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Ismet Canbulat
Anglo American Metallurgical Coal

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ROADWAY ROOF SUPPORT DESIGN IN CRITICAL AREAS AT ANGLO AMERICAN METALLURGICAL COAL'S UNDERGROUND OPERATIONS

Ismet Canbulat[1]

ABSTRACT: In order to ensure the stability of roadways Anglo American Metallurgical Coal (AAMC) has developed an advanced roof support design methodology that integrates analytical, numerical and empirical modelling. This methodology currently is based on a deterministic approach (a single factor of safety against failure is calculated). However, an improved methodology, based on stochastic modelling technique, has also been developed and currently being evaluated. The main advantage of this methodology is that as the design is based on probability distributions of input parameters, the outcome is based on a distribution of factors of safety rather than a single factor of safety. Evaluation of factors of safety may also be used to determine the likelihood of failure which in turn may be utilised to determine and evaluate the associated risks quantitatively in decision making process. This methodology has been evaluated at Grasstree and Moranbah North Coal Mines in the designs of roof support in various critical areas and has been proven to be successful and a better way of determining the roof support requirements. A demonstration of application of this methodology from Moranbah North Mine, where the "world's highest rated longwall" has recently been installed, is presented in this paper.

INTRODUCTION

Anglo American Metallurgical Coal Australia (AAMC) operates three longwall (LW) mines located in Central Queensland. There is an increasing emphasis on the reliability at these operations as the longwalls are getting deeper and facing more geologically challenging conditions. In addition, the Anglo American Vision is to achieve "Zero Harm" through the effective management of safety at all businesses and operations. In order to accomplish this vision, AAMC has developed a pro-active ground control management system for a safe and efficient production of underground reserves.

Pro-active ground control management involves an understanding of the impacts of the geotechnical environment on likely ground behaviour and consists of approximately 15 major elements. One of the most important elements of this pro-active ground control management is to utilise a roof support design methodology that considers different failure mechanisms and also takes into account all important elements. The design metho dology becomes even more important when the area in question is a *critical* area, which is defined as a high risk roadway where any failure may cause increased levels of safety and financial risks to underground workforce and operations.

One of the recent critical area support designs was at Moranbah North Coal Mine (MNC), where the 1 750 t longwall face was recently commissioned. Because of the size of this longwall, one of the widest longwall installation roads in Australia was developed and widened. Following a stand-up time of approximately two months, the longwall was successfully installed and no excessive roof deformations were encountered.

AAMC ROOF SUPPORT DESIGN METHODOLOGY

The AAMC design methodology is defined by the Geotechnical Systems and Standards of the Operations Management System, which provides a set of minimum standards for the primary and secondary roof and rib support assessment, analysis, design and presentation process at AAMC underground operations.

The aim of this support design methodology is to ensure the stability of roadways at AAMC underground operations.

[1] *Anglo American Metallurgical Coal, Brisbane, 4000*

There is no single universally accepted roof and rib support design methodology. In general, the design methodologies for bolt selection and design can be classified into the following six categories:

- Analytical models,
- Empirical models (i.e., statistical analysis of previous data/experience),
- Numerical modelling,
- Field testing and monitoring,
- Geotechnical classification (mainly CMRR etc), and
- Physical modelling (laboratory testing).

These methodologies require extensive input parameters and assumptions with regard to support and rock properties.

AAMC's strategy is to use a "combined support design methodology" for critical areas. This methodology considers all methods listed above, with the exception of physical modelling to ensure that the design is sound and acceptable.

In AAMC's roof support design methodology, four analytical models, namely buckling, shear, tendon in suspension and bond in suspension failures, are considered and, where possible, empirical modelling and geotechnical classification techniques are also used to back analyse the designs and/or to derive the input parameters (e.g., horizontal stress magnitudes).

As the details of these analytical models can be found in numerous previously published publications (Frith, 2000; Colwell, Frith and Guy, 2008; Canbulat and van der Merwe, 2009), it is not intended to present the fundamentals of these models in this paper; therefore, only a short summary of these mechanism are given below for the completeness of the paper:

Suspension Failure

The suspension mechanism is the most easily understood roof bolting mechanism. The design of roof bolt systems based on the suspension principle has to satisfy the following requirements (Canbulat and van der Merwe, 2009):

- The strengths of the roof bolts and/or long tendons have to be greater than the relative weight of the loose roof layer that has to be carried.
- The anchorage forces of the roof bolts have to be greater than the weight of the loose roof layer.

Shear Failure

This mechanism assumes that the coal mine roof contains a series of bedding planes/laminations and the shear strength of these bedding planes can be calculated using the well-known Coulomb theory with the inclusion of pre-tension and the shear strength of the roof bolts and/or long tendons.

In calculation of the factors of safety, the important consideration in this mechanism is the so-called *height of softening* to determine the shear stress generated by the surcharge load within the bedding planes. It is assumed in this mechanism that as the height of softening increases parabolically (even above the bolted horizon), the resultant shear stress generated within the bedding planes also increases (Canbulat and van der Merwe, 2009).

Buckling Failure

This mechanism has been extensively discussed by Frith (2000) and Colwell, Frith and Reed (2008). A factor of safety concept is also utilised in this mechanism, this being a measure of the load applied to that structure in comparison to its ability to accommodate that load without undergoing yield or failure.

In this mechanism it is assumed that the applied load acts horizontally across the roof and is a product of the *in situ* horizontal stress and concentration thereof as a result of the mining process. The load bearing ability of the roof strata and the load bearing ability of roof support are calculated using the buckling theory and mechanical advantage concept (Frith, 2000; Colwell, Frith and Reed, 2008).

To this end, Figure 1 provides a diagrammatical representation of AAMC's combined support design methodology.

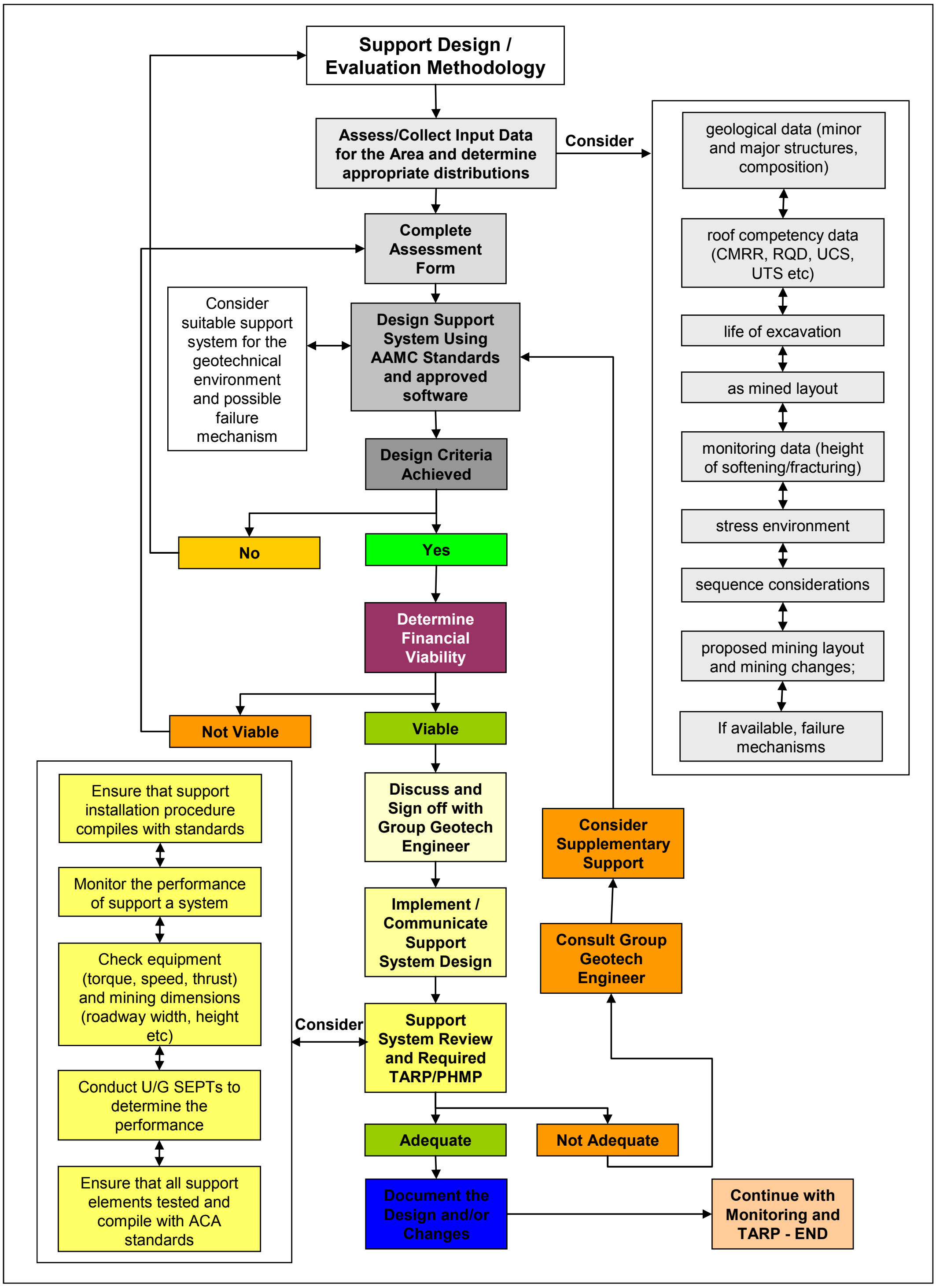

Figure 1 – AAMC's roof support design methodology

This methodology follows the below steps:

- Collection and assessment of geological, mining, and monitoring data, including:
    - o Development of probability distributions.

    - o Evaluation of the stress environment using local knowledge, measurements and numerical modelling.

- Prepare assessment form, following site inspections (where possible) and/or available information.
- Design the support system. In the design consider the geotechnical environment, a suitable support system and the four failure mechanisms.
- Check the design against the design criteria.
- Determine the financial viability of the design. Consider that the same factors of safety can be achieved using a combination of different support systems.
- Sign it off with the Principal Geotechnical Engineer and/or external experts.
- Implement the design and communicate it to mining personnel.
- Inspection and monitoring of the installation and performance of the installed support during mining operations for a comparison against the initial design performance. This will require:

UNCERTAINTIES IN ROOF SUPPORT DESIGN

Many investigations in Australia and elsewhere confirm that rock mass properties exhibit a high degrees of uncertainty. The performance of a support system is affected by these uncertainties and ideally they should be taken into account. In traditional deterministic (calculation of a single safety factor) roof bolt design methodologies, the input parameters are represented using single values. These values are described typically either as "best guess" or "worse case" values. Although in many deterministic roof support design cases, a series of sensitivity analyses are also conducted over one or more parameters, these analyses provide only some insight into the underlying mechanisms and usually cannot take into account the variations that exist in almost all input parameters in support design.

