Further Considerations on the Application of De-Bonded Cables

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FURTHER CONSIDERATIONS ON THE APPLICATION OF DE-BONDED CABLES

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ABSTRACT: De-bonding of the central portion of long tendons has been suggested if they are installed in low strength rock prior to a significant stress change, be that either at the maingate corner or in installation roadways. The purpose of the de-bonded section is to allow a reduction of the strains applied to the cables during failure of the rock mass and so increase the chance of them surviving large shear movements. A recent application of de-bonded cables in a longwall installation roadway resulted in a major reduction in roof deformations from in excess of 100 mm to less than 3 mm. Assuming similar rock strengths and ground stresses, the de-bonded cables apparently prevented the onset of vertical dilation in the roof. Compared to an installed shear resistance of 183 kPa, shear stresses of up to 1.5 MPa are predicted in the roof immediately above the bolted horizon. A mechanism is proposed that involves the onset of brittle failure followed immediately by a large amount of sub-horizontal shear displacement. Because of the free length of the de-bonded cables the induced strains were insufficient to cause the cable to rupture. The de-bonded cable was post-tensioned and this led to the closure of some of the dilated bedding induced on the first pass.

BACKGROUND

Seedsman (2015) proposed the use of de-bonded cables to improve the survivability of cables during stress-induced boundary crushing so that they would be available for later suspension of the failed material. That paper included a case study of a longwall installation road where roof movements at the 2.5 m horizon during the second pass exceeded 100 mm while movements at the 5.5 m horizon were less than 10 mm. In the last 12 months the same mine developed the next installation road on the other side of a 50 m wide chain pillar. De-bonded cables were used and the movements at all levels during the second pass were mostly zero, and never more than 3 mm. Whilst reduced movements were anticipated, the lack of any movement was not.

The performance of the latest installation roadway could be explained by:

1. Different geology, although the site investigation did not reveal major differences in lithologies or strengths.
2. Different stress field, possibly indicated by the poorer conditions in the stress relieving roadway. There was no intervening fault or other geological anomaly that could provide a change in the stress orientation.
4. This paper speculates on the behaviour of the de-bonded cables.

LOADING OF CABLES IN THE LABORATORY

Tension tests

Typical elongation properties of the wire used in long tendons are 0.85 % of the length at yield and 5.6 % at failure. With this information it is possible to calculate the deformation properties of the free length of a cable of any length under simple direct tensile loading (Figure 1). For a Double Embedment Test (DEPT), Clifford et al (2001) provided data on the deformation of a fully grouted double birdcage cable (Figure 1) which showed that the cable broke at a joint opening of about 9 mm. This result implies a free length of approximately 70 mm — however a direct comparison is not valid as the DEPT test result.
includes the impact of the bulbing of the cable. Other DEPT data for a typical Australian plain strand cable indicates a displacement of about 18 mm – hence an implied free length of 320 mm. It is noted that the elastic and yield deformations of the steel cannot be readily identified in the laboratory tests as they are masked to some degree by the yield and failure at the cable/grout interface which, in the Clifford test data, may be indicated at about 1 mm/300 kN.

Figure 1: Calculations of the deformation of cables based on supplied mechanical parameters and results of a double embedment pull test (after Clifford et al., 2001)

Shear tests

During 2015 a number of laboratory studies of the performance of cable bolts in shear (Figures 2 and 3) were published (Aziz et al., 2015, McKenzie and King 2015). Key results have been:

- High stiffness for the first 5 mm -10 mm of shear movement – probably the limit of the elastic behaviour of the grout ahead of the tendon (ACARP, 1995).
- Fully grouted cables breaking in tension at a shear displacement of about 50 mm (McKenzie and King 2015) or 60 mm to 80 mm (Aziz et al., 2015) of shear movement.
- No failure of de-bonded cables before 120 mm of shear displacement.

Figure 2: Shear loads-displacement graphs and a photo showing crushing of the grout, bending of the cable wires and their dominantly tensile failure (McKenzie and King 2015)
It is of value to convert these laboratory shear strengths to an installed shear resistance for typical Australian support densities. For this case study the density was 3.3 cables per metre for a 9 m span giving an installed shear capacity of 183 kPa (3.3*500/9).

