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# Preinstalled Cable Bolts in Longwall Installation Roads

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# PREINSTALLED CABLE BOLTS IN LONGWALL INSTALLATION ROADS

Ross Seedsman

**ABSTRACT:** Long tendon support is typically installed in the first pass of an installation roadway meaning that it is pre-installed as the full roadway width is developed. The recognition of additional compressive failure in a rock mass as the roadway is widened results in a set of challenges for the design of the pre-installed long tendons support. Cable bolts cannot prevent the onset or progression of failure and hence must be able to survive the associated deformations. After failure there are stress reductions in the immediate roof that may cause the cables to debond. In certain geological conditions pre-installed, pre-tensioned, fully grouted cables may be too stiff and could fail due to their inability to accommodate deformations of the excavation. A case study is presented and analysed using some readily available analytical tools that capture what are considered as the key aspect of the behaviour model.

## INTRODUCTION

Sedimentary rocks are often referred to as soft rocks to contrast them with igneous and metamorphic rocks. As well as being geotechnically soft (low modulus) they have lower strengths and this means that at comparable depths with respect to hard rock mines (where the dominant roof collapse mode is gravity fall of joint bounded wedges), soft rocks can undergo compressive failure. As identified in the logical framework (Seedsman, 2012), there can be several situations in longwall coal mines where compressive failure is possible – at the development face in low strength/high stress environments, at the maingate corner and in “super stress notches”.

Hutchinson and Diederichs (1996) provide a detailed coverage of all aspects of cable bolting. In the section on mechanistic design, they present discussions on how cables may interact with the rock mass during mining. They comment (page 254): “*cablebolts are unlikely to arrest the onset of rock failure under high stress, and may do little to alter the progression of such failure into the rockmass.... In highly plastic (deformable) rockmasses under high stress, it is also unlikely that cables will be effective in arresting the progression of failure. In addition, in these environments, the induced displacements may be too great for the system to handle and cable strand rupture may be inevitable in pre-installed systems.*”

The ground reaction curve concept is well established in mining rock mechanics (Brady and Brown, 1985). The concept is to allow the roof to move to some degree so as to minimise the required support density but not sufficient movement to allow a large failure zone to develop. It allows (requires?) roof deformation to redirect stresses away from the roadway. The concept requires consideration of both the strength and strain capacity of the ground support, with the concern that strong, stiff, but brittle support members may break before their load bearing capacity can be mobilised in the roof. Over the last decade, there has been a trend in the Australian underground coal industry to install fully grouted pre-tensioned cables as close as possible to the face so as to limit roof deflection with the view that this reduces the height of softening. This trend would appear to be at odds with the ground reaction curve concept.

Once compressive failure develops, the ground stresses are redistributed to elsewhere in the rock mass. This means that the stresses within the failure zone itself reduce. Hutchinson and Diederichs (1996) also discuss stress shadowing and relaxation effects (page 255): “*Zones of relaxation pose additional hazard for cablebolting. ... stress decreases across a cable array can seriously impair the bond strength of plain strand cablebolts. Rockmass stiffness is also dependent on confinement in fractured rockmasses and decreases with relaxation. ....It is for this reason that plating and the use of modified strand cablebolts are recommended in fractured-destressed rock*”

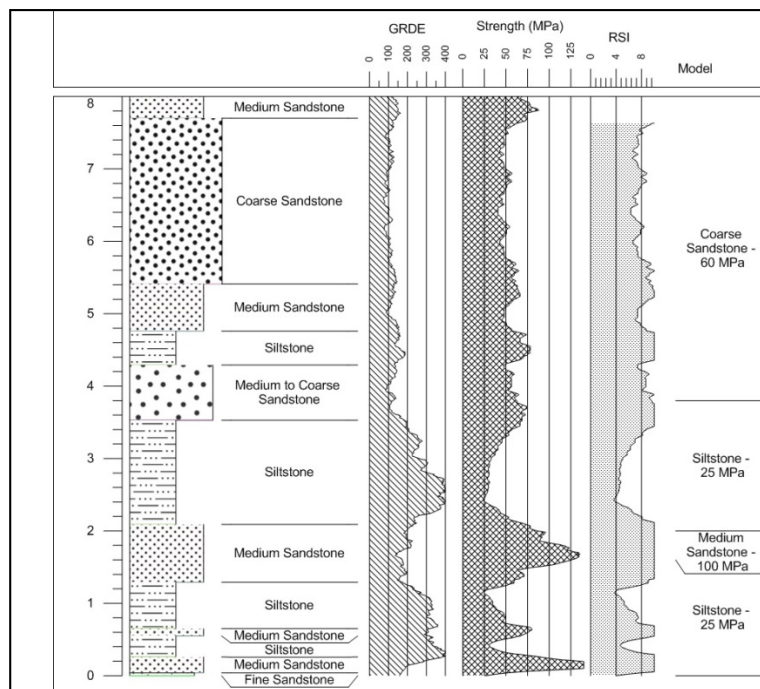
Longwall installation roadways can be required to be excavated in low strength/high stress environments. In a two-pass installation, say a 5.2 m wide roadway stripped to 9 m wide, long tendons are typically installed in the first pass and are hence pre-installed with respect to the second pass. Additional compressive failure as the second pass is extracted could induce displacements that could

cause cable rupture. The use of pretensioned fully grouted-cables could cause premature failure if the magnitudes of the deformations in the roof are too large. Subsequent destressing of the zone of compressive failure could lead to debonding of the cable.

This paper presents a summary of large and unexpected movements in a recent installation roadway and considers what may have happened in the light of these comments. As is common with mine operational problems there is inadequate data on the geological and geotechnical conditions at the site. It is acknowledged that there may be other explanations to what occurred.

