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IMPLEMENTING A SUSPENSION DESIGN FOR COAL MINE ROADWAY SUPPORT

Ross Seedsman

ABSTRACT: Suspension of the immediate roof is one of fundamental ground control strategies available to a coal mine engineer. Correctly implemented suspension offers the greatest improvement in roof stability and greatest reduction in longwall risk. In some circumstances an alternative strategy based on the reinforcement of bedding parting may be more appropriate. For the control of maingates ahead of retreating longwall faces the ideal suspension support (if required) consists of angled, partially debonded, medium-length tendons installed as far behind the development face as possible. For situations where the roof may be subjected to tensile horizontal stress, immediate support by equally spaced short vertical tendons is required. The step away from fully-grouted tendons improves their survivability during the onset of compressive failure in the roof, minimises the risk of isolated loading, and allows a more robust TARP for roof movement. All suspension systems require a sling or truss between the suspension elements.

BACKGROUND

Typically the immediate roof of a coal mine roadway consists of a regular assemblage of rectangular prisms of rock or coal defined by bedding and joints (Figure 1), with an important exception if the joints are not aligned normal to bedding such as near faults. With the joints normal to bedding, the roof horizon has a finite compressive strength but zero tensile strength. The logical framework identifies suspension as the appropriate response to disordered roof, compressive failure and tensile failure (Seedsman, 2012). As alternatives, prop support cannot be used if the roadways are to be used for traffic flow and beam reinforcement cannot be used if the rocks within the beam have undergone compressive failure, the immediate roof is in tension, or if there are discontinuities dipping at less than about 70°. Recently, suspension design has been dismissed as fundamentally flawed (Frith, 2011, Frith and Colwell, 2011) on the basis that roof collapses have occurred even though calculations based on a suspension design give installed capacities in excess of the dead weight of the collapsed volume. This paper examines the details of a suspension design and highlights where engineering skills and judgement are required.

Figure 1 - Representative coal mine roof geology and the logical framework for coal mine roof control

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There are two distinct applications for suspension designs in underground coal mines (Figure 2):

- Onset of compressive or tensile failure in the immediate roof, requiring that the broken and de-stressed rock mass be suspended from the intact rock that is arching over the roadway (Figure 2a).
- A possibly unstable immediate roof with a spanning unit within the bolted horizon (Figure 2b). This application requires determination of what is in fact a spanning unit.

**Figure 2 - Two suspension conditions (a) from a stable stress arch (b) from a strong bed (Stillborg, 1994)**

**DESTRESSED ZONE AFTER COMPRESSION FAILURE**

**Extent of compressive failure**

Seedsman (2011) proposed a method to predict the height of compressive failure consistent with recent developments in the understanding of brittle rock strength and calibrated to mining experiences. In the method, the height of compressive failure is used as a proxy for the “height of softening” as determined by extensometry, with the conservative assumption that the height of softening represents the height to which the roof would collapse if it were to be unsupported. The rock strength criterion is further discussed in Seedsman (2013). Combined with an elastic analysis of stress in a transversely isotropic (bedded) material, the method allows the calculation of the maximum height of failure above any shaped opening as a function of the Uniaxial Compressive Strength (UCS), the vertical and horizontal stresses (Figure 3). Figure 3 shows how the height of compressive failure can increase substantially between roadway development and subsequent longwall retreat. At the development face, normal bolting lengths (1.5 m - 1.8 m) may be adequate but at the maingate longer tendons would be required. Close inspection of Figure 3 reveals lower strength factors towards the roof/rib corners which can be interpreted to be a reflection of stress guttering reported in underground roadways.

The height of the zone in the roof with a strength factor less than unity depends on the ratio of the UCS to the pre-mining vertical stress (Roof Strength Index – RSI), the ratio of the horizontal to vertical stress and the roadway span (Figure 4). Very high values of the RSI are required for the failure zone to disappear, and it is noteworthy that the zone needs to be in excess of 0.5m before it extends fully across the roadway.