The uncertainties in roof support design can be divided into three distinct areas as shown in Figure 2. These areas are:

- *Given* conditions, which contain all geological and stress related design parameters, such as rock strength, unit thicknesses, friction, stress magnitude and direction. These conditions cannot be controlled or changed.
- *Responses* to the given conditions, which contain the support selection and mining selection, such as length, strength and effectiveness of the support, mining method, sequence and dimensions.
- The *resultant* conditions, which contain the height of softening, displacement and stress magnitudes that are resulted due to one or combination of *given* and/or *response* conditions.

As the given conditions cannot be changed, the aim of a roof support design should therefore be to improve the responses so that the resultant conditions are controlled and the roof design is successful in preventing the unexpected falls of ground.

It has been widely accepted and reported in coal mines that all the above parameters vary within a short distance in a panel. The roof stability is strongly dependent on these varying properties of the roof support system. This variation can be taken into account using either stochastic modelling or sensitivity analyses in deterministic models. However, as there are many parameters that are considered in a design, sensitivity analyses in deterministic models usually fail to demonstrate the impact of this variation. In stochastic modelling, which is a technique of presenting data or predicting outcomes that takes into account a certain degree of randomness, or unpredictability, these variations of the input parameters are included. It is therefore possible to quantitatively represent the uncertainties. This method is usually used in probabilistic design approaches in which it is acknowledged that realistically there is always a finite chance of failure, although it can be very small. This approach is considered to be a step forward in the design of coal mine roof support systems and therefore is utilised in AAMC's roof support design methodology to take into account the uncertainties that exist.

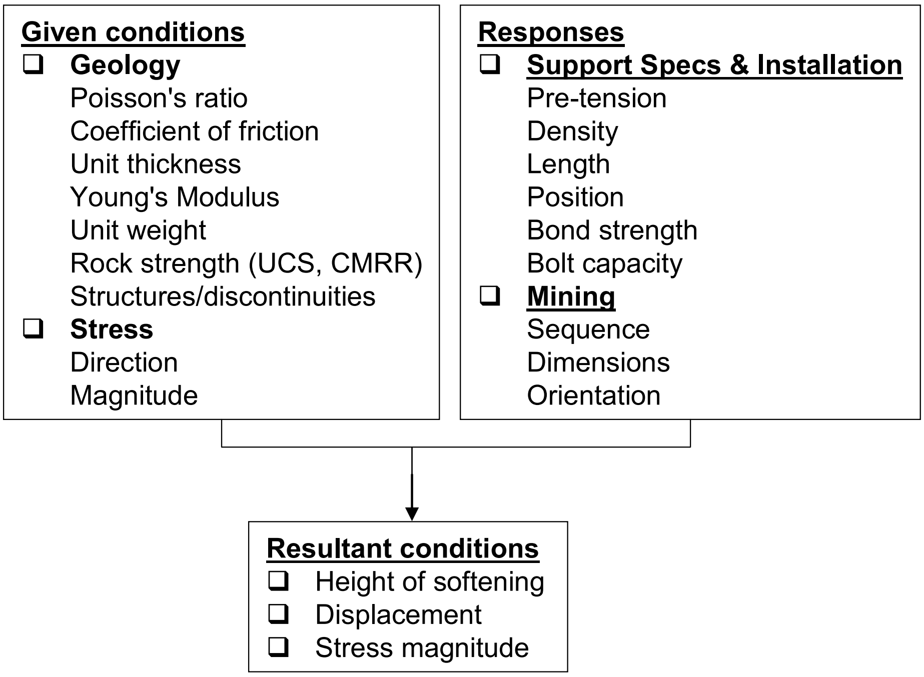

Figure 2 – Elements of uncertainties in roof support design

Stochastic modelling allows the input parameters to be taken as probability distributions rather than single values using the well known Monte Carlo simulation method. In this method, the distribution functions of each stochastic variable must be estimated. From each distribution, a parameter value is sampled randomly and the value of the performance function calculated for each set of random samples. If this is repeated a large number of times, a distribution of the performance function is obtained. Monte Carlo simulation is thus a procedure in which a deterministic problem is solved a large number of times to build a statistical distribution. It is simple and can be applied to almost any problem and there is practically no restriction to the type of distribution for the input variables.

A fundamental aspect of the Monte Carlo method is the process of explicitly representing the variations by specifying inputs as probability distributions. By describing the process as a probability distribution, which has its origins in experimental/measurement *continuous* data, an outcome can be sampled from the probability distributions, simulating the actual physical process/measurement.

This process requires a collection of actual measurements and determining the best fits to the data using the goodness of fit tests (GOF). GOF tests measure the compatibility of a random sample with a theoretical probability distribution function. Three most common GOF tests are (EasyFit, 2008):

- Kolmogorov-Smirnov
- Anderson-Darling
- Chi-Squared

The details of the probability distributions, GOF tests and random selection of design parameters can be found in readily available statistical modelling and reliability engineering publications (e.g., Harr, 1987 and Wolstenholme, 1999).

The stochastic modelling technique has been widely used in Civil Engineering, mainly in slope stability analyses. The development has not yet reached this point in the field of underground roof support design. Possible reasons for this were (i) defining a model which describes both the strength and the load acting on rock and (ii) extensive input parameters required in the analyses. Considering that AAMC underground operations collect a vast amount of geotechnical and geological information and the roof behaviour is well understood, it may be possible to apply the stochastic modelling in the design of roof support systems.

EVALUATION OF AAMC'S ADVANCED ROOF SUPPORT DESIGN METHODOLOGY

The above summarised roof support design methodology has been applied to a well-defined case study at Moranbah North Mine (Figure 3). The 2.0 m wide, 63 t shields rated at 1 750 t installed in August 2009 required a large installation road for LW 108. The install road width varied from 9.5 to 10.7 m increasing up to 11.5 m wide towards the tailgate (TG) end and up to 10.7 m wide towards the maingate (MG) end (Figure 4).

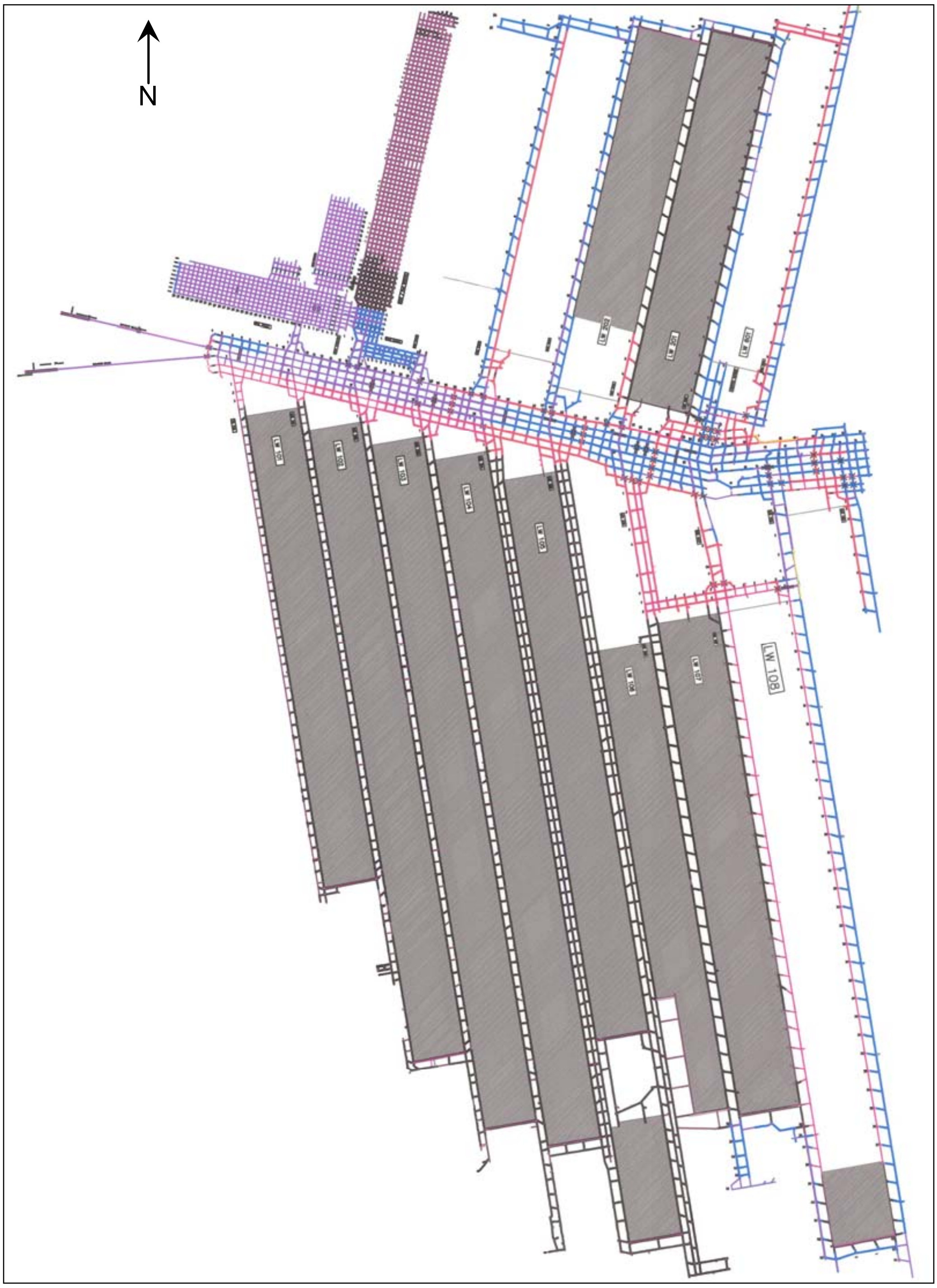

Figure 3 – MNC mine plan

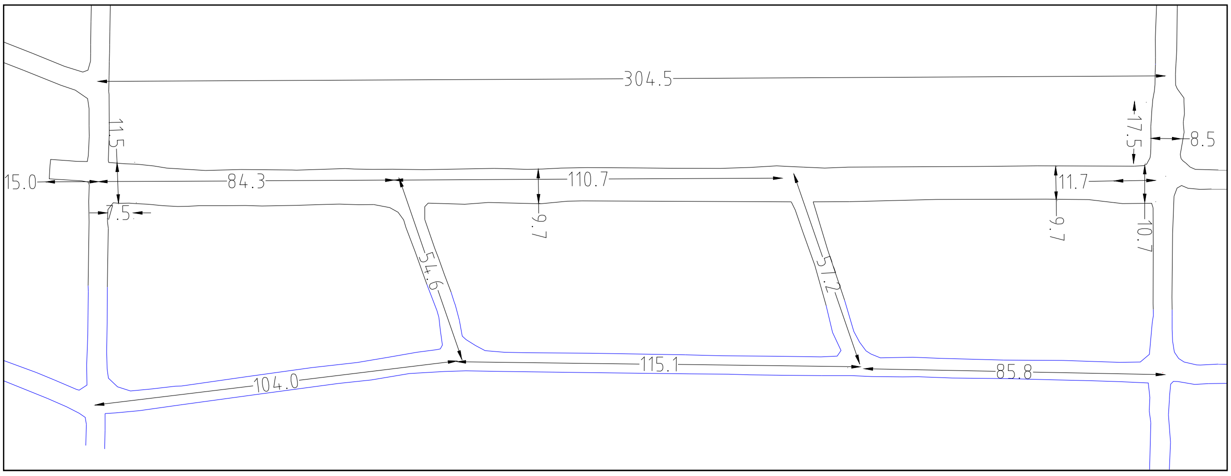

Figure 4 – LW 108 install road as mined dimensions (in meters)