**CASE STUDY**

A summary of the geology and geophysical logs of a borehole within 150 m of the site is presented in Figure 1. The siltstones have a unit rating of 42 and the medium grained sandstone a unit rating of 59 giving a Coal Mines Roof Rating (CMRR) of 54 without a strong bed adjustment. Based on the sonic-derived strengths, the Roof Strength Index (RSI) of the siltstones is in the order of 4.0 for a depth of 250 m. The mine is known to have a relatively high horizontal to vertical stress ratio (possibly in the order of 1.8 to 2.0) so a stress-relieving roadway was used.



**Figure 1: Roof geology and simplified geotechnical model**

The installed roof support in the 5.2 m wide first pass roadway was 6 of 2.1m X grade bolts every metre with 3 of 8.0 m long pre-tensioned fully-grouted 630 kN cables subsequently installed every two metres. The second pass to 9.0 m was supported with an additional 5 of 2.1 m X grade bolts every metre and one additional cable every two metres. The primary roof density index (Thomas, 2010) was 0.8 MN/m and the secondary roof density index was 1.12 MN/m. Based on Colwell and Frith (2013), the Primary Roof Support Rating PRSUP was 81.9 for the first pass and 63.3 for the second pass.

The stress relief roadway encountered some roof/rib guttering and some cables were installed. The first pass roadway was offset by about 14 m and excellent roof conditions were encountered. There was negligible movement recorded on the GEL extensometers on the first pass. Some minor guttering was observed on the outbye rib and indicated an apparent change in the stress direction.

During the second pass the roof extensometers consistently showed movement at both the 1.8 m and 2.5 m horizons and at the 5.5 m horizon in some locations. This movement was accompanied with some stress guttering on the outbye rib and some minor distress to the roof line between the first and second cables on the inbye side. There was no evidence of loading on the cable plates and only a few of the bolt plates. Operators and company mining engineers reported loud sharp noises that they related to strand rupture. In a previous installation roadway, broken cables had been recovered from the fall debris.

The GEL extensometers were installed no more than 1.5 m from the to-be-stripped rib and within 1.4 m of a cable installation. After 24 hours the movement at the roof line was in the range of 40 mm to 55 mm, at the 1.8 m horizon the movement was between 14 mm to 28 mm, and these then increased to about 28 mm to 42 mm in the next 24 hours. The creep rates after 48 hours were 10 mm/day and 5 mm/day respectively. The surge measured at the roof line would have been in excess of 100 mm.

The decision to re-support was based primarily on the movements at the 1.8 m and 2.5 m horizons, reference to published guidelines, the interpreted breakage of support based on the noises in the roof given the recovery of broken cables in the collapse of an earlier installation roadway.

## NUMERICAL SIMULATION

There is not a complete set of geological or geotechnical data so there are major constraints to the selection of geotechnical parameters for the numerical simulations. Consistent with the approach of Lambe (1973), there may be greater confidence in the interpretation of the outcomes if the sophistication of the analyses matches the available data. In this way Phase2 finite element analyses were used to examine the impact of the geological layering and to “calibrate” a simpler Examine2D model that allows ready sensitivity/parametric assessments. The Examine2D model considered just the siltstone with a revised UCS to account for the sandstone layer. The siltstone was modelled with brittle parameters and transverse isotropy (Seedsman, 2013). The Examine2D model can only be interpreted in terms of the roof conditions and possibly the floor (but not the ribs as the coal was not modelled).

An implication of the failure zones identified in the stress models will be a redirection of the stress field. This was simulated in the simple elastic stress models by forming a new excavation shape for the analysis of the next stress change. The same redirection of stresses allows the application of published voussoir beam methods to the sandstone layer as the layer will not have imposed stresses onto it.

It is acknowledged that the Examine2D model is very much simplified compared to the geotechnical conditions but is somewhat compatible with the available data. It is assessed to reflect an appropriate behaviour model for this installation road.

### Stress relieving roadway

The Phase2 model showed compressive failure in the siltstones above and below medium grained sandstone (Figure 2). The calibrated Examine 2D model has a compressive failure zone extending 3.9 m into the roof. Simulating this failure zone as a new excavation, the model then indicates that there would have been substantial relaxation of the horizontal stresses at the location of the first pass roadway – the predicted horizontal stresses at the roof line may have been in the order of 7 MPa to 8 MPa (left side of Figure ). It is noted that is consistent with some of the observations. Stress relief at this distance is also consistent with reports in the literature from the Southern Coalfield of New South Wales.