**Mining context**

It is important to note that these are elastic analyses, such that the numerical model does not redistribute the stresses after failure. It is assessed that more sophisticated models are not required for practical roof support design. Seedsman (2009) presented a conceptual model for stress redistributions in the immediate roof for conditions where the rock strength index would lead to the onset of large-scale compressive failure (Figure 5). In this model, horizontal stresses are not present within the immediate roof – these have been transferred to the stress arch as seen in the data reported by Mark et al. (2007) and the earlier work of Gale and Mathews (1992). This redistribution of stresses is

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1 In this paper Examine2D is used for simplicity to demonstrate concepts. For specific design, it is possible to use the same elastic parameters but should include different rock strength layers as appropriate in other numerical techniques.
not recognised in AMCMRR (Analytical Method – Coal Mine Roof Rating, Colwell and Frith 2010) which relies on the maintenance of elevated stress in the immediate roof line.

**Figure 3** - Extent of compressive failure on development and at the maingate corner (depth = 300m, UCS = 45 MPa, frictional limit = 33°, K=1.5, reducing to 1.4 at maingate)

**Figure 4** - Height of the zone with a strength factor less than 1.0 as a function of the roof strength index and the ratio of the horizontal to vertical stress

**Figure 5** - Stress redistribution about softened zones (Mark, et al., 2007; Gale and Mathews, 1992)
DESTRESSED ZONE AFTER TENSILE FAILURE

For the design of any ground control strategy at the roadway scale, the default assumption should be that the roof has zero tensile strength as a result of the presence of joints and bedding (Figure 1). This is totally consistent with the recommendations not to use the Generalised Hoek Brown criterion (Hoek and Brown, 1997) if there are only two discontinuity sets (joints and bedding). An important implication for numerical modellers is the need to use a tensile cut-off in a Mohr Coulomb criterion if continuum behaviour is assumed.

It is of interest to examine the height of failure (tensile or compressive) above the centreline of a typical development roadway for a rock mass with zero tensile strength and a Roof Strength Index (RSI) of 3.2 (Figure 6). Equally high zones may develop at very low horizontal to vertical stress ratios as at high stress ratios.

Figure 6 - Height of failure (strength factor = 1.0) varies with the ratio of the horizontal to vertical stress

Horizontal tensile stresses

If zero tensile strength is assumed, the extent of any tensile failure is the same as the zone of tensile stress. The discontinuity set of interest for roof collapse are the joints that are normal to the bedding and hence effectively vertical. These joints will contribute to roof collapse if the horizontal stress is either zero or tensile. Whether shear failure along joints leads to roof collapse will depend on the kinematics of the joint-bounded blocks which is related to the amount of dilation of the joints. Small dilations may allow block interaction and the formation of a voussoir beam. Larger dilations may mean blocks simply fall. Figure 7 shows the distribution of tensile horizontal stress for isotropic and a simple implementation of transversely isotropic ($E_1=E_2, E_1/G=15$) cases and three horizontal to vertical stress ratios. It is noted that the dimensions of the tension zones are independent of the magnitude of the applied stresses, and that the magnitudes of the tensile stress within the zones and the associated dilation increase as the stress magnitudes increase. The height of the tensile zones at the centreline decreases with an increase in stress ratio. The “ears” in $K=0.1$ do not define kinematically acceptable blocks that could collapse.

Mining context

Elastic stress analyses, as well as closed-form solutions of stress around holes (Poulos and Davis, 1974, Brady and Brown, 1985) all indicate a reduction of roof stress as the vertical stress increases compared to the horizontal stress. This is opposite to that proposed by Frith (2000) who invoke the so called Poissons Ratio effect, although the fundamental requirement for absolute zero lateral strain when invoking this effect cannot be satisfied at or near an excavation boundary given that the roadway itself deforms.
The key to anticipating the onset of tensile roof stress is to consider the situation where the vertical stresses are higher than the horizontal stress. The compilations of international stress data (Hoek and Brown, 1980) and Australian coal measures stress data (Nemcik et al., 2006) suggest that this is relatively rare. However in the coal mining sector, there are a number of specific environments where a low stress ratio should be anticipated as the "base-case":