In the past, the longwall install roads at MNC have been stable with slow long-term creep depending on stand up time before longwall installation. Past experience also indicated that the following factors are significant in an install road support design at MNC:

- Stress direction and magnitude/face road orientation;
- Development displacements and height of softening;
- Required roadway dimensions;
- Roof stratigraphy;
- Structures;
- Stand up time before longwall installation;
- Primary support density;
- Sequence of widening/floor brushing etc;
- Primary and secondary support positions;
- Monitoring regime;

In general, a complete install road roof support design should consider the following areas:

- Install road
- Wide areas in the TG and the MG sides of the install road (i.e. shearer stable, MG shields etc)
- Three-way intersections within the install road (i.e. cut-throughs (C/T) between the bleeder road and the install road)
- MG and TG intersections (including stubs), and
- Wide areas due to offline cutting or for another reason.

In the case of a back bleeder road (as in this case), the stability of the pillar located between the install road and the bleeder road should also be considered, as an under designed pillar may result in increased levels of roof deformation in the install road.

In addition, a Trigger Action Response Plan (TARP) should be developed for each install road to ensure a timely and quality response to changing conditions.

INPUT PARAMETERS USED IN THE ROOF SUPPORT DESIGN FOR LW 108 INSTALL ROAD

In order to conduct AAMC's roof support design methodology, the following input parameters are required.

- Stress environment (principal, intermediate and minor stress directions)
- k-Ratio (horizontal stress to vertical stress ratio) to be used in numerical modelling
- Depth of cover
- Roadway width
- Height of fracturing/softening into the roof
- UCS of rock at long tendon anchorage horizon
- Elastic modulus at long tendon anchorage horizon
- Fracture spacing at long tendon anchorage horizon
- Development displacements
- Roof bolt capacity
- Long tendon capacity
- Roof bolts' pre-tension
- Long tendons' pre- tension.
- Coefficient of friction between the layers (assumed to be negligible in this case)
- Unit weight of immediate roof and overburden
- Bond strength of long tendons

It is evident from the above list that there are many input parameters that need to be taken into account in the design and almost all of these parameters inherently vary. In addition, experience has shown that the support installation practice as well as the performance of support consumables can also vary significantly and ideally, their variation should also be taken into account in the design. It is therefore considered that it is nearly impossible to conduct a sensitivity analysis on all of these parameters using a deterministic design approach.

Immediate Roof

MNC extracts the Goonyella Middle (GM) Seam within the Bowen Basin Coalfield, Central Queensland. The seam thickness varies from 5.0 to 6.5 m; the development thickness in the gateroads, main headings and the install roads is approximately 3.5 m. The cover depth of current MNC workings varies from 100 to 300 m; the cover depth associated with LW 108 install road is approximately 300 m.

The roof of MNC is generally characterised by a weak immediate roof overlain by moderately bedded, stronger siltstone and sandstone units. Figure 5 shows the general stratigraphic column in and around LW 108 install road.

An analysis of 58 boreholes over MNC current and past workings indicates a relatively consistent Coal Mine Roof Rating (CMRR) of 40. Based on the study conducted by Molinda and Mark (1994), this roof can be classified as "moderate to weak". A CMRR value of approximately 40 is also a reasonable estimate for the LW 108 install road.

As part of routine geotechnical investigations, MNC geotechnical / geology department conduct numerous laboratory tests on roof and floor samples. These indicate that the UCS of the sandstone unit at long tendon anchorage horizon varies from 15 to 50 MPa with an average of approximately 30 MPa (Figure 6). The fracture spacing of this sandstone unit has an average of 374 mm (Figure 7) with a variation of 43 mm to 1 200 mm. Figure 8 shows the available elastic modulus test results in this database with respect to the target GM Seam.

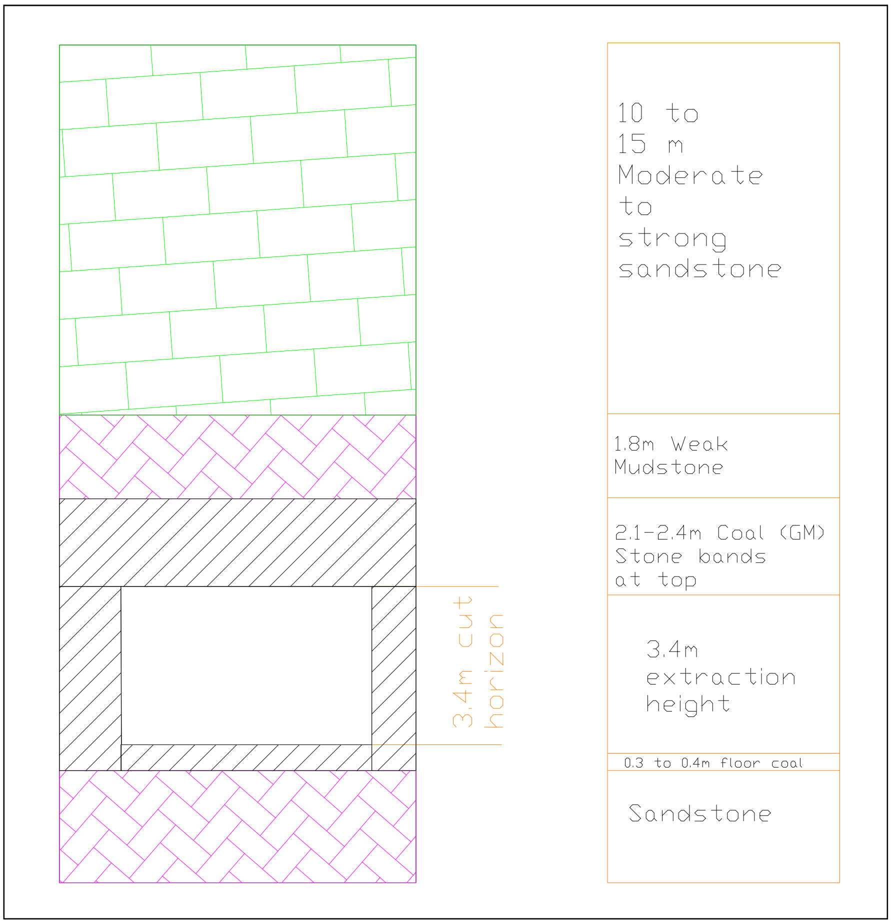

Figure 5 – Typical stratigraphic succession and section of roadway development

With regard to the unit weight of the immediate roof, the laboratory test results indicate a unit weight of 0.018 to 0.026 MN/m^3 for the immediate roof horizon (i.e., within 6.0 m top of the seam). In addition, it is assumed in the calculations that the overburden will have a constant unit weight of 0.025 MN/m^3.

Height of Softening

Experience gained in previous longwall install roads at MNC indicates that the height of roof softening (the height into the roof where the deformation/separation is minimal) may increase to 4.5 m (on average) into the roof. Table 1 summarises the sonic probe extensometer measurements obtained for LWs 105, 107, 201 and 202 where detailed roof and rib monitoring programmes were carried out. It is evident from this table that in all previous cases the height of softening was equal or greater than the primary roof length of 1.8 m.

Stress Environment

The information regarding the stress environment is required in numerical modelling as well as in determining the Stress Concentration Factor (Gale and Matthews, 1992) to estimate the anticipated horizontal stress levels in the roof in and around the install road. The aim of numerical modelling in this case is to verify that the empirically calculated stress levels in analytical models are in accordance with numerical modelling.

