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AMCMRR - an analytical model for coal mine roof reinforcement

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AMCMRR – AN ANALYTICAL MODEL FOR COAL MINE ROOF REINFORCEMENT

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ABSTRACT: An analytical model for coal mine roof reinforcement (AMCMRR) has been developed. AMCMRR utilises a Factor of Safety (FOS) approach which is commonly used in all forms of engineering. The starting point in the development of AMCMRR was an existing analytical roof behaviour and roof support design methodology/model originally developed by the second author. This technique was being successfully utilised in the Australian underground coal industry for roof support evaluation and design prior to and during the course of the ALTS 2006 research project and when used was essentially calibrated on a site by site basis.

It has long been recognised that bolts and longer tendons can modify the behavior and load bearing capacity of the reinforced roof via the concept of *beam building*. The break-through in the development of AMCMRR was combining the original analytical model with the ALTS database to effectively quantify this reinforcing effect and this in turn provides the "platform" by which this analytical model can be calibrated for the entire Australian underground coal industry.

The two techniques (analytical-AMCMRR and empirical-ALTS 2009) now work together as a part of an overall and more robust roof support/longwall gateroad design methodology. This paper focuses on the application and use of AMCMRR and the analyses undertaken to quantify the reinforcement offered to the immediate roof via the concept of beam building.

BACKGROUND

At the beginning of 2006, Colwell Geotechnical Services commenced a research project entitled*, "The Future Development and Integration of ALTS & ADRS for Improved Underground Roadway Design"*. The project was funded directly by several of the major coal producers as well as individual Australian collieries. The ultimate goal of the ALTS 2006 Project was to provide the colliery Strata Control/Geotechnical Engineer with user friendly and interactive state-of-the-art software tools and subsequent training/support to assist them with their design requirements and in their strata management role. While the software packages were the major deliverables from the project, it was in fact the geotechnical research which was the main focus of the project and drove the development of the software.

A major component of the project was to develop an analytical model for coal mine roof reinforcement that would complement the ALTS Design Methodology (Colwell and Frith, 2009). This has been achieved and the analytical model is called the, "Analytical Model for Coal Mine Roof Reinforcement" or AMCMRR.

The starting point in the development of AMCMRR was an existing analytical roof behaviour and roof support design methodology/model originally developed by the second author (as discussed by Colwell et al, 2008). This technique was being successfully utilised as a consulting tool in the Australian underground coal industry for roof support evaluation and design prior to and during the course of the project. The basis for this model (and for AMCMRR) is that slender beam behaviour or buckling is typically the dominant behavioural mechanism initially occurring within the immediate coal mine roof measures subject to elevated horizontal stress conditions.

Hoek and Brown (1980) reported that studies conducted by Australian Coal Industry Research Laboratories (ACIRL) in the 1970's (utilising physical models) clearly indicated buckling to be a dominant failure mechanism within the roof and floor of a layered deposit subject to elevated horizontal stress conditions. Uncontrolled roof behaviour of this type may then lead to other failure mechanisms occurring and to large scale roof displacements or roof falls.

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It follows that this dominant failure mechanism (i.e. buckling) should be accounted for in any empirical, analytical or numerical approach to coal mine roof support design. It simply cannot and should not be ignored. While it has long been recognised that bolts and longer tendons can modify the behaviour and load bearing capacity of the reinforced roof via the concept of beam building (Mark, 2000), the major problem faced by the authors in the development of AMCMRR was how to quantify the effect that bolts and longer tendons have on the load bearing capacity of the roof strata via said concept.

MODEL OVERVIEW

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

FOS = load bearing capacity of the structure/applied load *(1)*

This approach is commonly used in coal pillar design worldwide with the UNSW Pillar Design Procedure (Galvin et al, 1999) being one such example. In this case the structure is the coal pillar where the strength of the coal pillar is given by a specific equation that has been determined empirically, based on an industry database of stable and failed pillar cases, typically under reliably inferred full tributary area loading conditions. The FOS is essentially a risk based measure of the likelihood of the design being inadequate with acceptable values being related to the likely consequence of the design being inadequate and the associated impacts (business, safety or otherwise).