- thick coal seams where the immediate roof is coal (Seedsman, 2004). Very low horizontal to vertical stress ratios have been uniquely measured in coal and this is possibly related to shrinkage of the coal as it is dewatered by underground mining (gas may be of secondary importance). It is noted that lower stress ratios may result as the joint spacing in the coal reduces. A consequence is the need to be particularly vigilant when forming wide roadways with a coal roof.

- undermining of pillars in multiple seam operations. Transverse isotropy induced by bedding appears to result in a columnisation of the vertical stress (Suchowerska et al., 2013) and this is consistent with the author’s observations and simple analyses (Figure 8). The spread of vertical stresses implied by the simple Boussinesq models may not be appropriate for coal mine pillars.

- tailgates. The impact of yielding pillars and floor failure is discussed by Diederichs and Kaiser, 1999). Based on the author’s observations and an interpretation of the ALTS method (Colwell,
1998), on the other side of an adequately-sized chain pillar, the stress reduction associated with the adjacent longwall is not sufficient to lead to adverse stress ratios and the onset of tensile relaxation in the tailgate. Adequately sized in this context is one that is behaving elastically. If the pillar is designed to yield, then the additional vertical deflections in the pillar will lead to tangential relaxation in the roof line. This is considered to be the fundamental mechanism behind ALTS: the pillar strength equation is a criterion and roofs with low CMRR are those that are more closely bedded, more closely jointed and hence more susceptible to tensile failure. A bearing failure of weak layers in the floor of a pillar system may lead to sufficient vertical deflection to mobilise this mechanism as well.

IDENTIFYING A SPANNING UNIT

In this case the key question to be asked is “What defines a spanning unit?”. Voussoir beam theory presumes jointed rock and so removes the theoretical difficulty in applying elastic beam equations to a no-tension material. There are several solutions to the statically indeterminate analytical formulation of a voussoir beam and the method of Sofianos and Kapensis (1998), with the low value for the fractional loaded section (n) has been found to be appropriate. Although not strictly rigorous, it is possible to examine the effect of an axial stress applied to a voussoir beam by reducing the UCS value without changing the modulus. The impact of an applied axial loading on the required beam thickness is not significant until the applied stress is about 80% of the UCS (Figure 9).

An interesting aspect of voussoir beam analysis is the recognition that the deflection of a beam at the onset of compressive failure is independent of its span or its thickness, but is a function of the UCS (and by implication its modulus) and the surcharge carried (Figure 10). This may be of value when developing Trigger Action Response Plans (TARPS) based on roof deformations.

![Figure 9 - The sensitivity of the required thickness of a voussoir beam with a 5 m span to an applied axial stress](image)

![Figure 10 - Centreline deflection (mm) of a voussoir beam at the onset of failure (E=250 UCS)](image)
Mining context

Inspection of Figure 9 suggests that the thickness of a spanning unit needs to be in the order of 300 mm. As defined, the spanning unit in this context is similar to the minimum of 300 mm for the “strong bed adjustment” referenced in the CMRR system and the requirements of the US regulations - 30 CFR 75.204(f)(1) which states that “roof bolts that provide support by suspending the roof from overlying stronger strata shall be long enough to anchor at least 12 inches into the stronger strata”.

A note of warning is important: a high degree of confidence in the sedimentary geology of the strong bed is required or it will be necessary to commit to reinforce potential layers within the strong bed. The base of coal measure units such as sandstones and conglomerates may be locally thinly bedded, unlike limestones that often form the strong beds in the US mines.