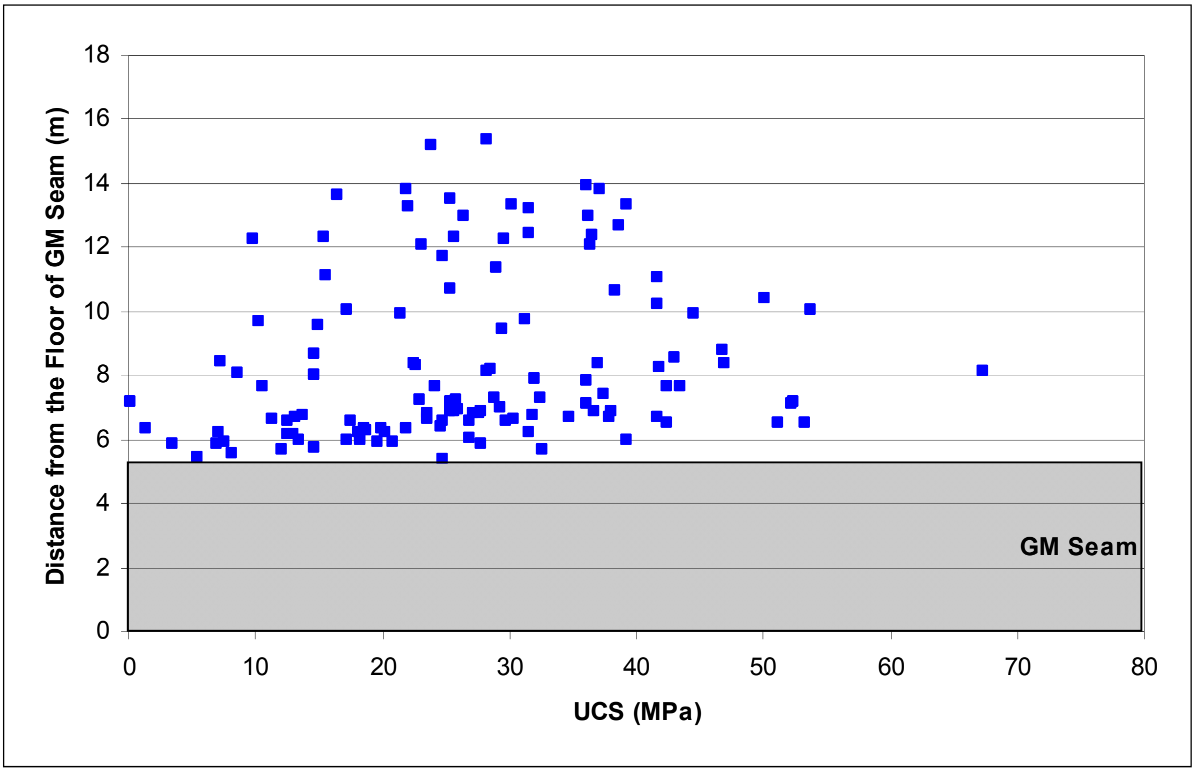

Figure 6 – Distribution of UCS from the GM Seam floor

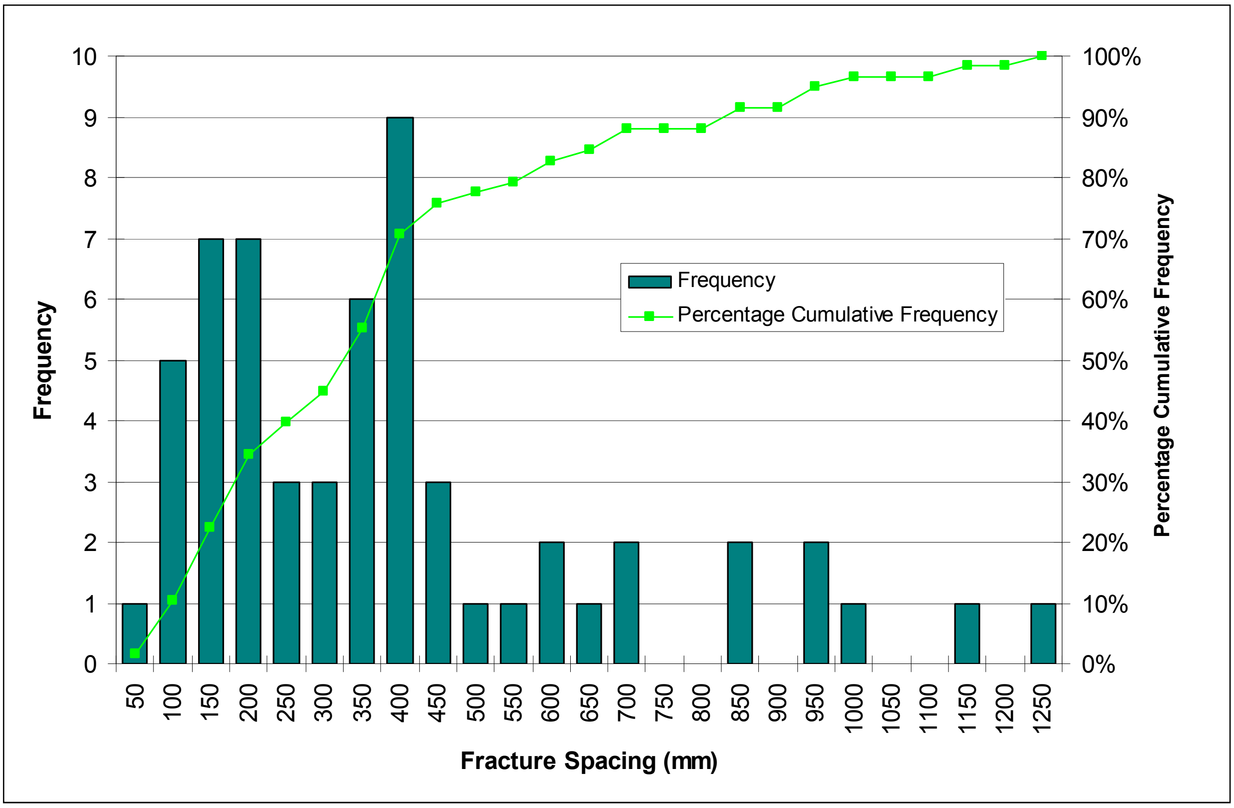

Figure 7 – Fracturing spacing distribution at long tendon anchorage horizon

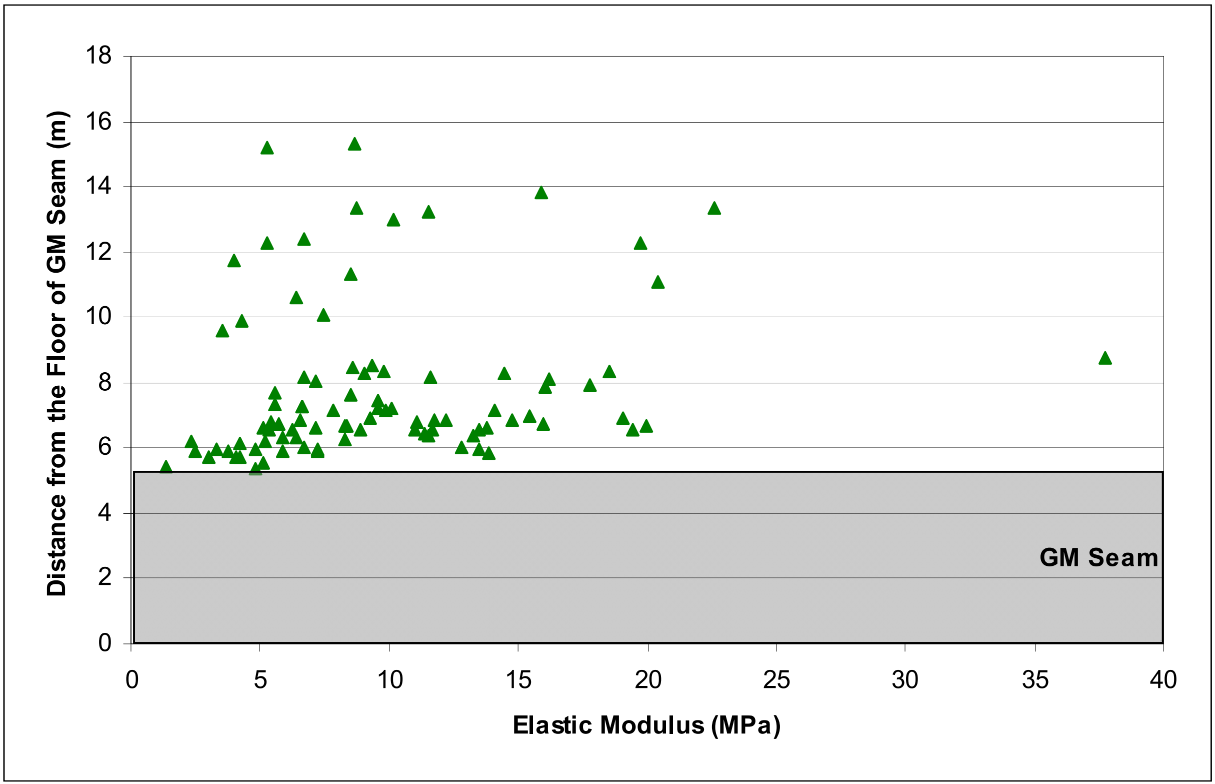

Figure 8 – Distribution of roof elastic modulus data with respect to the GM Seam

Table 1– Summary of height of roof softenings measurements obtained in previous longwall install roads at MNC

Hole ID	Initial Displ. (mm)	Final Displ. (mm)	Height of Softening (m)
LW 105			
105.80	5.4	20.0	3.9
105.120	6.3	28.0	5.2
105.160	0.0	19.0	4.9
105.240	5.9	22.0	4.3
105.40	17.5	97.0	5.5
105.200	5.0	26.0	5.0
105.180	5.4	40.0	5.0
105.60	9.0	18.0	1.8
105.265	3.0	9.0	4.5
105.255	0.0	17.2	4.5
LW 107			
107.230	9.1	12.3	3.4
107.70	7.5	15.1	3.0
107.270	0.0	39.3	4.0
107.190	24.2	33.2	5.0
107.Bld	2.0	12.1	3.2
107.293	0.6	64.5	7.0
107.310	0.6	41.9	4.5

Hole ID	Initial Displ. (mm)	Final Displ. (mm)	Height of Softening (m)
LW 201			
210.TG	5.9	18.7	5.0
201.305	10.7	31.6	6.4
201.5	9.9	20.0	5.5
201.187	3.7	11.7	4.9
201.220	1.3	3.8	4.5
201.152	0.0	13.4	3.5
201.80	1.2	31.4	4.5
201.100	12.6	34.2	4.5
201.118	6.8	23.9	4.4
LW 202			
202.22	11.5	29.6	4.1
202.100	0.9	21.3	5.3

Gale and Matthews (1992) linked the stress concentration factor (SCF) with the angle between the gateroad drivage direction and that of the major horizontal stress as shown in Figure 9. This model is also utilised to calculate the anticipated horizontal stress levels in the buckling failure model.

In order to determine the stress environment, a series of stress measurements were conducted at MNC. Figure 10 shows the details of the compiled MNC's stress map. This figure indicates a strong NNE-SSW stress direction across the MNC, which is consistent with regional Bowen Basin experience. The stress measurement data also indicates a stress orientation of approximately 10^o East of North for MNC.

A summary of MNC in situ stress measurement data is presented in Table 2. Using this data it is possible to calculate the tectonic stress and the major horizontal stress using the methodology given by Nemcik, Gale and Mills (2005) to utilise in buckling failure model.