In using AMCMRR it is critical that the user understand that the Factor of Safety has the following general definition:

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

It is not a FOS against a roof fall occurring as (a) the conditions under which a roof fall finally occurs are not well defined and (b) practical mining considerations requires that the roof be maintained as stable as possible during longwall retreat so as to minimise any potential impact on face production. Furthermore productivity and safety can be adversely affected by simply excessive roof convergence trapping equipment or deteriorating roof conditions necessitating the installation of additional roof support. Risk-based mining considerations dictate that designing roadway roof support against the occurrence of a roof fall is inappropriate as we need to be designing roof support against the triggering of the colliery's Trigger Action Response Plan (TARP).

The consequence of an inadequate design when using the above-defined FOS is logically the triggering of the colliery's TARP as a part of its strata management plan and the installation of additional support. It is not the imminent occurrence of a major roof fall and this always needs to be kept in mind when considering the actual magnitude of an adequate design Factor of Safety.

Consistent with the above, AMCMRR does not incorporate standing support (i.e. timber cribbing, link-n-locks, tin cans etc) as standing support is not used to reinforce the rock mass in the same way as encapsulated/tensioned bolts and cables. In terms of standing supports' contribution to overall roof stability/roadway serviceability (as a part of a roof support strategy); standing support is, in practical terms, utilised to prevent a potentially failed rock mass becoming a fall. It therefore "supports" rather than reinforces the rock mass.

It is noted that cables can also act in a similar manner to standing support, in that they can support a "dead weight" load where the rock mass has yielded or has essentially lost its ability to resist the applied horizontal stress. Some practitioners still design bolt/cable installation patterns in this manner where the dead weight load is directly related to an assumed height of softening. However this is an example of designing against a roof fall because if the design fails, the roof falls in. AMCMRR evaluates roof stability and support requirements much earlier in the instability process when reinforcing support can be applied. It is the authors' contention that this is vastly preferable in terms of tendon support (i.e. reinforcing) effectiveness and more importantly, design reliability and robustness.

Within AMCMRR, bolts and cables are considered within the design process (and in the calculation of the FOS) to firstly increase the load bearing capacity of the strata (which is limited by the material's yield strength) and where longer tensioned cables are employed they also contribute to overall roof stability by directly resisting the applied load via the concept of mechanical advantage (MA – Frith, 2000). AMCMRR provides for a static load-balance design, where the goal is to design a tendon roof support strategy which allows the reinforced roof to maintain the horizontal stress acting across the roof so as to prevent the onset of a process of delamination and/or buckling that (otherwise left "unchecked") could lead to a major roof fall.

As previously indicated; in terms of roof support design in some instances factors of safety have been calculated based on the height of softening and a related dead weight load supported by the cables. However it is important to note that in this instance the "structure" under consideration when calculating the FOS is in fact the cables not the roof. Also with this approach it is accepted that the horizontal stress has exceeded the yield strength of the roof material and essentially a "failed" rock mass is being suspended by the cables.

Whilst it is far less common to do so, there is no obvious technical reason as to why roof stability in mine roadways cannot be evaluated and designed for where the reinforced roof is the structure under consideration in equation 1. The problem has always been in being able to reliably assign magnitudes or quantities to the various components of the equation. However as with the approach taken for coal pillar design (i.e. the use of an empirically derived strength equation rather than one based on first principles), industry or individual mine site experience can potentially be used to "calibrate" various elements of the problem and so allow an industry or site specific FOS approach to be adopted.

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

FOS = [load bearing capacity (reinforced roof strata) + load resistance via MA]/applied load (2)

As will be discussed, the load-bearing capacity of the reinforced roof strata is a function of the installed roof support and the roof strata itself. It is noted that this combination of the roof strata and roof support into a single load-bearing entity (as a part of equation 2) is not the same as reinforced concrete for example. The use of steel reinforcement in concrete is specifically aimed at overcoming concrete's poor performance in tension rather than increasing its load bearing capacity in terms of compression. In the case of the analytical model, the applied load is compressive as a result of the horizontal stress acting. The roof support via the roof reinforcement mechanism of beam building increases the load bearing capacity of the strata (up to its yield strength) to resist the horizontal compressive forces and so limit vertical strata movements.

APPLICATION OF THE MODEL

The analytical design model is focused primarily on the design of primary and secondary tendon roof support for the gateroads associated with longwall mining. Figure 1 is a plan schematic of a typical Australian longwall mining layout, depicting a fully extracted longwall panel, one currently being extracted and a third where the twin gateroads (A and B Headings) are still to be completed to fully delineate the longwall panel.