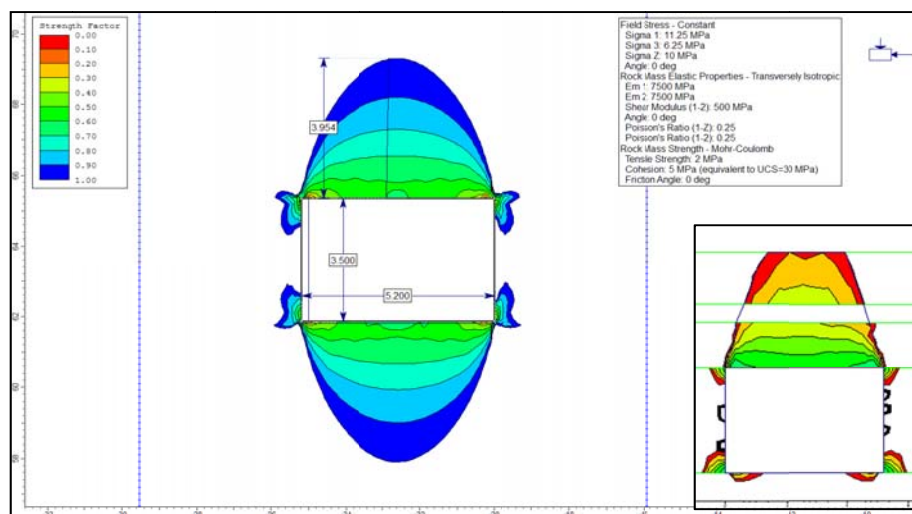
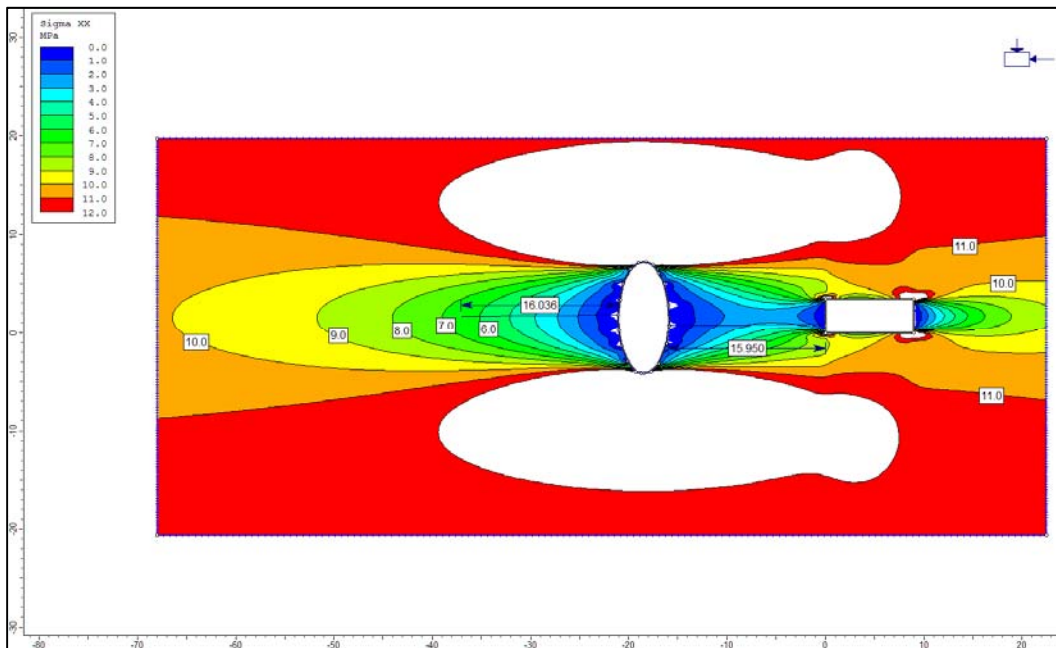


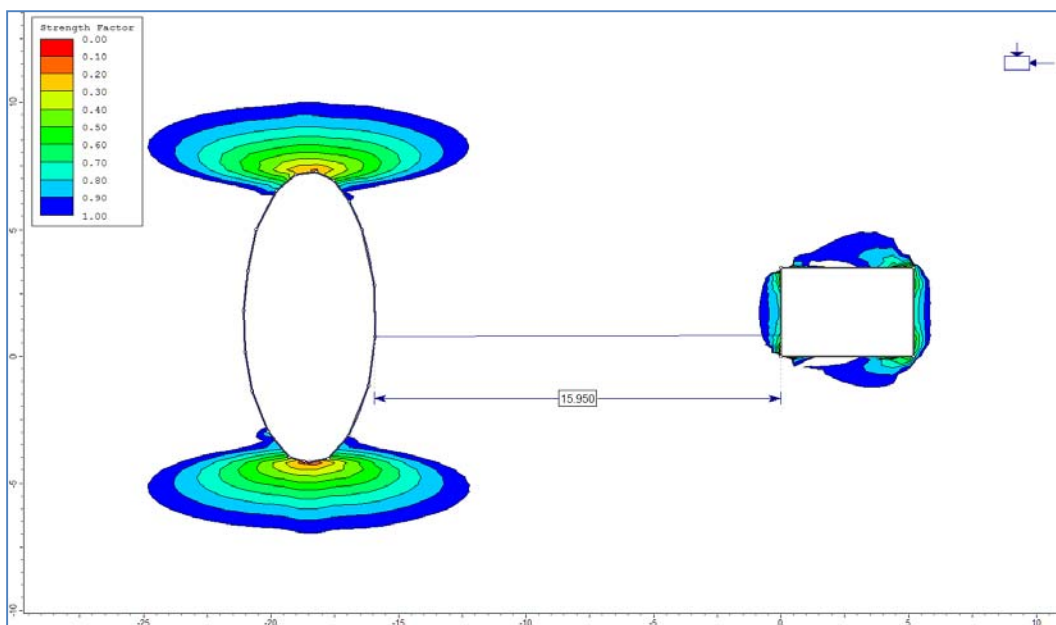
Figure 2: Failure zone (strength factor < 1.0) developed around the stress relieving roadway (inset – Phase2 model)

**First pass**

By comparing Figure with Figure , it can be seen that there is a significant reduction in the extent of compressive failure and it is now less than 2 m above the roof line. In reality, this would be seen as failure in the siltstones below the sandstone layer and such material could have been readily suspended from the sandstone layer with the 2.1 m long bolts. The model suggests that there may have been some stress guttering developed on the outbye rib as was observed.



**Figure 3: Reduction in horizontal stresses associated with the stress relieving roadway (note simulated excavation)**



**Figure 4: Failure zone developed above the first pass roadway in the shadow of the stress-relieving roadway**

**Second pass**

In Figure , the shape of the first pass roadway has been modified to account for the compressive failure modelled in the first pass (Figure ). There is additional compressive failure in the roof, extending 5.8 m above the original roof line (to the base of the coarse sandstone?). It is noteworthy that there is very minor

stress guttering at the outbye roof/rib corner. The associated elastic strains (those before failure) in the roof at this time are only in the order of 1 mm/m to 1.5 mm/m.

