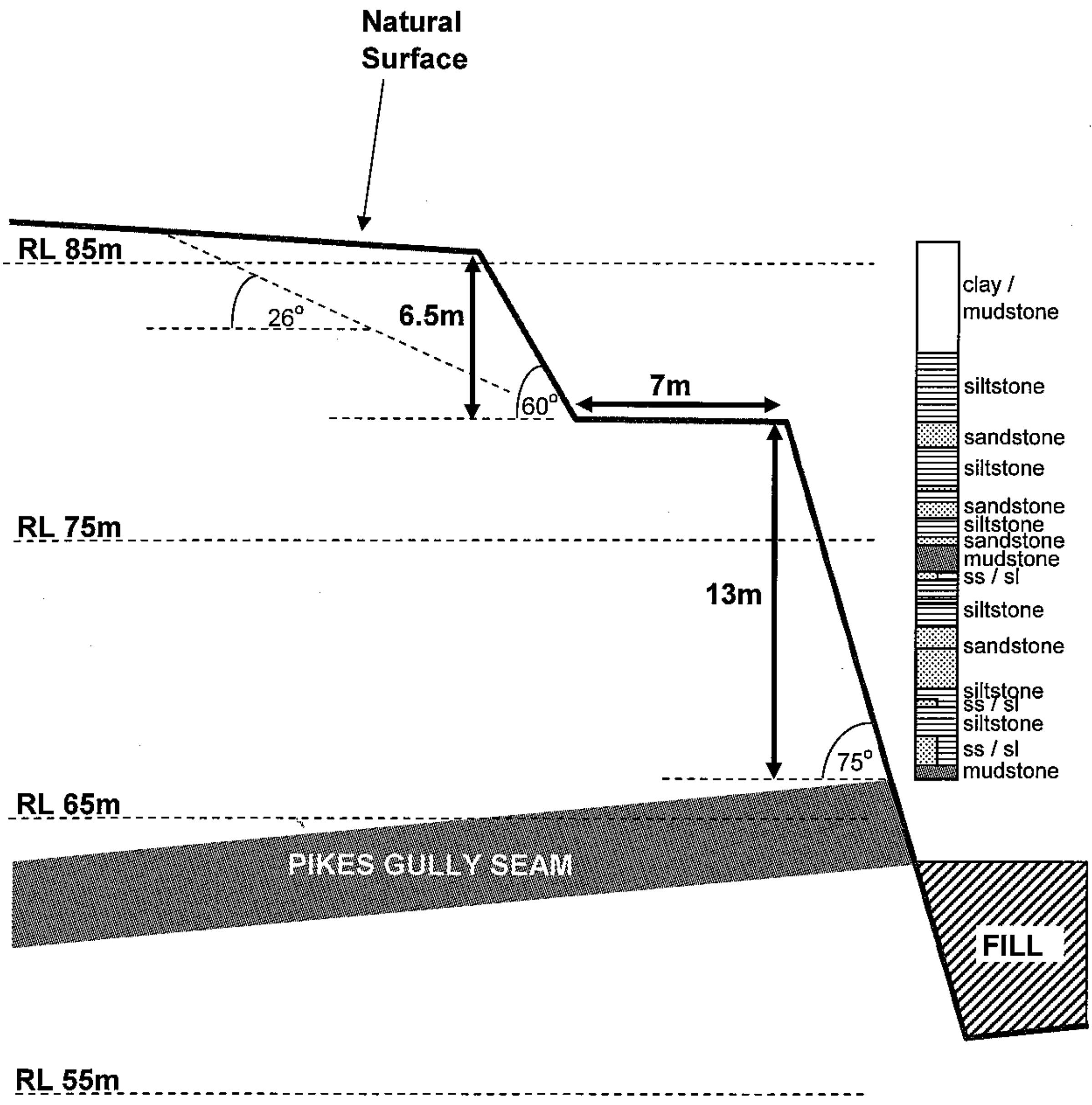
Kind Regards,

STRATA ENGINEERING (Australia) Pty Ltd

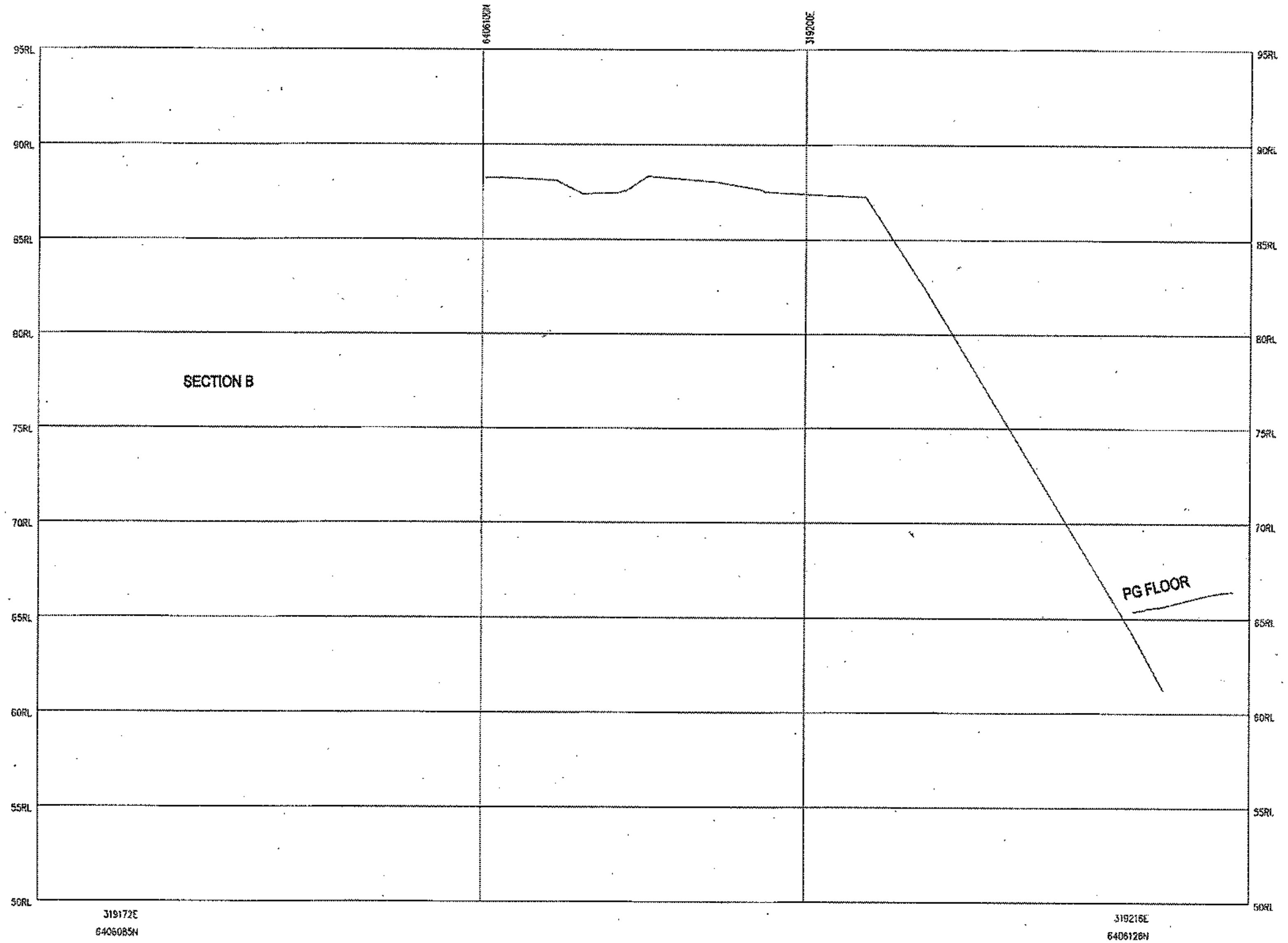
David Hill
Principal

Reference

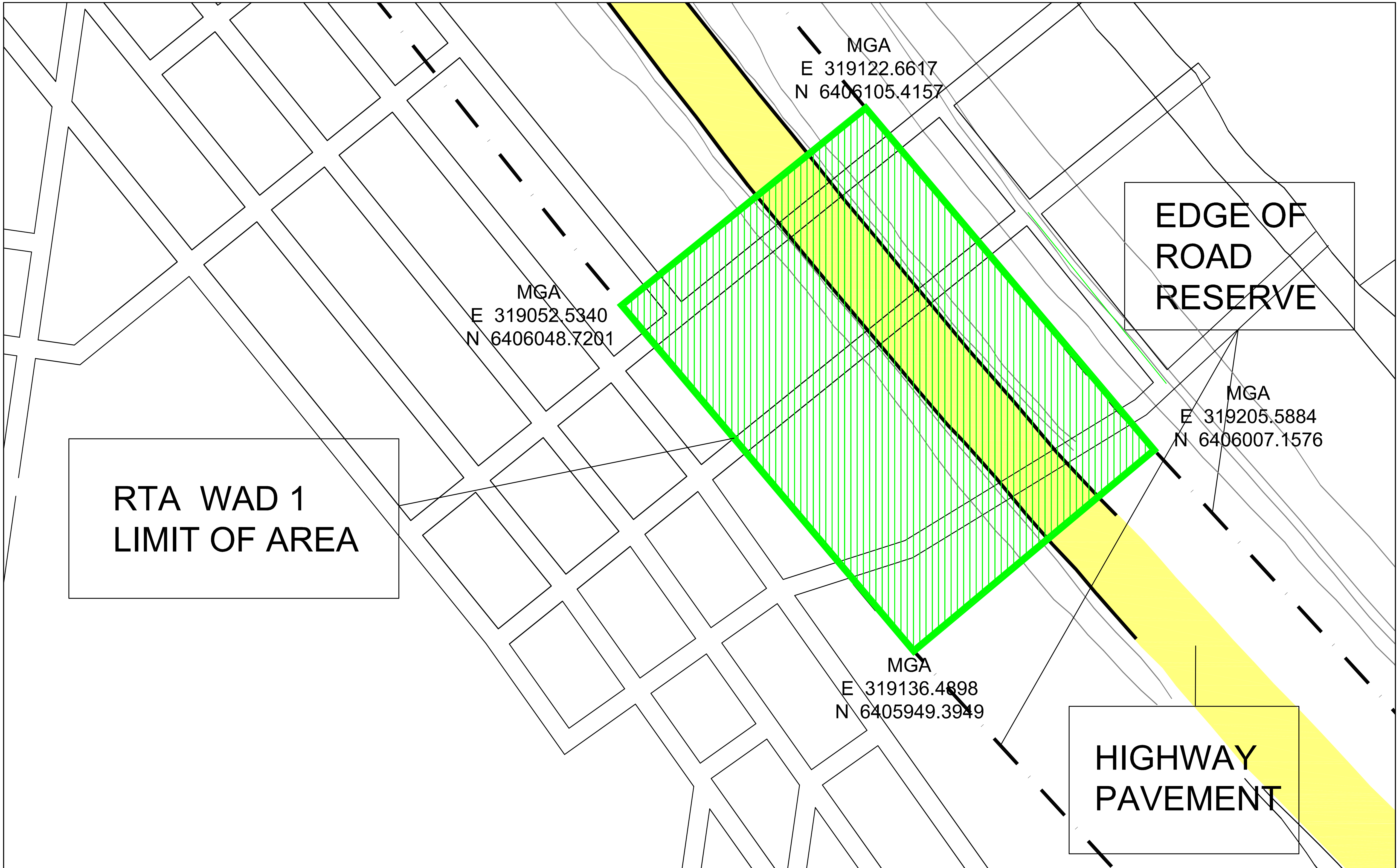
Hill, D.J. (2000). **Underground Monitoring of Roadway Roof Behaviour in Relation to the Use of Highwall Mining Techniques for Initial Punch Mine Entry Development**. Report for ACARP Project C9007.



ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-2A	**(Australia) Pty Ltd**	
DATE:	08.09.05	TITLE:	Initially Suggested Slope Configuration for the Portal Area		FIGURE
SCALE:	NTS				1



ENGINEER:	D. Hill	CLIENT:	Ashton Mine	**STRATA ENGINEERING**	
DRAWN:	D. Hill		R04-001-ASH-2A	**(Australia) Pty Ltd**	
DATE:	08.09.05	TITLE:	Section Through Actual Arties Pit Highwall at		FIGURE
SCALE:	NTS		Proposed Location of Conveyor Portal (Section B)		2