AMCMRR deals with those cases where the horizontal stress acting can be realistically calculated (as a result of industry research) and importantly the roadway width remains reasonably constant. Therefore the analytical model cannot currently be used with respect to tailgates subject to double (or 2^{nd}) pass longwall extraction and ALTS 2009 should be employed as state-of-the-art design for such cases. Realistic estimates of the horizontal stress acting is a critical input parameter and with respect to tailgates subject to double (or 2^{nd}) pass longwall extraction it is assessed that further industry research is required in relation to this issue. Also effective roadway widening due to rib spall/softening is far more variable when a ribline has been subject to both maingate and tailgate loading (Positions c and d – Figure 1).

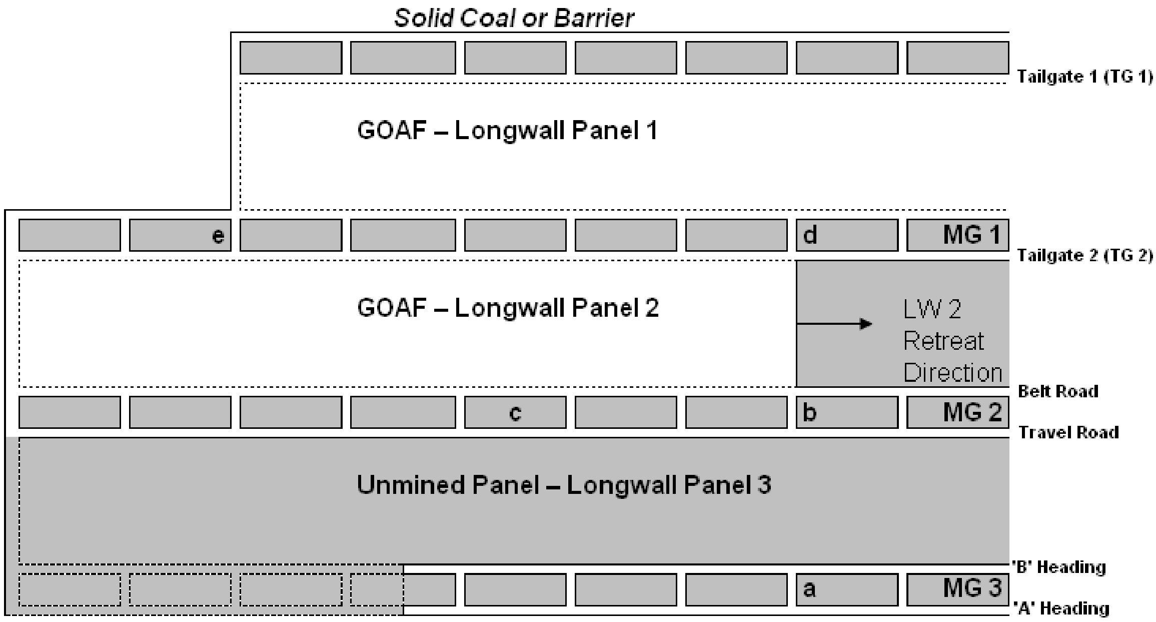

Figure 1 - Typical Australian longwall layout

AMCMRR considers the following four cases:

1. Roadway development and the maintenance of a stable roof prior to any increase in horizontal stress as a result of adjacent longwall extraction (position a – Figure 1).

2. Maingate belt road roof support design to deal with the increase (or notching) of the *in situ* horizontal stress about the belt road intersection with the travelling longwall face (position b – Figure 1).

3. A tailgate where there is no adjacent goaf; for example where the tailgate is the first in a series of longwall panels (e.g. Tailgate 1 – Figure 1). In this instance the tailgate roof is subject to an increase (or notching) of the *in situ* horizontal stress, which is referred to as a *Tailgate - Single Stress Notch*.

4. The 4th scenario is a transient, but not uncommon situation where a tailgate roof is subject to a *Super Stress Notch* (position e – Figure 1). To occur, the longwall commences inbye of start-line of the previous LW panel, in this case LW 2 in relation to LW 1. Significant horizontal stress increase occurs as the faceline of LW 2 approaches and passes the start-line (or installation face) of LW 1.

In terms of the use and application of AMCMRR there are four basic components:

1. Evaluation of horizontal stress acting across the roof within individual roof units at various key points in the mining process (Colwell *et al*, 2008 and Colwell & Frith, 2009).