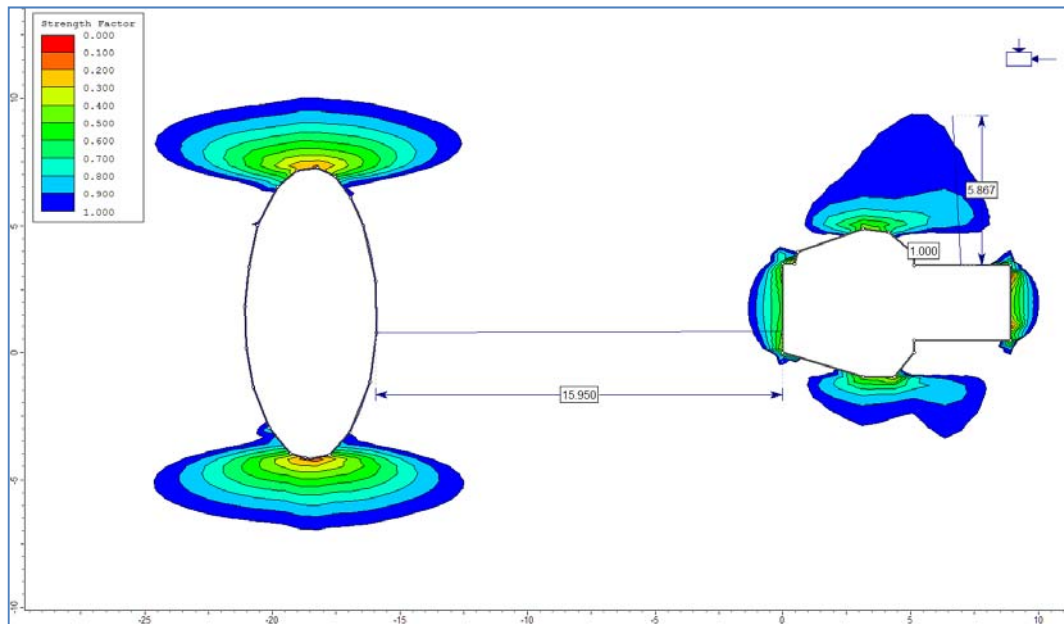


Figure 5: Failure zone developed as the roadway is widened to 9 m

### BACK ANALYSIS

The surge certainly exceeded the published databases for acceptable outcomes (Colwell and Frith, 2013). The databases are unpublished and it may be the definition of an acceptable surge is based on very conservative assessment of approach to the required serviceability of installation roadways. It is noted that the decision to resupport was not based on the surge value alone. The decision to resupport the roadway was made primarily on the basis of the roof movements at 1.8 m and 2.5 m as it was acknowledge that there was some delamination in the immediate roof.

Extensometers and borescopes confirmed that the highest movement is located at the base of the coarse sandstone. By reference to Figure , a possible collapse mass assuming a general parabolic shape is  $0.67 * 5.8 * 9 * 2.5 \text{ t/m} = 87 \text{ t/m}$ . The installed capacity of the cables was 128 t/m. If the load was evenly distributed between the cables, each cable would be carrying 43 tonnes, representing a “factor of safety” of 1.5.

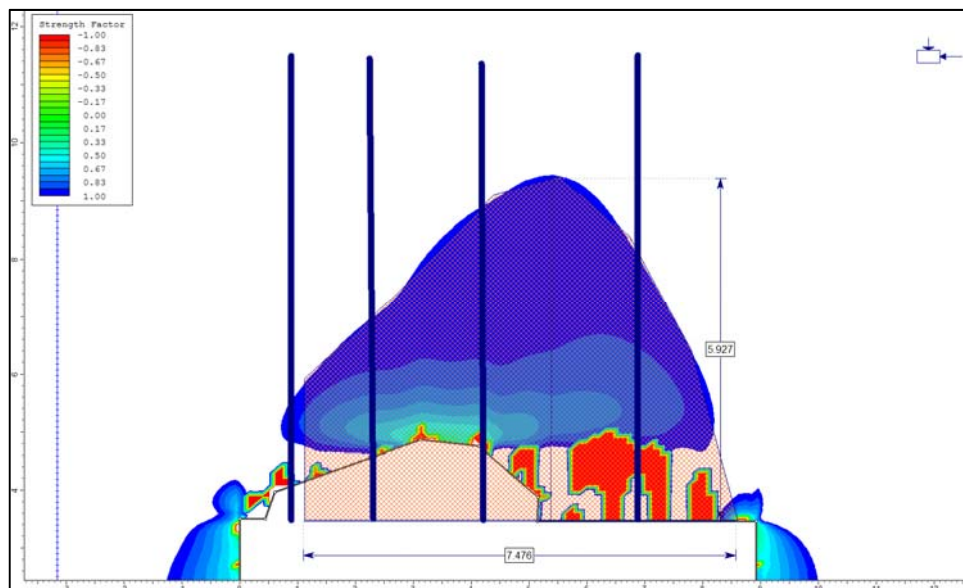


Figure 6: Close up of the modelled failure zone above the 9 m wide roadway

So four questions need to be answered:

1. Why did the immediate roof skin move so much?
2. Why did the 1.8 m and 2.5 m anchors move?
3. Why does the calculated factor of safety based on suspension of a possible detached block not reflect the unsatisfactory outcome?

In the following discussion, it will be assumed that the materials supplied to the mine were to specification and that they were installed by the experience and well trained workforce.

### **Adequacy of the geological model**

Subsequent roof drilling has identified highly variable lithologies along the length of the installation roadway. It is possible that the geological and geotechnical properties of the borehole were not representative of the bulk of the installation roadway.

### **Adequacy of the behaviour model**

The design was based on precedent and practice at the mine and hence a specific behaviour model was not proposed. Precedent/practice is an acceptable design approach if it can be demonstrated that conditions have not changed. The drilling referred to above highlighted the weakness of this assumption.

The a-posteriori application of a simple suspension model certainly needs more consideration. Frith and Colwell (2011) have stated that suspension is fundamentally flawed, while Seedsman (2014) has argued that it continues to have validity once the survivability of stiff cables in a compressive failure regime is considered. The following discussion is based on a suspension support model and the logical framework.

The empirical Analysis and Design of Faceroad Roof Support (ADFRS) model was not used by the mine. The key tenet of ADFRS that "*roof softening can be reduced by limiting roof displacement, this then increasing overall roof stability*" is not accepted. Instead it is considered that roof displacement beyond the elastic regime is a consequence of failure, be it opening of angled structures, shear along joints, and compressive failure of the rock mass or delamination of thin beams. The roof stress model invoked in ADFRS is also not supported by mine instrumentations (Seedsman 2014). Given the extreme divergence of views, it would be inappropriate to discuss the possible application of ADFRS to this case study.