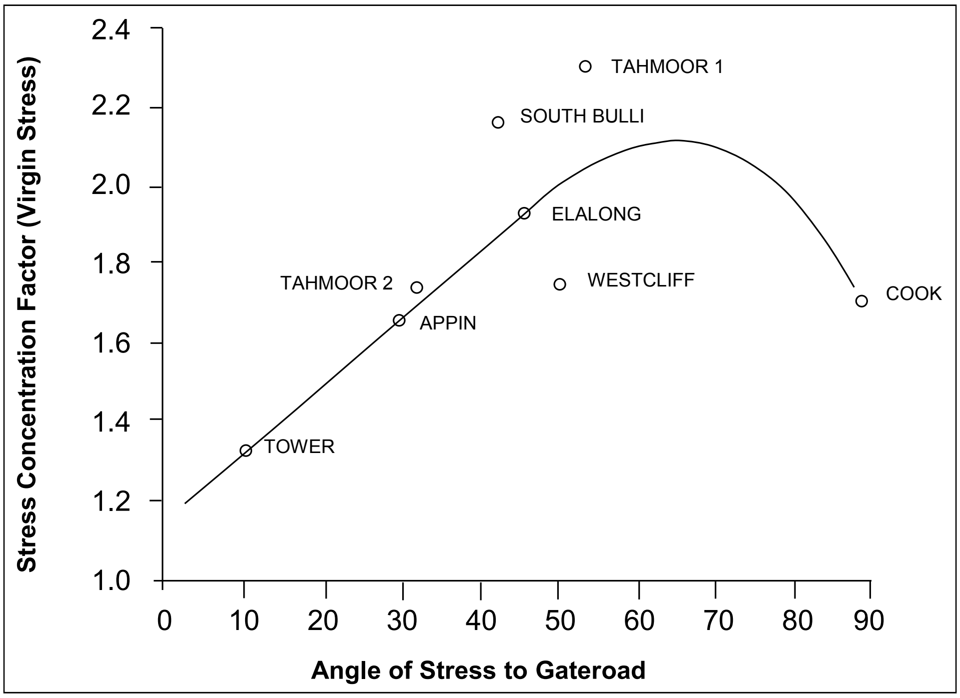

Figure 9 – Relationship between SCF and angle of gateroad to stress direction (after Gale and Matthews, 1992)

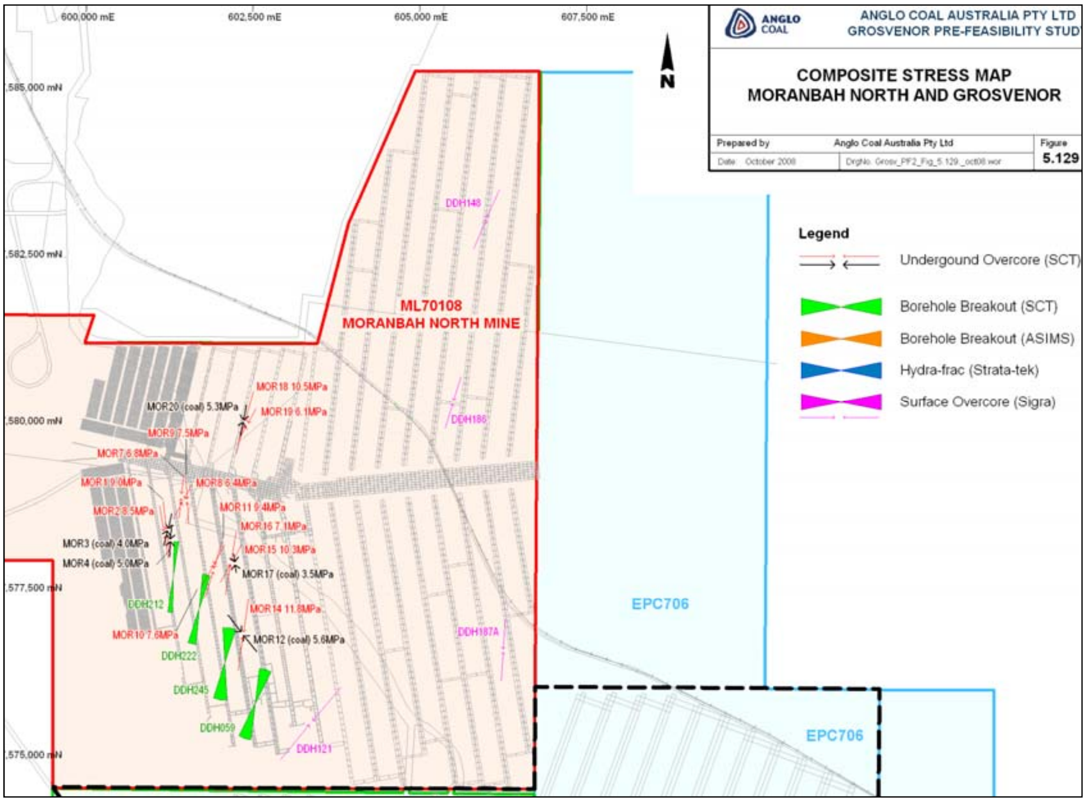

Figure 10 – Moranbah North Mine stress map

Roof Bolt and Long Tendon Bond Strengths

A series of short encapsulated pull tests were conducted at MNC as part of the quality control procedures. The long tendon tests were conducted using 4 m long cables with 300 mm encapsulation. The roof bolt short encapsulated tests were using the standard roof bolt length of 1.8 m with 300 mm encapsulation. The results indicated that:

- long tendon pull out resistance varies from 0.3 to 1.43 kN/mm (calculated as the maximum load achieved/encapsulation length) with an average of 1.2 kN/mm.
- roof bolt pull out resistance varies from 0.3 to 0.6 kN/mm

Based on these variations and also that an initial analysis indicated that the impact of this variation is insignificant, the pull out resistance of roof bolts and long tendons are entered as single values of 0.3 kN/mm rather than probability distributions. It should however be noted that in areas where the pull out resistance is critical, bond strengths of roof bolts and long tendons should also be entered as probability distributions.

Probability Distributions of Input Parameters

As mentioned above, the roof support design methodology presented herein requires the input parameters as probability distributions rather than single values. In order to determine the representative probability distributions of input parameters, a series of GOF tests were run for each parameter. A summary of the GOF test results is summarised in Table 3.

Note that the results presented in Table 3 are based on a limited number of data points and the limits of the software utilised to conduct the Monte Carlo simulations. Therefore, some of the best fit probability distributions obtained from GOF tests are only marginally better than the others. It is also of note that in some areas, the roadway width was slightly wider than the planned 9.5 m, therefore the probability distribution for roadway width is assumed.

It should also be noted that the roof bolt and long tendon pre-tensions of 50 kN and 150 kN are also assumed in the calculations respectively.

Table 2 – Summary of *in situ* stress measurement data

Hole No 1		Dip (deg)	Bearing (deg)
σ_1	5.1	2	354
σ_2	4.9	86	229
σ_3	3.7	3	84
Depth (m)	145m		
Rock Type	Siltstone		
Position	6.74m above roof		
E (MPa)	4.7		
ν	0.44		

Hole No 7		Dip (deg)	Bearing (deg)
σ_1	7.9	38	343
σ_2	7.1	50	186
σ_3	5.6	11	82
Depth (m)	145m		
Rock Type	Siltstone		
Position	7m above roof		
E (MPa)	8.4		
ν	0.39		

Hole No 2		Dip (deg)	Bearing (deg)
σ_1	7.1	79	270
σ_2	6.6	1	5
σ_3	4.8	11	95
Depth (m)	140m		
Rock Type	Sandstone		
Position	17.23m above roof		
E (MPa)	9.6		
ν	0.41		

Hole No 8		Dip (deg)	Bearing (deg)
σ_1	6.9	72	316
σ_2	6	4	212
σ_3	3.9	17	120
Depth (m)	140m		
Rock Type	Sandstone		
Position	12.34m above roof		
E (MPa)	9		
ν	0.35		

Hole No 3		Dip (deg)	Bearing (deg)
σ_1	4.9	3	356
σ_2	4.3	75	255
σ_3	3.6	15	87
Depth (m)	140m		
Rock Type	Sandstone		
Position	11.67m above roof		
E (MPa)	5.7		
ν	0.37		

Hole No 9		Dip (deg)	Bearing (deg)
σ_1	10.9	6	27
σ_2	7	43	291
σ_3	4.1	47	124
Depth (m)	175m		
Rock Type	Sandstone		
Position	5.55m above roof		
E (MPa)	15.5		
ν	0.23		

Hole No 4		Dip (deg)	Bearing (deg)
σ_1	15.7	5	18
σ_2	11.8	35	284
σ_3	6.9	54	116
Depth (m)	175m		
Rock Type	Sandstone		
Position	5.75m above roof		
E (MPa)	19.6		
ν	0.32		

Hole No 10		Dip (deg)	Bearing (deg)
σ_1	12.9	21	212
σ_2	8.4	65	354
σ_3	5.4	14	116
Depth (m)	185m		
Rock Type	Sandstone		
Position	4.37m above roof		
E (MPa)	22.4		
ν	0.32		

Hole No 5		Dip (deg)	Bearing (deg)
σ_1	14	9	9
σ_2	7.9	24	275
σ_3	6	64	119
Depth (m)	230m		
Rock Type	Sandstone		
Position	5.3m above roof		
E (MPa)	12.1		
ν	0.23		

Hole No 11		Dip (deg)	Bearing (deg)
σ_1	9	1	210
σ_2	4.5	15	300
σ_3	4.2	75	115
Depth (m)	178m		
Rock Type	Siltstone		
Position	5.41m above roof		
E (MPa)	16.3		
ν	0.29		

Hole No 6		Dip (deg)	Bearing (deg)
σ_1	13.2	13	197
σ_2	7.3	4	288
σ_3	5.1	77	35
Depth (m)	178m		
Rock Type	Siltstone		
Position	5.22m above roof		
E (MPa)	12.7		
ν	0.37		

Table 3 – Summary results of GOF tests

Parameter	Representative probability distribution	Scale Parameter	Shape/location Parameter
Elastic modulus	Gamma	2.56	4.83
Displacement	Lognormal	0.30	2.30
Fracturing spacing	Lognormal	0.76	-1.25
Height of softening	Weibull	4.57	8.19
Poisson's ratio	Lognormal	0.21	-1.10
UCS	Lognormal	0.20	3.44
Roadway width	Normal	0.10	10.0
Unit weight	Weibull	0.024	14.30

SUPPORT DESIGN AND EVALUATION

Numerical Modelling

AAMC has developed mine-wide numerical modelling layouts for all underground operations for detailed geotechnical investigations. An example of MNC's modelling layout is shown in Figure 11. The aim of numerical modelling in this study was to demonstrate that the magnitudes of horizontal stress notching calculated using empirical modelling (i.e., the methodology of Gale and Matthews, 1992) are not significantly different than numerical modelling and also the fact that the surrounding mining may have an impact on the magnitudes of the horizontal stress notching. In order to achieve this comparison, a detailed elastic numerical modelling study was conducted using Map3D.

Map3D is a 3-dimensional (3D) fully integrated stability analysis package based on indirect boundary element numerical modelling computational method. It is used extensively in mining applications for stress and displacement analysis. The elastic version of Map3D incorporates simultaneous use of fictitious force (FF) and displacement discontinuity (DD) boundary elements. This facilitates the definition of vast mining areas, where computing resources can be optimised by using DD elements for large mining areas away from the areas of interest and then using FF elements to construct detailed and true representation of the three dimensional mining geometry at the areas of interest (van Wijk, 2009). The input parameters used in the numerical modelling study are summarised in Table 4.

Table 4 - Map3D modelling input parameters

Parameter	Value
Elastic Modulus – Host Rock	12 GPa
Elastic Modulus – Coal	3 GPa
Poisson's Ratio – Host Rock	0.25
Poisson's Ratio – Coal	0.3
Vertical Stress Gradient	0.025 MPa/m
Major Horizontal to Vertical Stress Ratio	2.0
Major Principal Stress Trend	10° East of North

Figure 12 shows the numerical modelling results at the final stage (following the development and widening) of the install road. Note that the grid plane where the results are shown is located approximately 2.0 m into the roof. It is evident from this figure that stress magnitudes of approximately 20 MPa are reasonable to expect in this case.