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

3. Determining the reinforcing effectiveness of the installed roof support (bolts and/or long tendons) in terms of their impact on controlling the horizontal stress acting. For primary roof bolts this is done by evaluating their ability to increase the effective beam thickness of the roof strata within the bolted interval. For long tendons they are evaluated both in terms of their contribution to beam building within the bolted interval and the control of buckling via the concept of Mechanical Advantage (Frith, 2000).

4. Utilising a load-balance approach (which incorporates aspects of slender beam behaviour and mechanical advantage) the FOS is calculated. Engineering judgement needs to be applied in

selecting a suitable FOS for design purposes this being a risk-based consideration that is always discussed with mine management as part of finalising design outcomes.

The following specifically discusses the analyses undertaken to quantify the reinforcement offered to the immediate roof via the concept of beam building.

LOAD BEARING CAPACITY

As previously discussed, roadway roof behaviour and associated instability is primarily based around the uncontrolled buckling of slender horizontal beams under the action of horizontal stress. Therefore at "its core" the analytical model is based on slender beam behaviour and that the bolts and cables can modify that behaviour via the roof reinforcement mechanism of beam building. The concept being that the bolts and cables create "thicker" beams within the reinforced section (or the bolted interval) and that a thicker beam will have a greater load bearing capacity than a thinner beam.

A clear example of this concept of beam building is demonstrated in Figure 2, which illustrates the behaviour (or response) of a section of maingate roof during and subsequent to longwall retreat. Under the action of horizontal stress, bedding and or weakness planes can be forced apart and thinner discrete beds or beams of roof material start to form. This inevitably results in discernible roof displacements and roof softening (i.e. delamination) for a distance into the roof.

The dashed horizontal line on Figure 2 represents the top of the 1.8m primary bolted interval and there is an obvious difference in roof behaviour at this location within the roof. The response of the roof within the bolted interval is that of thicker beams as compared to the roof material overlying this interval up to the extent of the roof softening, which is approximately 4m.