The onset of compressive failure requires the application of brittle behaviour proposed by Martin, Kaiser and McCreath (1999) and implemented in soft rock by the method of Seedsman (2014).

### **Compressive failure and strand rupture**

Most of the compressive failure on the first pass will have developed prior to the installation of the cable bolts. At the time of installation both the vertical and horizontal stresses in immediate roof will have already relaxed. The combination of the bolts and the cables will have resulted in negligible roof movement – at the stage the installed capacity would have been 366 tonnes compared to the weight of say 2 m layer of siltstone which would have been 27.5 tonnes.

Now consider the failure zone above the second pass into which the cables were pre-installed. According to the simple numerical model the elastic deviatoric stress in this zone are in the order of 10 MPa to 20 MPa and this is sufficient to cause failure. Consistent with Hutchinson and Diederichs (1996), the localised deformations within the failed rock may have been sufficient to cause strand rupture.

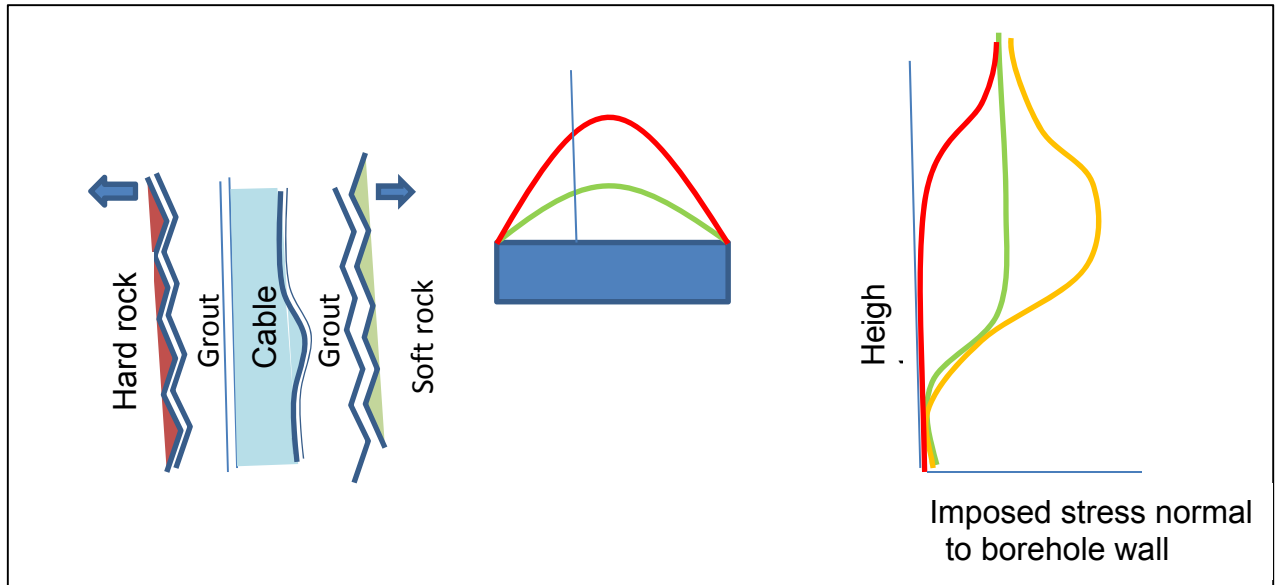
### **Destressing and relaxation**

During the second pass and after the onset of compressive failure in the roof there would have been a substantial relaxation of the stresses; at a simple level the elastic deviatoric stresses would have reduced to zero. At the same time the broken rock mass would now have a much reduced deformation modulus. This is not to say there were zero horizontal stresses in the mine roof; the interaction of the broken rock with other blocks and with the support elements would result in some induced body stresses.

The mechanism here is stress reduction allowing the hole diameter to increase and hence cause a reduction in the shear strength of the rock/grout interface through a reduction in mechanical interlock.

If the gap exceeds the asperity height at the interface there can be no stress transfer. There are two interfaces where this can happen – the steel to grout and the grout to rock (Figure ). Continued shear resistance requires a rough interface so the surfaces interlock: hence the adoption of bulbing for the steel/grout interface. At the grout to rock interface the problem is greater in lower modulus (= soft/sedimentary) rocks because the deformation per unit stress change is greater and hence a potentially larger gap. If a deformation modulus of 400 MPa is assumed for broken and sheared mudstone, a 45 mm borehole would have expanded by 1 mm which is well in excess of the likely roughness of the borehole wall especially the low strength and clayey composition of the rocks.

Inspection of Figure 6 indicates that such relaxation could have developed over a minimum length of 2.4 m. If a yield strain of 1% and an ultimate strain of 3.5 % are assumed, such a free length could allow 24 mm to 84 mm of deformation. For a 4 m free length the deformations would be 40 mm to 140 mm.



**Figure 7: Stress relaxation and its impact on anchorages**

It is not known if such debonding developed. Whilst such debonding would be anticipated to transfer loads to the plates (which was not observed) it is important to note that relaxation of stresses does not occur at the roof line during the second pass. This area underwent compressive failure prior to the installation of the cables and hence was already relaxed. There may have been a suitable length of grouted anchorage at the roof line such that loads were not transferred to the plates.