Using (i) the stress measurement data, (ii) the methodologies of Nemcik, Gale and Mills (2005) and Gale and Matthews (1992) and (iii) same input parameters used in numerical modelling, an average stress level of 20 MPa is also obtained in this study to utilise in the buckling failure model. Based on these results it is concluded that the results obtained from the empirical model is in accordance with numerical modelling.

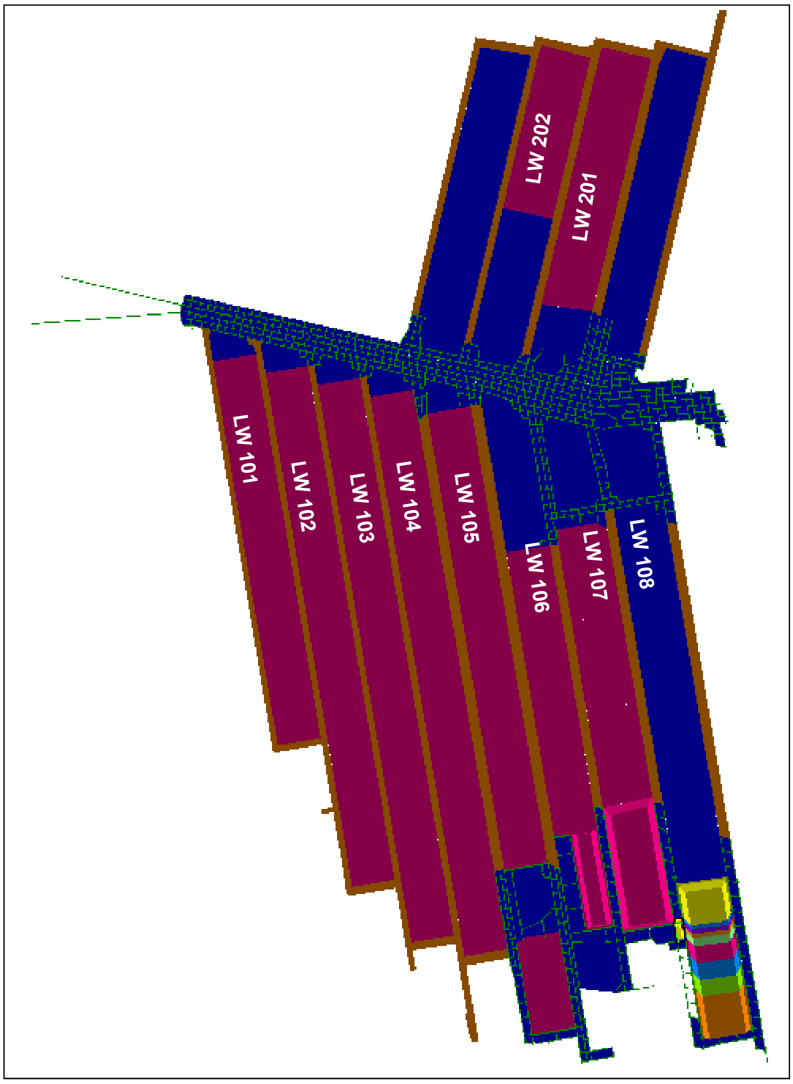

Figure 11 – Moranbah North Mine Map3D model layout

Analytical Modelling

As mention previously, AAMC roof support design methodology requires that all four analytical models (i.e., shear, buckling and suspension tendon and suspension bond failures) are run in a critical area in order to ensure the stability of roof. In the case of LW 108, all areas of the installed road, including the TG and MG intersections and wide areas were evaluated. However, for the sake of the simplicity of demonstration, only the standard 9.5 m wide area is presented in this paper. The probability distributions of input parameters used in analytical models are presented in Figure 13.

Figure 14 shows the probability distribution of expected horizontal stress notching. This indicates that stress magnitudes of up to 40 MPa may be expected.

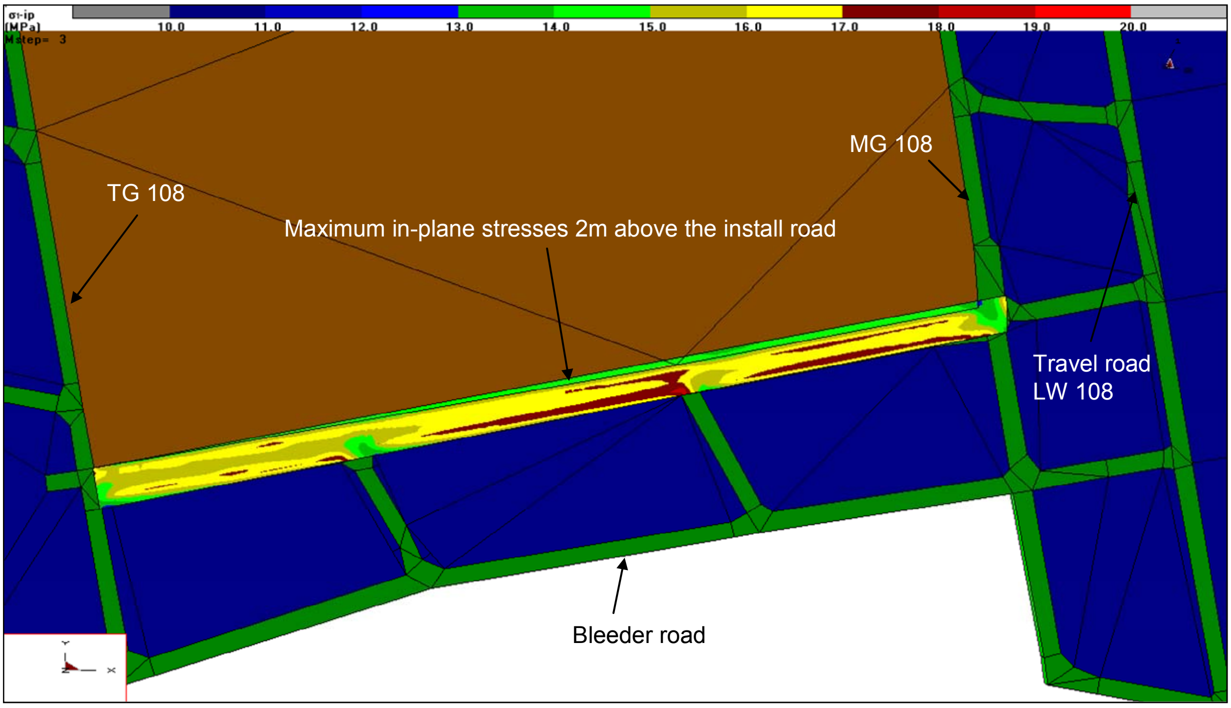

Figure 12 – Numerical modelling results (maximum in-plane stress) at the final stage (following the development and widening) of the install road

Recommended Roof Support

An initial evaluation of required roof support densities was conducted using the analytical modelling. This evaluation study indicated that for 9.5 m wide areas of the install road, the following support patterns provided an acceptable distribution of factors of safety (Figure 15):

- First pass primary support development: 6x1.8 m long X-grade roof bolts installed at 1.0 m spacing.
- First pass secondary long tendon support: 3x8.1 m long tendons (nominal 48 t) installed at 2.0 m spacing.
- Second pass primary support: 6x1.8 m long X-grade roof bolts installed at 1.0 m spacing.
- Second pass primary support long tendon support: 2x6 m long tendons (nominal 45 t) installed at 2.0 m spacing.
- Second pass secondary support: 1x8.1m long tendon (nominal 48 t) support installed at 2 m spacing.

The distributions of the factors of safety for different failure mechanisms are presented in Figures 16 to 19. Using the average values of input parameters, this support pattern indicated the following average factors of safety:

- Buckling failure 3.73
- Shear failure 1.60
- Suspension tendon failure 1.52
- Suspension tendon bond failure 2.91