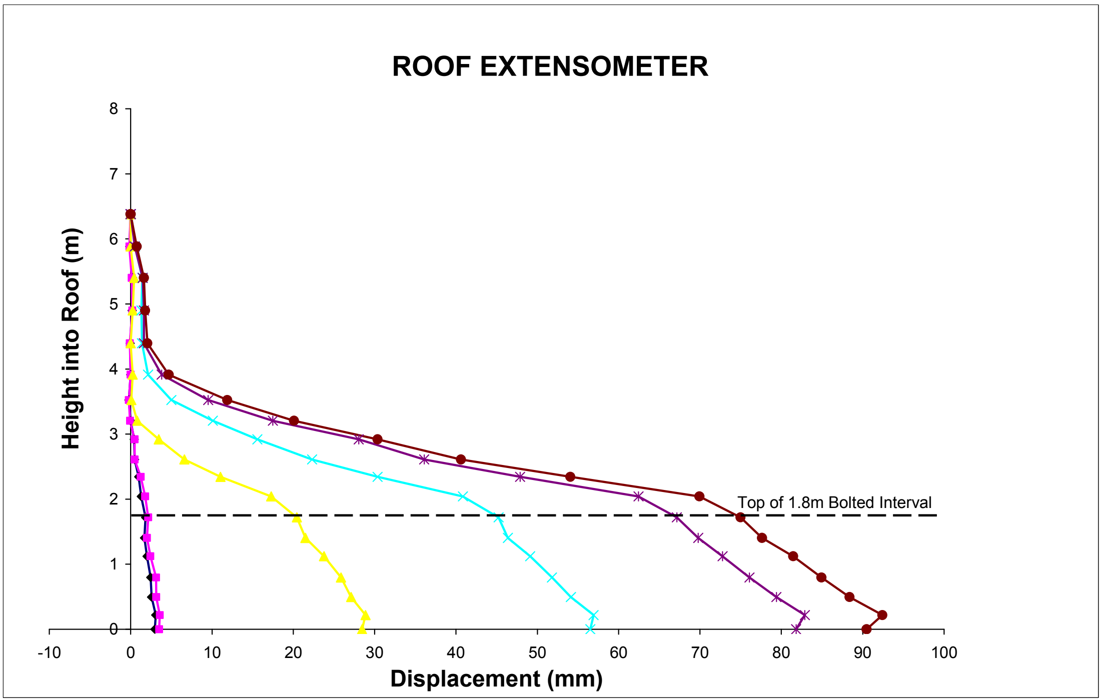

Figure 2 - Roof behaviour adjacent to longwall extraction

The starting point in developing a relationship to quantify the reinforcement concept of beam building is first to understand the behaviour and load bearing capacity of the unreinforced rock units (designated as P_{unit}) within the immediate roof and the input parameters required. Once this is understood a relationship is then required which relates the reinforced beam thickness (RBT) to the initial or unreinforced beam thickness which is referred to as the effective fracture spacing (FS_{eff}). The FS_{eff} for an individual rock unit is derived from the individual rock Unit Rating, which is a part of the Coal Mine Roof Rating (CMRR) calculation (Colwell, 2009).

Evaluating the Load Bearing Capacity of the Unreinforced Roof Units (P_{unit})

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

Behaviour outside the Euler range can be defined by a number of different structural concepts. For the purpose of this model use will be made of what is termed as the Johnson formula (see Foley, 2009 or Beer, Johnston and DeWolf, 2006 for more general information on this topic). Utilising these concepts of beam behaviour under axial load in conjunction with the roof's material & physical properties an estimate of its load bearing ability (P_{unit}) can be deduced.

Structural elements that are loaded axially are generally referred to as columns and in terms of buckling have typically been divided into three general types:

(i) Short (stumpy) columns
(ii) Intermediate columns, and
(iii) Long (slender) columns.

Long (slender) columns fail by buckling i.e. large lateral deflections and the failure is elastic. A short beam will not fail due to buckling, as the ratio of the beam length to the effective cross sectional area is too small. Rather a short, 'thick' beam, axially loaded, will fail in simple compressive failure; that is when the load/area of the beam exceeds the allowable stress. Columns of intermediate slenderness exhibit a combined failure mode involving both yielding and large lateral deflections.

The critical or allowable stress associated with long beams/columns is governed by equation 3 (Euler Formula).

$$\sigma_{crit} = \pi^2 E / [12(L_{eff} / d)^2] \qquad (3)$$

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

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

There are a number of semi-empirical formulas for buckling in beams/columns in the intermediate length (and short) range. One of these is the J.B. Johnson Formula (refer Foley, 2009). The J.B. Johnson formula is the equation of a parabola with the following characteristics. For a graph of stress versus slenderness ratio, the parabola has its vertex at the value of the yield stress on the y-axis. Additionally, the parabola is tangent to the Euler curve at a value of the slenderness ratio, such that the corresponding stress is one-half of the yield stress.

The Johnson equation for the allowable stress is as follows:

$$\sigma_{crit} = [\, 1 - (L_{eff} / \acute{r})^2 / (2\, C^2)]\, \sigma_y \qquad (4)$$

Where, $\acute{r}$ is the beam's Radius of Gyration and C is the beam's critical slenderness ratio

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

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

Essentially when the beam's slenderness ratio ($L_{eff} / \acute{r}$) is greater than the beam's critical slenderness ratio (C) then equation 3 is used to calculate the beam's load bearing capacity and when the beam's slenderness ratio less than C then equation 4 is invoked.

Therefore in undertaking these analyses the information required is modulus (E) and σ_y (where σ_y = 0.7 x UCS) of the rock unit as well as the beam's effective length (L_{eff}) and thickness (d). In terms of the individual beams that will form within the roof; firstly it is assumed the end fixing condition is pinned and therefore L_{eff} equals the roadway width and secondly the beam thickness is equal to FS_{eff}.

Evaluating the Load Bearing Capacity of the Reinforced Roof Units (P_{RBT})

As previously discussed the analytical model is based on slender beam behaviour and that the bolts and cables can modify that behaviour via the roof reinforcement mechanism of beam building. The concept being that the bolts and cables create "thicker" beams within the reinforced section (or the bolted interval) and that a thicker beam will have a greater load bearing capacity than a thinner beam. Therefore in terms of an equation this concept is expressed as:

$$RBT = f\ (FS_{eff}\ \&\ \text{a measure of the installed level of roof support}) \qquad (5)$$