#### **Installation of pre-tensioned fully grouted cables too stiff**

Referring once again to Hutchinson and Diederichs (page 274): *“the cablebolts installed normal to the laminations covering the span area should be designed as stiff reinforcement within the zone of rock equivalent in thickness to a self supporting beam as calculated in this analysis.....Beyond this limit, an optimum cable array should have a more ductile response to allow the beam to deflect a small amount to generate the required compression for stability. Beyond this should be a suitable anchorage length.”*

Currently in the Australian underground coal industry much is made of the supposed advantages of the stiffness inherent in fully-grouted and pre-tensioned cables. Seedsman (2014) has argued that this may be due to the habit of plotting the maximum height of movement (height of softening) as the Y axis against displacement at the roof line as the X axis. This plot leads to an interpretation that the height of softening can be controlled by limiting roof displacement, which is the key tenet of ADFRS. If the data is plotted the other way around, it can be equally valid to assert that roof displacement is the consequence of the dilation associated with rock failure in the roof.

Laboratory tests on cables grouted into steel tubes result in the cables breaking at about 8 mm of opening of a joint in a double embedment pull test using steel tubes (Clifford *et al.*, 2001). It is not known if this test is at all representative of loading conditions in the field. Immediately prior to the onset of compressive failure, it could be argued that the elevated stresses result in compression of the boreholes in the same way that relaxation leads to dilation and this compression could lead to very stiff



anchorages. At the simplest level, the inference would be that the fully grouted cables may have ruptured as a result of the measured bedding dilations which are inferred to be well in excess of 8 mm.

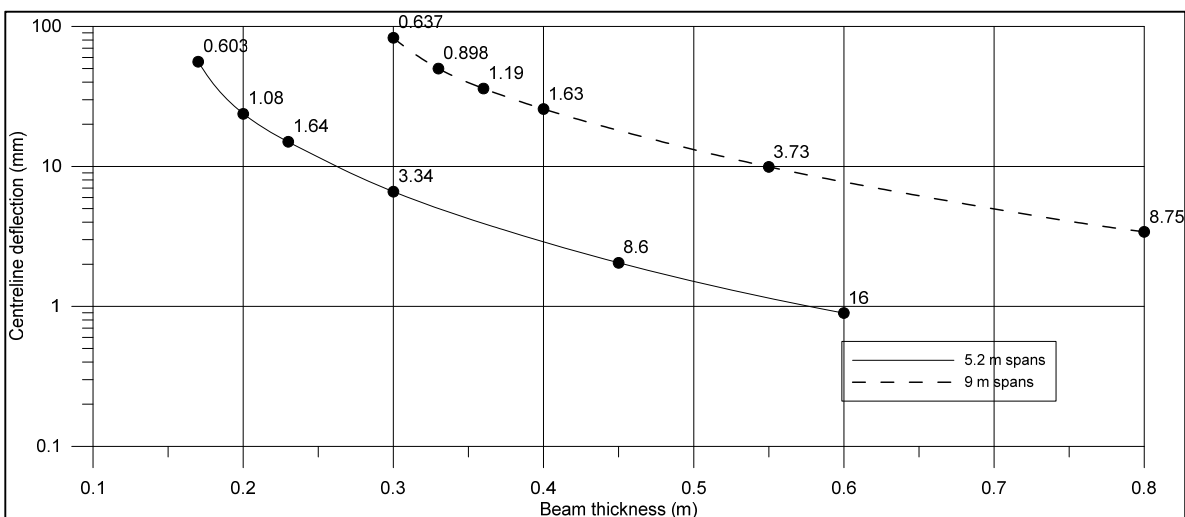
**GROUND CHARACTERISTIC AND SUPPORT REACTION**

The ground characteristic of a gravity-driven collapse is a horizontal line, and hence the ground reaction concept has limited application to suspension designs in the compressive or tensile failure steps of the logical framework (Seedsman, 2012). The concept can be applied if there is some capacity of the rock mass to arch across an excavation and hence transfer stresses. One of the difficulties in applying the ground reaction concept in practice is the difficulty in determining the ground characteristic – numerical methods are required. In the two-pass installation roadway context, numerical analyses are further complicated by the stage formation of the roadway requiring complex three-dimensional considerations. When considering the bolts, the method can only consider a support pressure, expressed as equivalent stress applied to the roof: a support load density as  $\text{kN/m}^2$ . Consequently, the following back analysis has required numerous simplifications.

In the first pass driveage, the roof will respond by relaxing elastically into the roadway. These movements are substantially complete by the time the bolts and tendons are installed. In the following analysis, the elastic movements associated with the second pass will also be ignored as they are likely to be very small.

At the same time as the elastic movements develop, compressive failure will develop in the lower strength materials. *In situ* stresses are transferred away and it will be assumed that the immediate roof performance will be determined by the spanning capacity of the high strength sandstone layer. This layer will need to carry the failed material above as well as that associated with the bolted roof below. This layer may be adequately thick to span the 9 m wide roadway; if not the roof will fully detach and possibly collapse to a height determined by the failure of the low strength mudstone. In this section, these two mechanisms will be considered separately and then combined into a simplified ground characteristic.

Figure presents the results of a voussoir beam analysis giving the relationship between thickness, deflection and stability for 5.2 m and 9 m spans. For a 5.2 m span and assuming a 0.4 m thick layer, the indicated stability is high and the deflection is in the order of 4 mm. Bearing in mind much of this will develop before an extensometer is installed this result is consistent with the measurements underground. For a 9 m span, the same 0.4 m thick layer would be self-supporting and deflect 30 mm while 0.3 m and 0.2 m thick units would not be self-supporting. The analysis indicates that a stable beam would need to be at least 0.34 m thick.



**Figure 8: Deflection and stability of voussoir beams in medium grained sandstones (100 MPa, E= 25 GPa, 5 m failure height)**

A possible ground characteristic can be obtained by altering the density of the material in the voussoir beam. In this way Figure presents the ground characteristic for 9 m spans of the medium grained sandstone as a function of the thickness of the layer. The 0.4 m thick unit is self-supporting (the double

yellow line intersects the horizontal axis) and would not require any additional cables. The 0.3 m and 0.2 m units are not self-supporting.