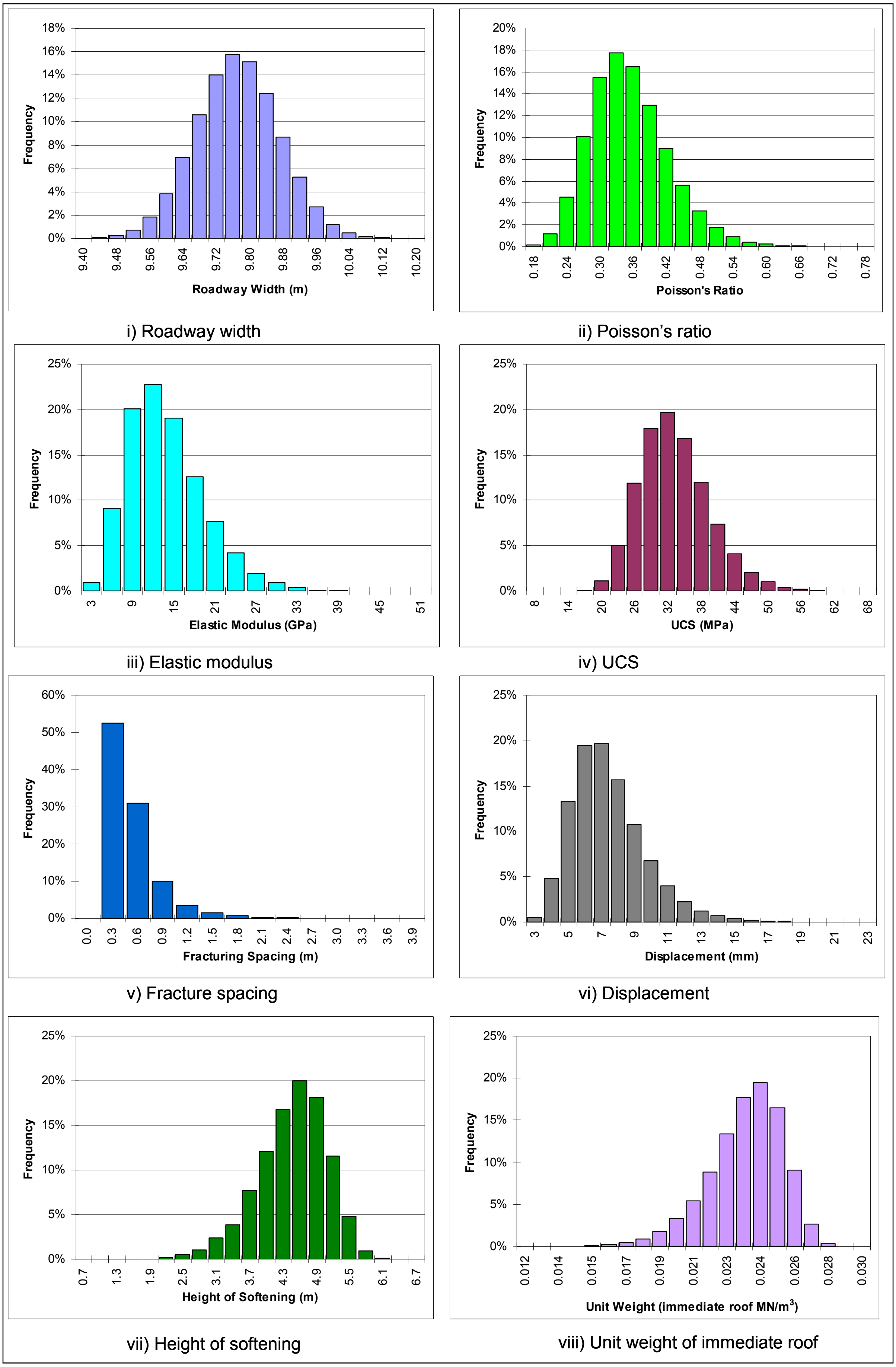

Figure 13 – Probability distributions of input parameters

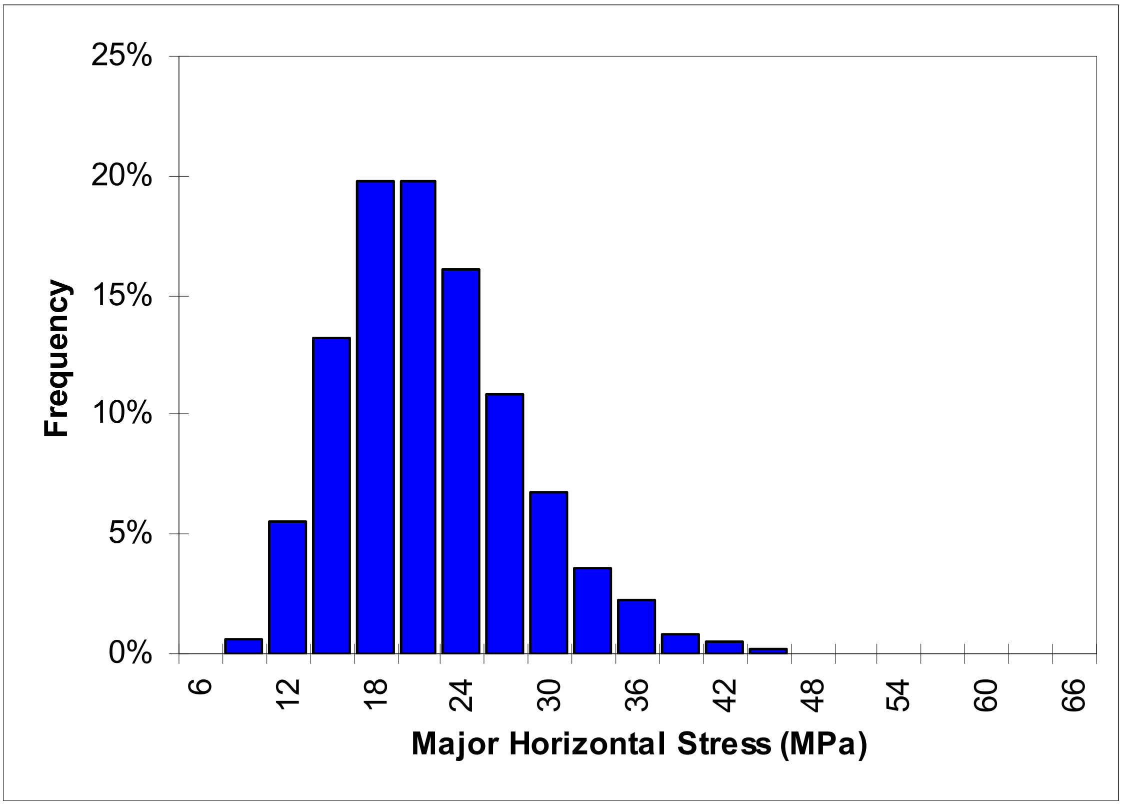

Figure 14 – Probability distribution of expected horizontal stress notching

The resultant likelihood of failures (the areas of under the curve of the factors of safety of <1 in distributions of factors of safety) of the overall system and individual failure mechanisms evaluated are as follows:

- Overall system (i.e., failure in any one of the mechanisms) $3x10^{-6}$
- Buckling failure 0.0023
- Shear failure 0.00004
- Suspension tendon failure 0.0005
- Suspension tendon bond failure 0.0007

The above results demonstrate that while the average factors of safety of different failure mechanisms failure are acceptable, there is a likelihood of failure of 0.0003% of the system due to one or more of these mechanisms. Although this value of likelihood of failure alone may not indicate the associated risks without the calculations of exposure and financial costs, it demonstrates the fact that factor of safety alone cannot give an indication of exposed risks.

CONCLUSIONS

This paper summarises the AAMC's roof support design methodology, which includes analytical, numerical and empirical modelling. The aim of this so called "combined support design methodology "is to ensure the stability of roadways at AAMC underground operations. This methodology currently uses the deterministic approach (calculation of a single factor of safety). The limitations of this design methodology with respect to using a single factor of safety are presented in this paper.

A summary of an improved design methodology, based on stochastic modelling, is also presented. The main advantage of this methodology is that as the design is based on probability distributions of input parameters, the outcome is based on a distribution of factors of safety rather than a single factor of safety.

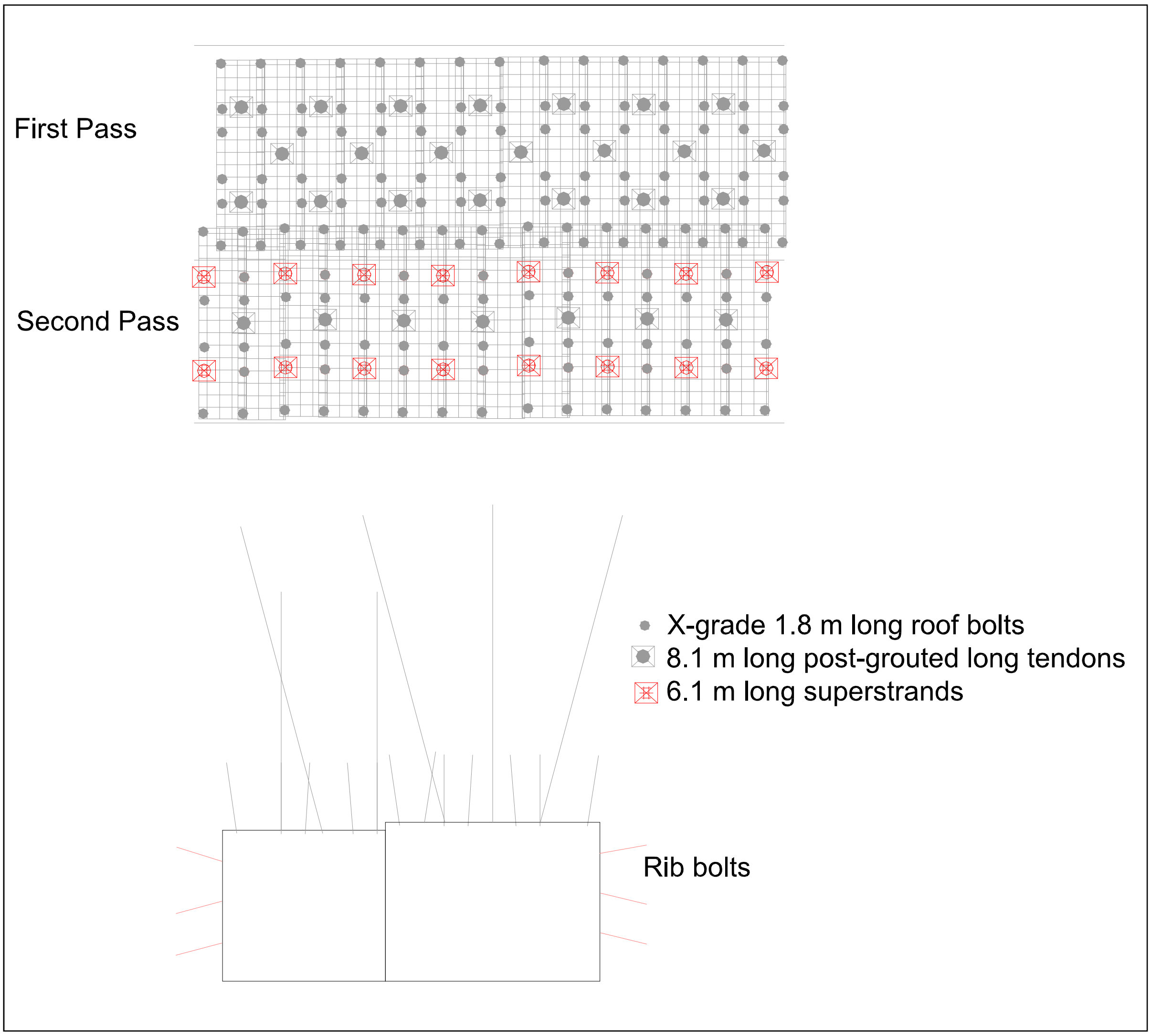

Figure 15 – Recommended roof support design

A demonstration of application of this approach from Moranbah North Coal Mine is presented. The application of this design methodology to LW 108 install road indicated that while the resultant factors of safety of different failure mechanisms against roof falls using the average values of input parameters are acceptable, there is always a likelihood of failure, even though it is very small. This likelihood of failure may also be used to determine quantitative risks (safety and/or financial) associated with the development, widening and installation of the install road. As this methodology allows the user to determine these associated risks, it is considered that this design methodology based on a stochastic modelling is a step forward in the design of roof support systems.