Essentially the only practical way that the above relationship can be established is empirically via back-calculation of a database with reasonable assumptions made. It was decided to use the ALTS primary roof support database (comprising 109 cases) in an attempt to establish this relationship. Primary support is defined as roof and rib support which is installed off the continuous miner or off a mobile bolter as part of a cut & flit operation. The main reasons for using the primary roof support database were:

1. There is far less variability with respect to the installation of primary bolts as compared to secondary cables. A very large proportion of the primary bolts now utilised in Australian are of x-grade (or close to x-grade) steel, are installed off the miner near the face as part of a "cut and bolt" operation with a two-speed resin system and are then tensioned within a range of approximately 5 t to 10 t. Furthermore by far the predominant lengths used in Australia are the 1.8 m and 2.1 m bolts (comprising 96 of the 109 cases).

2. As indicated above, in Australia the vast majority of primary support is installed near the face as a part of a "cut and bolt" operation. While the distance between the last line of support and the face may vary it will be within an operationally viable range and therefore similar across the industry. Furthermore there is some level of flexibility in that distance at a minesite level where for example the miners will have the option (if the ground conditions warrant) to "close up" that distance to limit roof displacement at the time of installation. The level of roof movement at the time of installation is a critical aspect with respect to the effectiveness of roof reinforcement and it is assessed that the level of roof movement associated with the installation of primary support is far less variable (across the industry) when compared to that of longer cables.

3. There is greater confidence with respect to the estimate for the horizontal stress acting perpendicular to the roadway orientation on development (i.e. σ_R-Dev, Colwell and Frith, 2009) as compared to those scenarios associated with a *notching* of the horizontal stress during longwall retreat (e.g. *maingate stress notching*).

Prior to undertaking the analyses a logical reduction of the primary roof support database was necessary to both simplify the analyses and to recognise certain geotechnical limits. Furthermore one important assumption was required as a part of that process. The assumption being that for development, Australian longwall operations typically aim to achieve a minimum reinforced roof FOS of approximately 2 (albeit unknowingly in most cases as there is no FOS based design for primary support to refer to) and that the eventual trendline associated with the database analyses will represent a reinforced roof FOS = 2.. Experience would suggest that the Australian industry prefers "Table Top" roof conditions on development and a minimum reinforced roof FOS of 2 would reflect that choice.

While this assumption plays a crucial role in terms of the analyses it is not a critical assumption in terms of utilising AMCMRR. It has always been the authors' intent that the analytical model resulting from this project should initially be used and calibrated on a site by site basis. This in fact is typical of how many (if not most) analytical and numerical models are utilised by experienced geotechnical engineers. Furthermore any suggested Factors of Safety emanating from the database analyses will be relative to this assumption. The database was reduced in the following manner:

1. It was decided to initially only include those cases where the individual units within the bolted interval were geotechnically very similar i.e. similar UCS and FS_{eff}, such that the individual rock Unit Ratings

were similar. This ensured that the reinforcement mechanism of beam building was predominantly being analysed and not being complicated by other roof reinforcement mechanisms such as when a bolt is anchoring into a "strong bed". This reduced the database from 109 to 97 cases.

2. While the assumption is that collieries aim for a reinforced roof FOS of 2, in reality there a quite a number of collieries where the unreinforced roof FOS on development (i.e. P_{unit}/σ_R-Dev) is already greater than 2 and to a degree the bolts (in terms of overall roof stability on development) are essentially simply "holding the roof mesh in place". It was found that this typically related to those collieries where the CMRR > 50 (i.e. the strong roof category). Including such collieries in the analyses, where the unreinforced roof FOS > 2, is simply of no benefit and actually detrimental to the analyses in terms of developing a realistic relationship between RBT and FS_{eff} & the installed level of roof support. Furthermore to ensure that there is essentially a "gap" between the unreinforced and reinforced roof FOS, it was decided to eliminate those cases where the unreinforced roof FOS > 1.5. This reduced the number of cases from 97 to 75 cases.

3. There are those instances where no matter what level of roof support is installed the reinforced roof FOS cannot be greater than two (2) due to the limiting factor being the yield strength of the material. For example if the rock unit's UCS is 20 MPa then its yield strength is taken to be 70% of the UCS or 14 MPa and if σ_R-Dev ≥ 7MPa then independent of the number of bolts installed (and the resultant RBT), the fact is that in this situation the reinforced roof FOS can never be greater 2 as the reinforcement offered by the bolts cannot make the material stronger than its yield strength. Accordingly it was decided to eliminate those cases where 2 times σ_R-Dev ≤ 0.7 UCS. This reduced the database form 75 to 60 cases.