SPECIFYING THE SUSPENSION PATTERN

In the circumstances outlined above and if there is a need to keep the roadway open for access or egress, then a suspension design is required. The design model is simple and involves the dead weight of volume of rock or coal with no imposed horizontal stresses. Some horizontals stress may be induced by subsequent block rotations within the fractured volume but these cannot be relied on in a robust support design. It is noted that these assumptions rule out the use of ground reaction curves – the model invokes gravity dead weight so there is no arching of loads or stresses.

The parameters to be considered are the height and weight of the suspended block, the length of the anchorage, the capacity of the tendon, the location of the tendons, the “sling” between tendons, the mode of installation, and the resultant improvement in the stability of the roof.

Height and weight to be suspended

The density of the material to be suspended is usually well known (1.4 - 1.6 t/m³ for coal, 2.5 t/m³ for rock) so the design quickly focussed on the height and shape of the collapse volume. For compressive failure the height (H_max) can come from an analysis similar to Figure 4 and the shape can be assumed to be a parabola (cross sectional area = 2/3 base*height). For tensile failure, the height can come from analyses like Figure 7 with the shape assumed to be a rectangle.

Tendon capacity

Assuming the anchorage is adequately designed (see below), the capacity to use is either the tensile strength of the tendon or the collar assemblage if the tendon is point-anchored. There is an important question about whether the tendons will be equally loaded. This is likely to be the case for the spans being considered but it is good practice to have some ductility in the cables so that there is an increased ability to redistribute loads evenly between them.

Length

The anchorage can be estimated using the methods of Littlejohn (1993), viz:

\[ L_a = \frac{M^2 \cdot 2 \cdot \cos \Phi \cdot T}{(\pi \cdot d \cdot \text{UCS} \cdot (1 - \sin \Phi))} \]

Where \( L_a \) = required anchorage length, \( T \) = tendon capacity, \( \text{UCS} \) of the rock or grout (MPa) and \( \Phi \) = friction angle, and \( M \) is a confidence factor.

Analyses should be conducted for the cable/grout interface and at the grout/rock interface, with the former considering the maximum diameter of bulbed or nutcaged cables.

Location

The vertical height of a parabolic shaped collapse zone is given by:

\[ L_{vh} = H_{max} - 4x^2 \cdot \frac{H_{max}}{W^2} \]

Where \( L_{vh} \) = vertical height, \( W \) = roadway width, and \( x \) is distance from the centreline. The algebra for angled cables is more complex and it may be better to determine the length with a scale drawing.

For compressive failure, an optimum design is obtained by locating the anchorages angled away from centreline to reduce the exposure of the design to uncertainties in the height of failure that could lead to the anchorage being over-ridden (there may be little if any deformation warning of this). The collars should not be located within 1 m of the rib corner as the extent of damage may be too great to allow the
transfer of loads from the tendon to the roof. Routinely pairs of cables are used spaced 1/3 and 2/3 across a development roadway and angled at no more than about 15\(^\circ\) from the vertical.

For tensile failures and suspension from a spanning unit, evenly spaced tendons are probably the best.

**Inclined tendons**

Hutchinson and Diederichs (1996) present a discussion on the capacity of angled tendons based on a series of laboratory tests of plain, nutcaged and bulbed cables. The results for the plain cable were anomalous and are the result of the cables slipping at the cable/grout interface. For the other cable types, where the effective diameter of the cable is much greater, the experimental design was valid, and the results show no significant difference in the peak loads. The decrease in capacity proposed by Frith (2011) and Frith and Colwell (2011) by referencing structural engineering is not observed and this has been attributed by many researchers to the yielding of the grout/rock ahead of the tendon (Holmberg and Stille, 1992).

**Experimental design**

The angled cables show greater deformation which is advantageous for a roadway suspension system in that it allows a greater opportunity to redistribute the loads between the cables and also provides a level of deformation that can be used in a TARP.