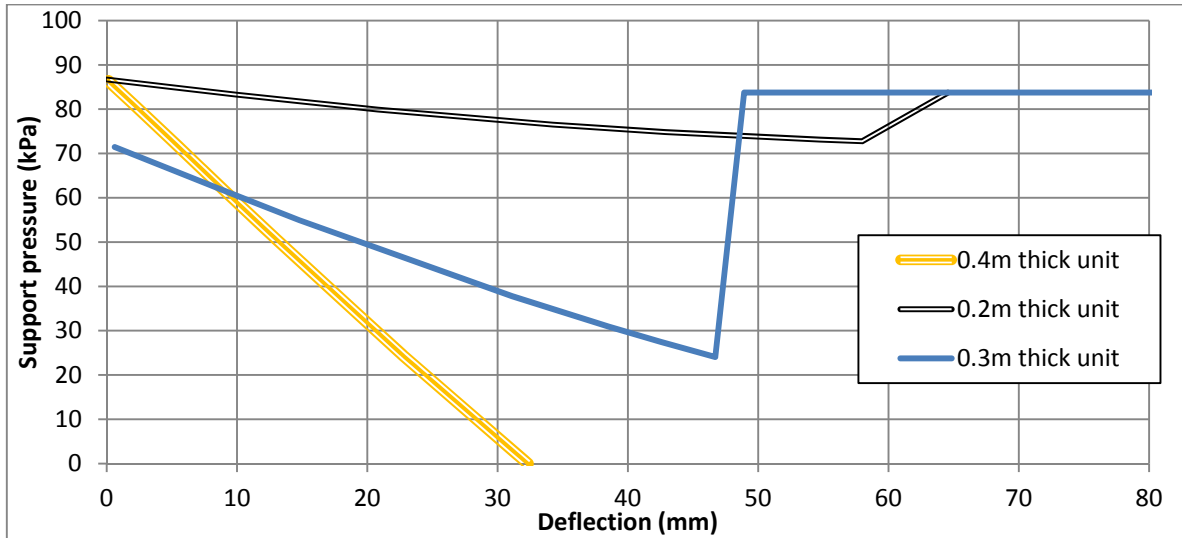


Figure 9: Composite ground characteristic of voussoir beam and collapse of the compressive failure mass

For the 0.3 m and 0.2 m sandstone layers the voussoir beams fail eventually (at 46 mm and 58 mm deflection respectively). When this happens the ground characteristic is determined by the weight of the detached block. For a 5 m high parabolic block over a 9 m span this is equivalent to a horizontal ground characteristic of 84 kN/m<sup>2</sup> as shown in Figure .

When considering pre-installed cables it is necessary to consider their efficiency in dealing with the moments associated with their location with respect to the centre of the 9 m wide roadway: it is possible that they are not equally loaded. Prior to the installation of the cable in the second pass the effective locus of the support reaction is located well inside the line of action for the weight (Figure 10). This results in a 30 % reduction in the efficiency of the installation. When the cable is installed in the second pass the efficiency improves to 93 % assuming the first pass cables have not ruptured.

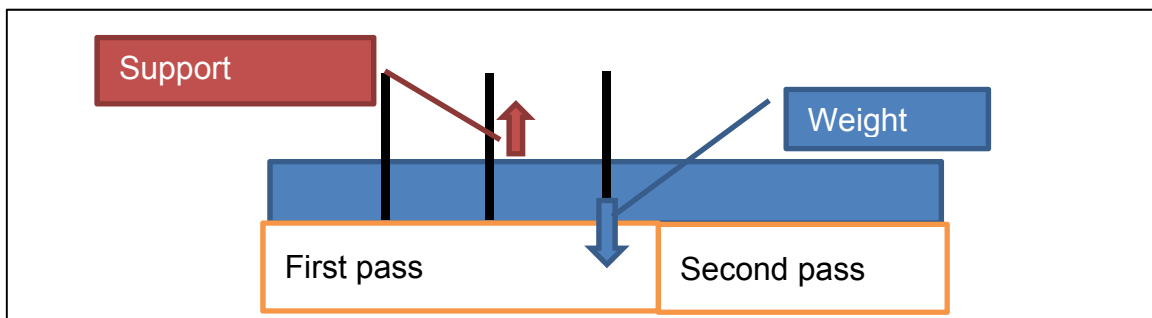


Figure 10: Moment loading of the pre-installed cables

The cables have been modelled assuming a tensile load of 540 kN at 1 % strain and 610 kN at 3.5 % strain. To simulate the laboratory test, a debonded length of a fully grouted cable is assumed to be 0.23 m. The point anchored cable has a debonded length of 3 m. It has been assumed that the roof deflects 30 mm before the final cable is installed in the second pass. The analysis has not considered the impact of pre-tensioning.

The ground characteristic and the support reactions are shown in Figure . The figure suggests that neither cable patterns would be required if the sandstone layer was 400 mm thick. Since the fully-grouted line does not intersect the 200 mm thick ground characteristic, failure of the support design is indicated prior to the installation of the cable in the second pass. The fully-grouted pattern would be considered marginal for a 300 mm thick unit. It is noteworthy that the point-anchored cable line indicates

that its full capacity is mobilised at in excess of 80 mm of movement; importantly the roof would be stabilised at 33 mm of deflection.