Finally only the "headings" section of the primary support database was used as the beam length is a critical factor in terms of the back-analyses. While there is relatively good control of roadway width with respect to headings throughout the industry, the formation of intersections varies from colliery to colliery and there is significant variation in the effective control of pillar corners. Therefore there are operational issues that also impact on the resultant level of primary support utilised within and adjacent to "intersections".

Based on the assumptions previously discussed and the logical reduction of the database, it was found that an exceptionally strong relationship (R^2 = 0.91) existed between the ratio of RBT to FS_{eff} and the primary roof support rating (PRSUP, refer Colwell and Frith, 2009) which is illustrated in Figure 3. It is worth noting that the individual components of the PRSUP calculation (i.e. length of the bolt, ultimate tensile strength of the bolt and density of the pattern) were also separately and collectively significant predictors of the RBT/FS_{eff} relationship.

When calculating the RBT based on the installed level of primary support (as measured by PRSUP) and FS_{eff} (derived from the geotechnical logging/geomechanical testing of the core) the trendline displayed on Figure 3 can be expressed as:

$$RBT = 0.28e^{0.05341 * PRSUP} \times FS_{eff} \qquad (6)$$

In terms of AMCMRR the load bearing capacity of the reinforced rock units (designated as P_{RBT}) is then calculated in the same manner as that previously outlined for P_{unit}, while now utilising RBT rather than FS_{eff} for the effective beam thickness. Once again it is important to note that irrespective of the resultant RBT the beam's load bearing capacity will be limited by the material's yield strength (refer equation 4).

Where additional roof support is installed subsequent to development and prior to longwall extraction then equation 6 is still utilised to calculate RBT, where PRSUP is replaced by the total ground support rating (GRSUP, refer Colwell and Frith, 2009). However it should be noted that where longer cables are installed then their incorporation within the PRSUP and GRSUP calculations has been modified to account for the impact of any additional roof movement subsequent to installation of the primary roof bolts.

DISCUSSION

The reinforcement mechanism or concept of beam building (as discussed by Mark, 2000) associated with the installation of roof bolts has long been recognised in the underground coal mining industry. While numerous researchers (e.g. Peng 1998, Gale et al 1992 and Seedsman et al 2009) have discussed the various mechanisms by which the bolts act to "create thicker beams" (i.e. by maintaining friction on bedding planes etc) this is the first such study that has in a practical way attempted to quantify the beam building effect and then incorporated that effect within an analytical model.