ACKNOWLEDGEMENT

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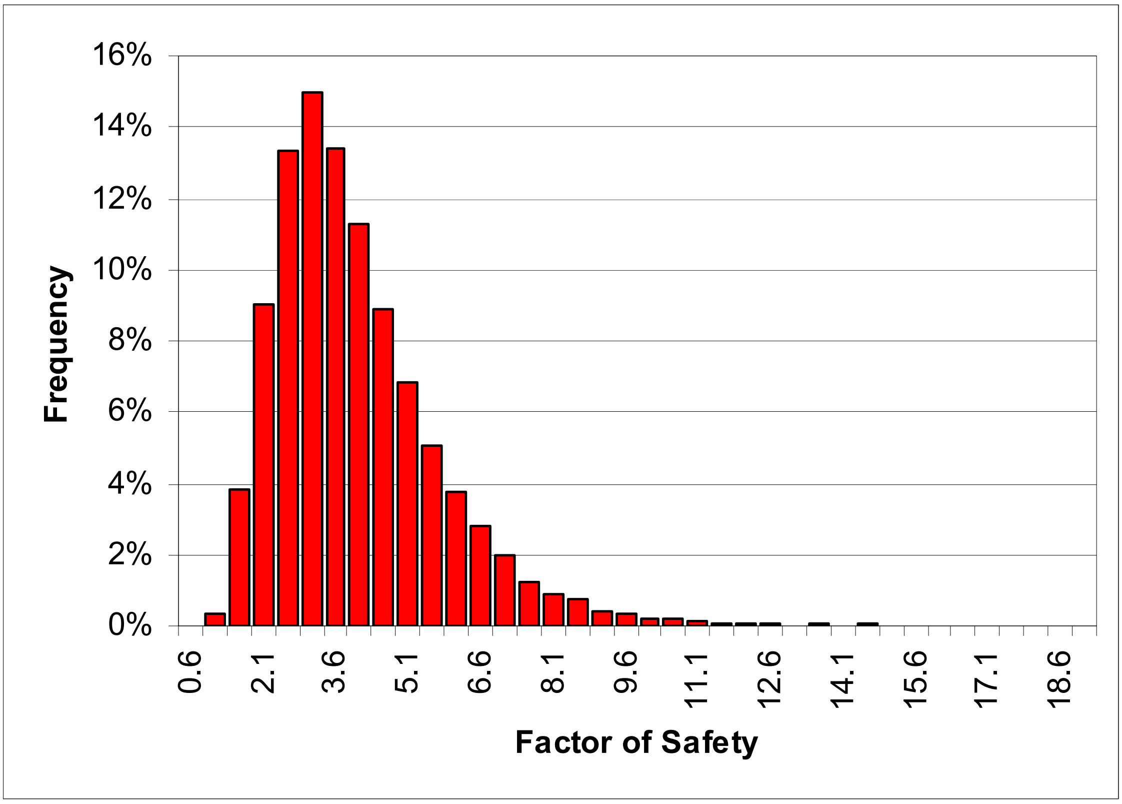

Figure 16 – Probability distribution of buckling failure mechanism (9.5 m wide areas)

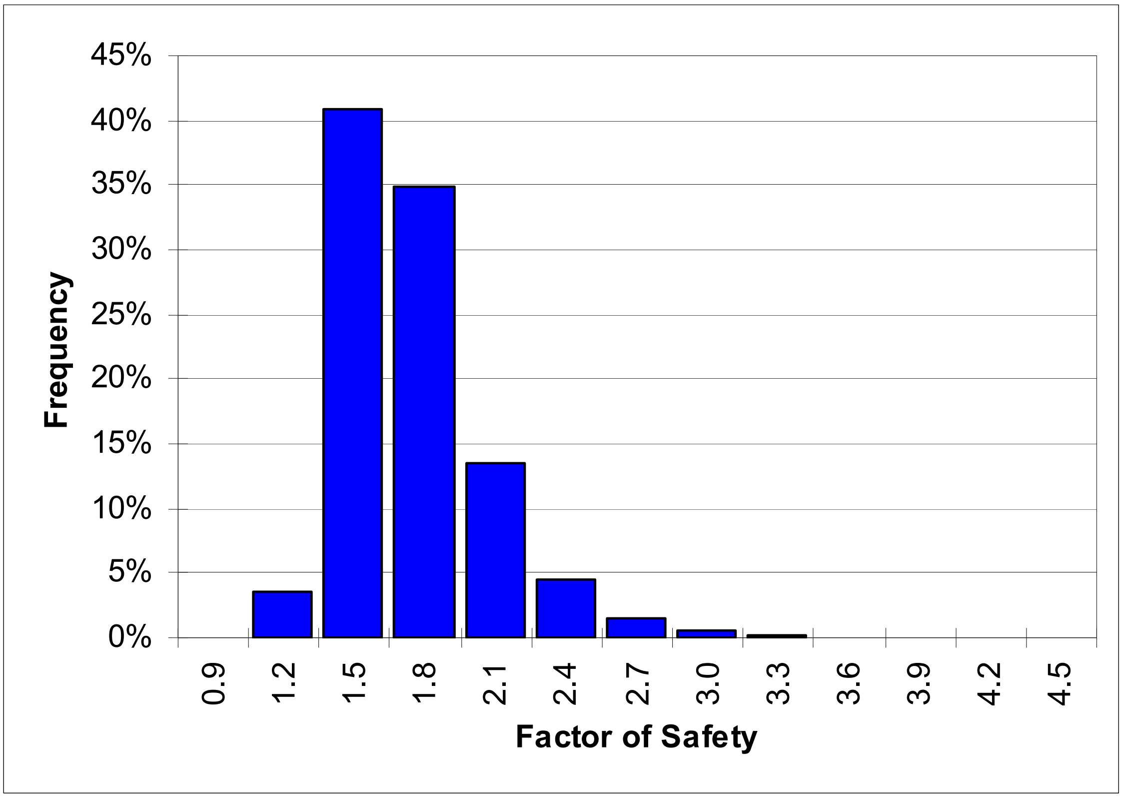

Figure 17 – Probability distribution of shear failure mechanism (9.5 m wide areas)

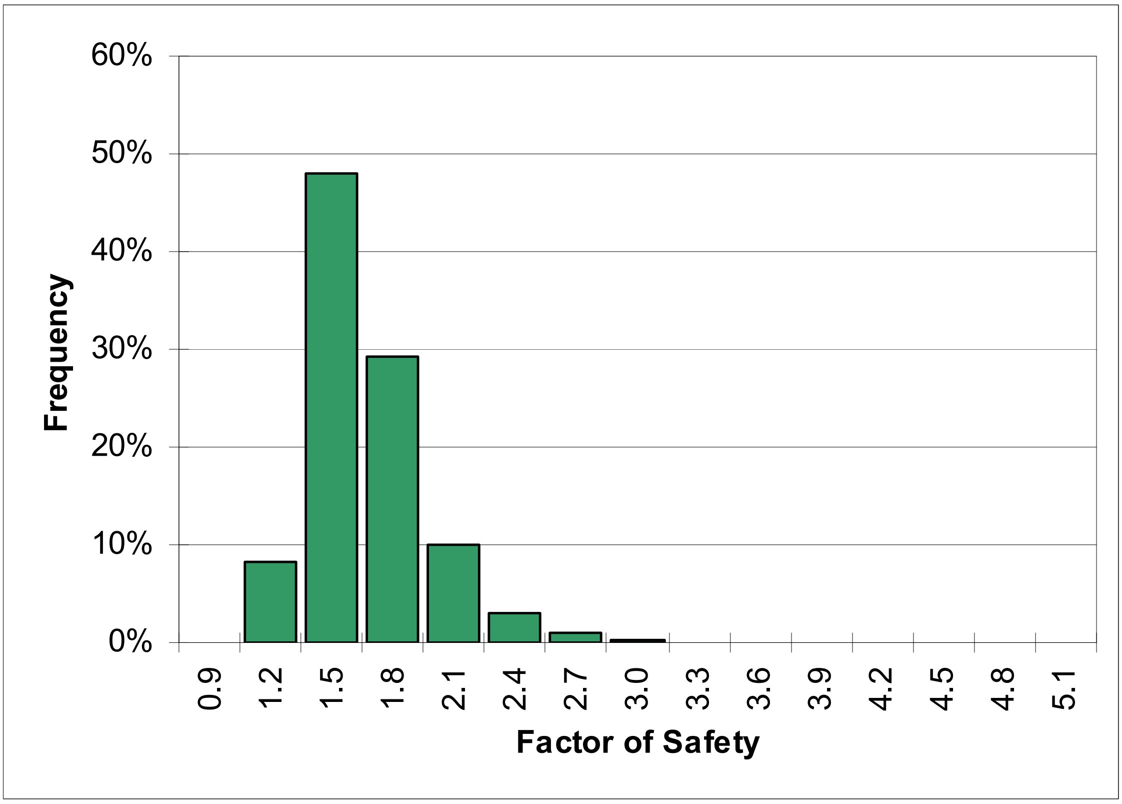

Figure 18 – Probability distribution of suspension long tendon failure (9.5 m wide areas)

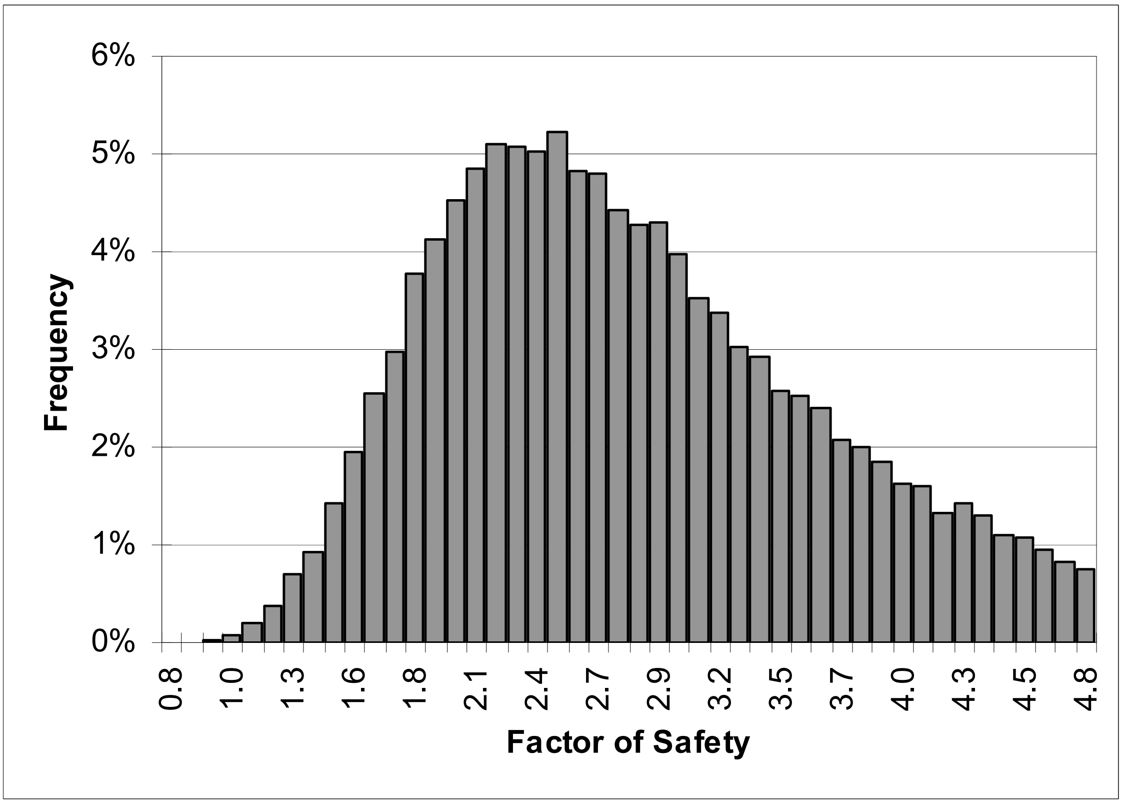

Figure 19 – Probability distribution of suspension bond failure (9.5 m wide areas)

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