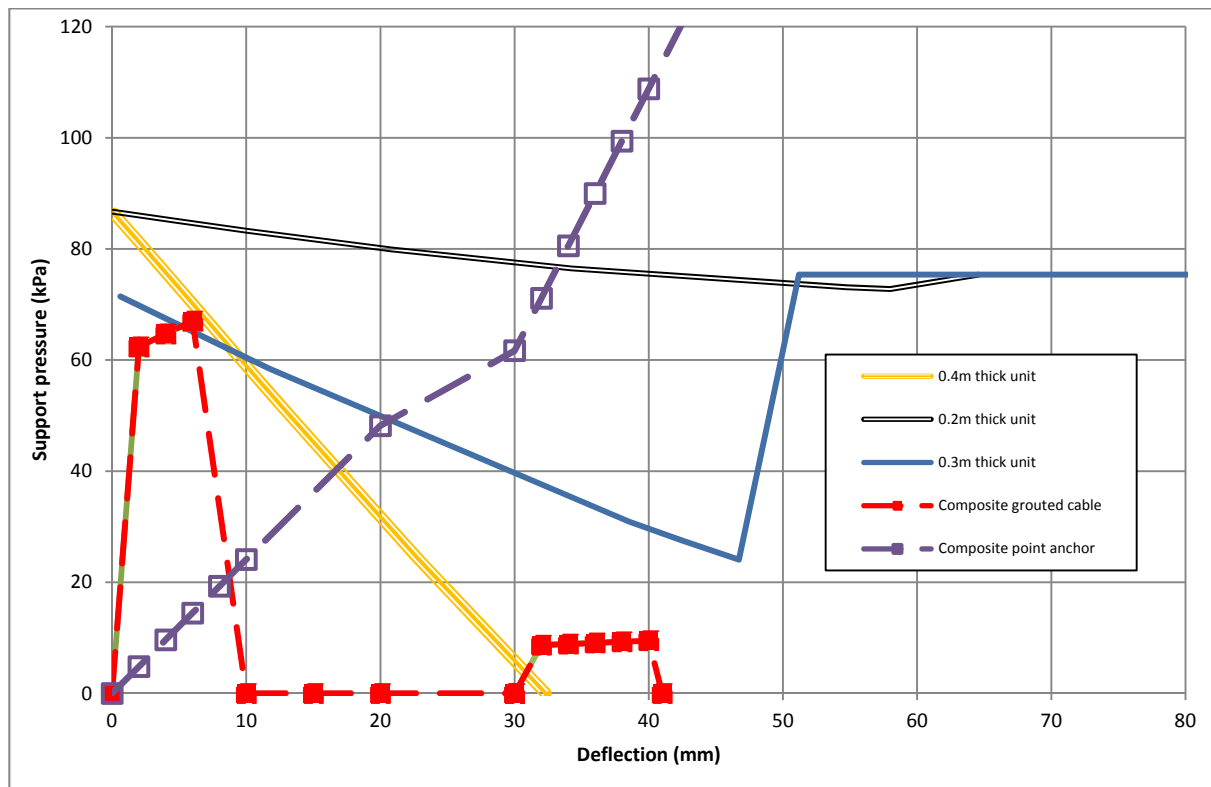


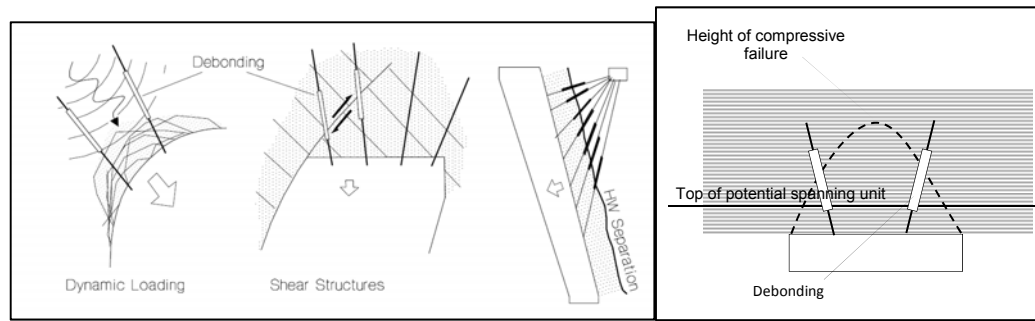
Figure 11: Full ground/reaction curves for the case study installation road

## DISCUSSION

It is not possible to know what happened within the rock mass above the installation road as the mine decided to install additional support to prevent collapse. Putting emphasis on the observations of high frequency noises, it is considered likely that there was strand rupture at some location in the roof. The mine had recovered broken cables from previous installation roadways. It is noted that there are frequent reports of broken fully grouted cables in the Australian underground coal sector (Frith and Colwell, 2011). The state of knowledge of what happens with pre-installed cable bolts in a rock mass undergoing compressive stress failure condition is inadequate and the decision to re-support was justifiable based on the available information.

Based on an assessment of the geotechnical conditions and the presumed stress field, it is highly likely the low strength siltstones within the first 5 m of the roof underwent compressive failure. A possible consequence of such compressive failure was the breakage of the preinstalled fully grouted cables during the overstressing of the rock mass. If this compressive failure mechanism did not develop, it is possible they were too stiff and broke as they tried to prevent the roof deformation associated with the development of a voussoir beam.

A “softer” cable installation method would certainly address the hazard of deformations associated with the deflection of a voussoir beam, and would possibly address the survivability of the cables in a compressive failure environment by providing greater deformation capacity (Figure 12). As discussed by Hutchinson and Diederichs (page 104), such an installation can be achieved with paint, grease, or preferably plastic tubing; reference is made to deboning of bulbed cables. The anchorage is located outside the zone of compressive failure, and the collar is grouted to minimise loading on the plate and the barrel and wedge. In critical situations even if the cable survives the onset of compressive failure, relaxation and debonding probably develops at the rock/grout interface so there is now real concern if debonding at the steel to grout interface is engineered into the tendons before installation.



**Figure 12: Desirable debonding situations (left – metal mines (Hutchinson and Diederichs), right – coal mines)**

Recommendations for future installation roadways at the site of the case study included:

1. Site characterisation before driveage of the first pass including roof core at tailgate of previous face line, geological and geotechnical log, rock strength testing. It is acknowledged that fracture logging may not be indicative of the pre-mining condition of the rock mass if the rock strength is relatively low.
2. Prediction of likely ground behaviour using simple stress elastic and voussoir beam models.
3. Support design based on dead-weight suspension and site precedent practice.
4. Use of full grouted but de-bonded cables to provide unquestioned deformation capacity.
5. Revision of critical roof deflection levels based on acceptable strain over de-bonded length of the cable.
6. Core drilling down the length of the first pass of the installation to confirm the geotechnical model used in the design.
7. Detailed extensometry and bore scopes to confirm predictions made during design.

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