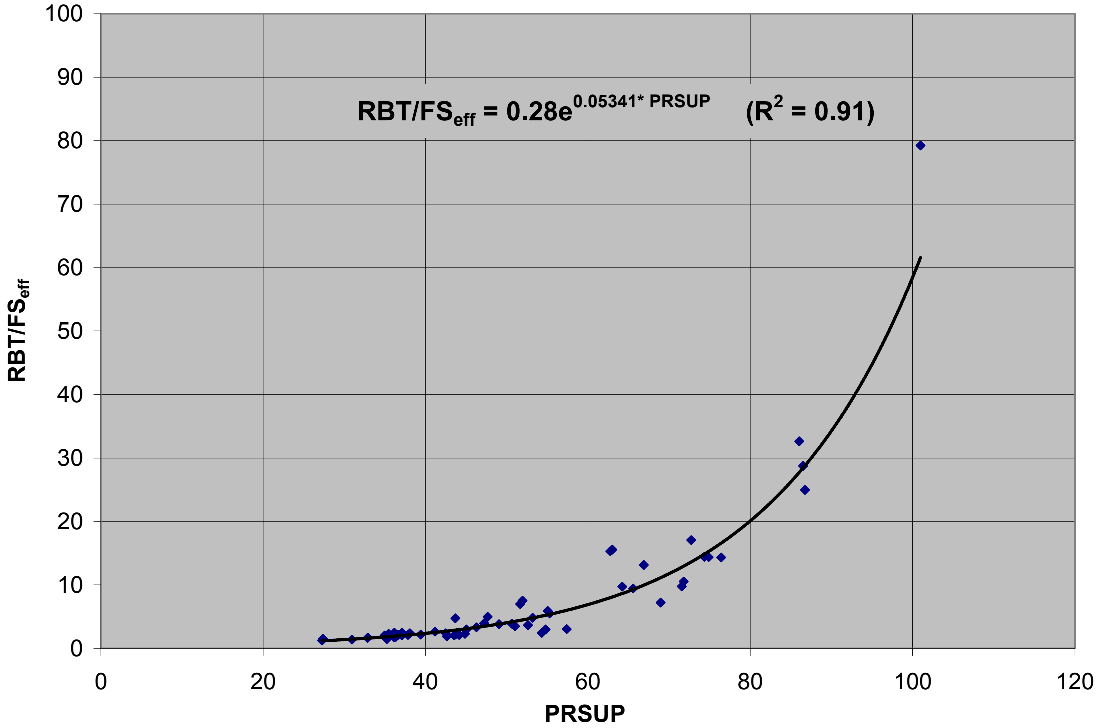

Figure 3 - Relationship between RBT, FSeff and PRSUP

Slender beam behaviour (i.e. buckling) is the dominant failure mechanism within a coal mine roof subject to elevated horizontal stress conditions and should be accounted for in any empirical, analytical or numerical approach to coal mine roof support design. It simply cannot and should not be ignored! So why has it taken so long for a model to be developed which directly includes this mechanism and has also attempted to quantify the increase in load bearing capacity of the reinforced roof via the concept of beam building?

By 1980 it was clear via observation, measurement and the use of physical models that buckling was a dominant failure mechanism associated with a coal mine roof, rib and floor, however its incorporation in geotechnical models associated with coal mine roof and rib reinforcement virtually disappeared for some 15 years in Australia. At that time the geotechnical engineer had essentially four forms of modelling open to them for both design and research purposes being empirical, analytical, physical and numerical. However with the increased power of computers there became (and to some degree there still is) a clear preference within the worldwide rock mechanics fraternity to utilise numerical modelling for research purposes and physical modelling has all but vanished.

Unfortunately numerical modelling (then and now) cannot truly represent the geotechnical environment in sufficient detail so as to take account of dominant behavioural and associated failure/reinforcement mechanisms associated with a coal mine roof and tendon reinforcement. Because of these limitations such models require manipulation of the host rock's material properties in an attempt to *mimic* the behaviour of the unreinforced and/or reinforced rock mass. Because numerical modelling could not account for buckling most numerical modellers simply ignored this fundamental mechanism occurring within the roof, ribs and floor. The use of numerical modelling in this manner has actually been detrimental to advancing our knowledge and understanding of both the unreinforced and reinforced rock mass behaviour.

AMCMRR utilises a FOS approach which is commonly used in all forms of engineering and it was intended that the ALTS database would provide the basis by which certain aspects of the model could be improved and/or calibrated and indicative design FOS values could be provided. The prospect was that in combining these two design/evaluation techniques (empirical – ALTS 2009 and analytical - AMCMRR) an even more robust roof support design procedure would result, this being analogous to the pioneering work of Salamon and Munro (1967), and later the University of New South Wales (Salamon et al, 1996), in developing coal pillar design for bord and pillar workings to its current level of reliability.

CONCLUSIONS

The authors' experience in developing geotechnical models and design tools for the Australian underground coal industry suggests that the geotechnical environment and the way in which roof and rib support interacts with the rock mass are complex issues and without prudent simplification, the complexity of the problem will overwhelm all current geotechnical methods of modelling. Without question judicious simplification is at the heart of all engineering design as it is in all branches of science (termed *reductionism*). However the problem should not be oversimplified (i.e. the dominant failure mechanisms or critical data input parameters should not be ignored).

Empirical modelling has its limitations and an important maxim in the application of any empirical model is that the model should only be utilised within the bounds of the database from which it was developed. However, the great advantage of this approach is its firm links to actual experience and a worldwide proven track record in the underground coal industry of providing solutions to complex mining issues in a timely and cost-effective manner and therefore if judiciously applied (as all models should be), it allows for credible design within the rigours of a well constructed strata management process.

The authors contend that a clear understanding of the dominant mechanics of the geotechnical problem under consideration is required before a credible model (utilising any one or a combination of the three forms of modelling) can be developed for geotechnical design purposes. This paper demonstrates that a combined empirical and analytical approach is currently the most practical way of developing credible geotechnical design tools for the Australian or indeed any other underground coal industry.

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