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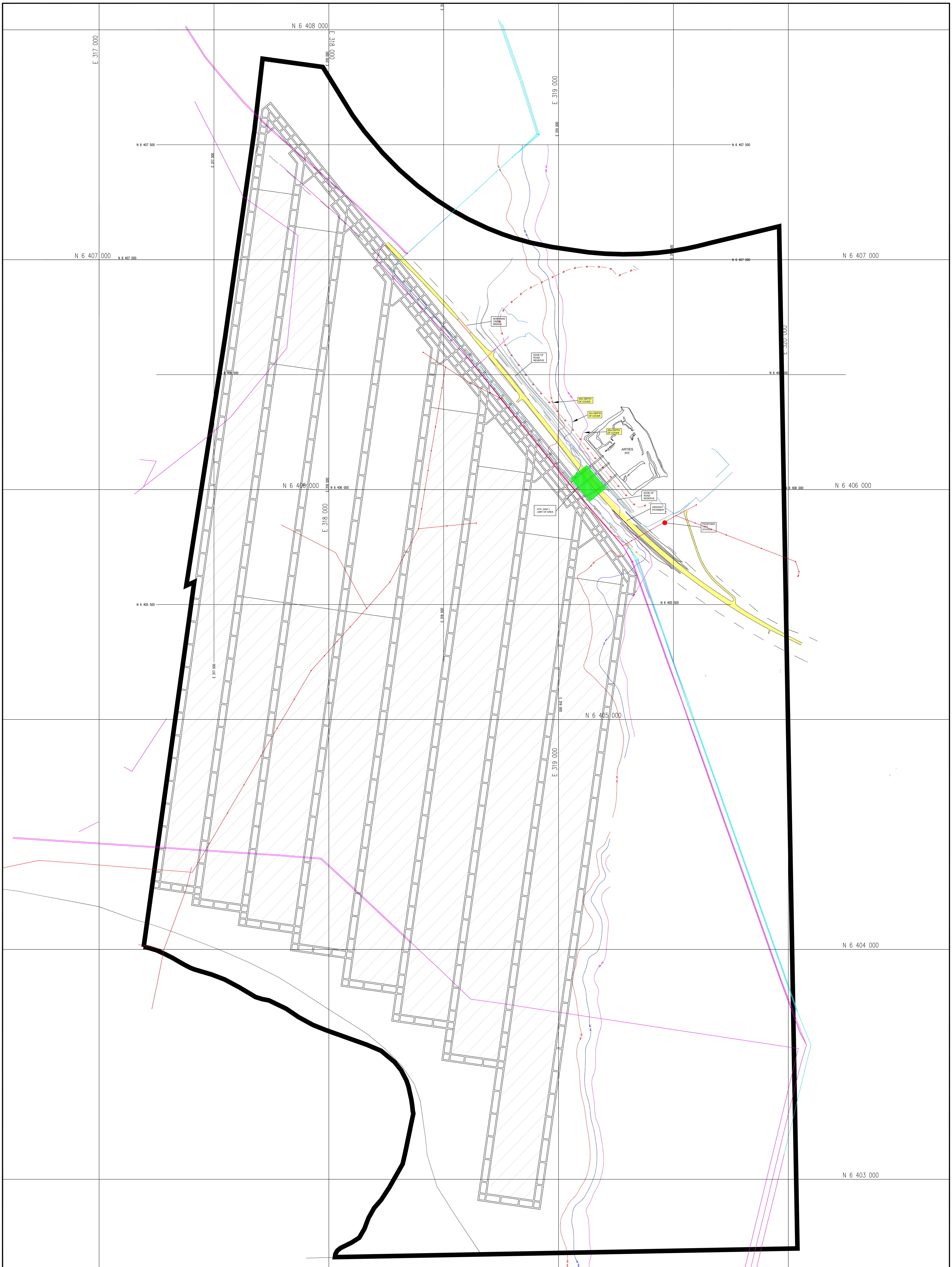
ASHTON UNDERGROUND
RTA WAD 1- LIMIT OF AREA
U/G Roadways beneath New England Highway

Date	Scale:	Drawn	Checked	Approved	Drawing No.	Revision No.	Sheet Size
16/1/2006	1:1000	JG	Checked	DB	A–3001	A	A3



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ASHTON UNDERGROUND MINE
RTA WAD
EXTENT OF UNDERGROUND WORKINGS

Date	Scale:	Drawn	Checked	Approved	Sheet Size
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Drawing No. A–3003

Revision No. A