**Stiffness - Post installation shear**

The conventional view is that a stiff installation is required to limit the development of the height of softening. This view may have developed as a consequence of the way in which the empirical data has been presented (Figure 12). It is equally valid to plot the height of softening as the independent variable (abscissa) and the roof deflection as the dependent variable or ordinate. With this presentation an alternative interpretation is possible - that roof deflection is a response to failure in the roof, possibly in response to the dilation that occurs at failure. This interpretation leads to a conclusion that it is futile to seek to limit deflections given the very high stresses that drive failure. In rock massess, the ground stresses can arch above a roadway so the ground control approach is to seek to control unacceptable deflections until the stresses are redirected into the arch.
This interaction between deflection and stress redistribution is the basis of the ground reaction concept, and the classic discussion of this concept involves the early installation of stiff support. Referring back to Figure 3, installation of roof support at the development face or even in front of the longwall abutment front is “early” in the context of the stress changes that are about to impact the maingate corner. A stiff installation will not be able to prevent the onset of failure in the roof and more importantly may not be able to sustain the deformation. Test data on the shear stiffness of fully-grouted and point-anchored cables has not been found, but data on double-embedment tension tests (Clifford, et al., 2001) has a fully grouted cable failing at 7 mm to 8 mm of opening of a joint compared to in excess of 40 mm for a point anchored cable (Figure 13).

![Figure 12](image1.png)

**Figure 12** - Replotting of height of softening and roof deflection data changes interpretation of empirical data (after Gale, et al., 1992)

![Figure 13](image2.png)

**Figure 13** - Comparison of double embedment test on a grouted cable (after Clifford, et al., 2001) and calculated deformation of a 4m free length of cable

Under simple direct loading, a point-anchored cable has lower capacity than a fully grouted cable and this is the result of yielding and failure of the collar assembly. To address the stiffness concerns while still being able to mobilise the full tensile capacity of the cable it should be possible to fully grout the tendon but to decouple the middle section. As well as increasing the survivability of the tendon, the roof deflections before failure will be greater and this will allow a more robust deflection trigger for any TARP. However there would be no deformation at the collar assembly itself.
Mesh/sling

If a broken de-stressed rock mass is assumed, there may be zones within the immediate roof that are not directly restrained by the tendons (Figure 14). If these zones were to collapse, the roof would unravel around the long tendons leading to total collapse. Depending on the load distribution angle beneath the collar and the spacing of the tendons, loads of up to 3 to 5 tonnes can be reasonably expected and these need to be retained by the roof mesh, W strap or a truss. Laboratory tests on standard mesh panels (Thompson, 2004) indicate a capacity of 2 to 3.5 tonnes depending on the bolt spacing. It is concluded that any suspension strategy must also include specific measures to ensure that collapse of the roof between the tendons does not lead to unravelling of the roof. Closer spaced, lower capacity tendons may be preferable despite the greater installation time.

![Figure 14](image)

**Figure 14 - Zones which need to be supported to transfer loads to the tendons and the capacity of typical mesh panels**

Design factors

The term “Factor of Safety” is still commonly used although its misuse and incorrect application is now being recognised. At its simplest arithmetic level, a factor of safety is the ratio of the estimated strength to the estimated load. In the real world, there may be uncertainty in the estimates. A design is then considered acceptable if the likelihood of the strength being exceeded by the load is acceptable given the consequences. In some industries, for example aerospace, the factors of safety can be relatively low compared to geotechnical designs because the uncertainties in the strength and load estimates are low even though the consequences of failure are high - high factors of safety could lead to planes that are too heavy to fly. In geotechnical designs, the factors may be high because of the difficulties in estimating strengths. From a design perspective, confidence in these estimates determines the factor of safety that is acceptable.

With this use of the term in geotechnical engineering, factors of safety should not be pre-determined although it is recognised that mining regulation is tending to move that way. It is meaningless to apply factors of safety from other industries, for example crane cables. If a factor of safety value is mandated on a mine geotechnical engineer, it should be used to determine the required site investigation to reduce the uncertainties to the implied level.

When finalising the design, the mine geotechnical engineer should consider:

- Height factor – how reliable is the estimate of height? How reliable are the estimates of rock strength and the in-situ stresses, what have back analyses suggested, is there a precedent at your mine or close by? Lower bound strength values and any available stress data should be used when if accepting the heights given by Figure 4. This parameter leads directly to the loads to be suspended, with little uncertainty regarding width or density.
- Loading factor - will the tendons be loaded equally? Given the general industry expectations regarding “Factors of Safety” it is may be appropriate wise to add about 20% to calculated loads.
- Survivability factor - could they fail due to deformation incompatibility? An arbitrary 50% reduction in the mobilised strength of fully grouted cables can be considered used but it should be recognised that this is without technical justification – clearly more research is required.
• Anchorage length – The civil engineering literature suggests a doubling of the calculated length to address uncertainties about installation quality, values between 1.25 and 1.5 have been used.

CONCLUSIONS

Correctly designed, suspension is a very robust roof support design approach. There are some important subtleties in the design which conflict with some of the incorrect concepts that have been published in the last decade. A suspension design accepts that it is not possible to prevent tensile or compressive failure of the roof and that what is needed is a support regime that can survive the onset of failure and then prevent the subsequent gravity-driven collapse. A corollary of this is that deformations are inevitable and can be utilised in a TARP process, recognising that the triggers will be related to deformations of flexible systems. Suspension systems are installed proactively, in that they anticipate the onset of failure and are ready to prevent collapse. In better ground where compressive or tensile failure is not likely, it may be efficient and economic to prevent the deterioration of a bedded roof by reinforcing against shear.

REFERENCES


NEW FRACTURE MODEL FOR THE PROGRESSIVE FAILURE OF ROCK SLOPES

Gaetano Venticinque and Jan Nemcik

ABSTRACT: An improvement to previously developed constitutive FISH User-Defined-Model subroutine by Venticinque (2013) is demonstrated here to simulate the initiation and progressive propagation of fractures through rock structures. This model is based on the amalgamating failure and fracture mechanics theory applied to the finite difference FLAC code. The prior validation of fracture propagation in isotropic rock has been modified to simulate fracture propagation in anisotropic rock. It is shown that the model is capable to accurately simulate fracture distributions in both isotropic and anisotropic rock mass. Furthermore, application of the model to study rock slope stability highlights several characteristics relevant to the progressive failure process of hard rock dry wall slopes. Moreover, the model introduces new potential insight towards the effectiveness of rock and cable bolt supports. This work contributes towards improving safety in mines through an increased understanding of key fracture and progressive failure characteristics within geological structures.

INTRODUCTION

In open cut mines the failure of high wall rock slopes is regarded as a non-simultaneous occurrence. Instead massive shear failure events are recognised as being a progressive consequence of local fracture propagation from one area to another within the slope, (Bertoldi, 1988). The knowledge of when fractures initiate and how they may propagate through a rock mass is fundamental to the safety and efficiency of stable mine design. During simulation it is often useful to determine if and more importantly how a structure will fail. This paper presents an example to further demonstrate the capability of the recently developed model by Venticinque (2013).

The fracture model is a constitutive FISH subroutine driving the FLAC 2D geotechnical software by Itasca (2005). It offers independent simulation of rock fracture initiation and propagation behaviour. This model has been previously verified for isotropic rocks where it was proved to offer more realistic solution of the brittle fracture propagation and post failure response of rock as illustrated in Figure 1. The modelled fractures exactly matched the fracture geometry that developed in the isotropic marble rock tested in the laboratory. It also shows the difference between the brittle fracture propagation and the unrealistic plastic-like failure akin to rock at critical stress modelled without the FISH subroutine.

Furthermore in its current form, the model incorporates all three combinations of fractures described by: Mode I tensile, Mixed Mode I-II and Mode II pure shear. This was verified through simulated application over the entire brittle-to-ductile transitional failure range of rock and shown in Table 1.

Figure 1 - Comparison of current existing and newly developed modelled failure against real isotropic marble rock tested in the laboratory (Venticinque, et al., 2013)

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