**BOND STRENGTH IN THE FIELD**

Assuming that there are no stress changes applied to the borehole wall after the cable is installed, the performance of the steel/grout or grout/rock interfaces is determined by the lower of the undrained shear strength (=cohesion) of either the grout or the rock. Increased tensile loads applied to the tendon will induce shear movement at the weaker of the two interfaces, recognising that the shear stresses will be greater at the steel/grout interface because of the lower surface area.

The shear strength of typical chemical resins, as determined by punch shear tests is approximately 30 MPa with a typical uniaxial compressive strength (UCS) of 56 MPa and a Youngs Modulus of 4 GPa (Aziz *et al.*, 2014). For cement grouts, the UCS is a function of the water:cement ratio and at a ratio of 35-40% the UCS is typically in the order of 60 MPa with a Youngs Modulus of about 12 GPa (Hutchinson and Diederichs, 1996); a cohesion of 15.3 MPa at a water:cement ratio of 40 % is quoted by Moosavi and Bawden (2003). For typical friction angles, the cohesion of coal measure rocks is about 30% of the UCS so at 50 MPa the cohesion is 15 MPa.

Based on the method of Littlejohn (1993) for stronger rocks failure will be in the grout or the resin, but for weaker rocks encapsulated with resin failure will be in the rock (Figure 4). It is noteworthy that the analyses in Hutchinson and Diederichs (1996) are particularly focussed on the failure in cement grouts.

![Figure 3: Shear load-displacement results and photos of cable failures(Aziz *et al.*, 2015)](image)

![Figure 4: Typical calculations of anchorage capacity with no post-installation stress change](image)
Anchorage capacity changes if there is a stress change in the rock mass after the cable is installed. For a stress reduction, the borehole expands and if this expansion exceeds the roughness of the borehole wall there will be a loss of anchorage – this is also the reason cables are bulbed and why they should always be installed with a plate. For a stress increase, there will be mobilisation of an additional frictional strength that increases the efficiency of the anchorage.

POSSIBLE LOADING OF CABLES DURING THE SECOND PASS OF AN INSTALLATION ROADWAY

The same Transverse Isotropic Brittle (TIB) elastic model as used previously (Seedsman 2015) has been re-examined using Examine2D. In this model the UCS is 30 MPa, Youngs Modulus is 7.5 GPa and the Independent Shear Modulus is 500 MPa. The initial stress regime involves a horizontal major principal stress of 11.25 MPa and a vertical stress of 6.25 MPa. In Figure 5, the grey shaded area is where the rock has failed during the second pass mining according to the TIB model. The black zones immediately above the second pass driveage indicate the onset of tension related to relaxation into the softened zone above the first pass. A cross-plot of the final elastic stresses at each point in the prediction grid indicates that failure was primarily due to a reduction in the minor (vertical) stress and not the result of a concentration of the horizontal stresses.

A small elastic stress reduction in a horizontal plane is indicated prior to the onset of TIB failure and once failure is initiated the stresses normal to the borehole will reduce to zero. Above the TIB failed zone, at heights greater than 9.5 m, there is an increase in the horizontal stresses, and there is no change at the bolted horizon as this was de-stressed on driveage and hence before cables 1 – 3 were installed.

The expansion of a 42 mm diameter borehole in 7.5 GPa modulus rock when the confining stress is removed is estimated to be 0.04 mm. This expansion is assessed to be negligible compared to the likely roughness of the borehole wall and hence the bond strength for fully grouted cables is unlikely to have been adversely affected. For de-bonded cables, the stress reduction within the TIB failed zone is not a factor.

The continuous contour lines in Figure 5 are the shear stresses that developed during the second pass and hence after cables 1-3 were installed. Prior to the onset of TIB failure, the shear stresses imposed on cables 1 – 3 are in excess of 500 kPa and can exceed 1.5 MPa within 3.5 m of the roof line. Clearly these shear stresses are in excess of the installed capacity of the cables which is in the order of 180 kPa for 5 cables every 1.5 m across a 9 m span. Cables 4 and 5 were installed during the second pass and most of the shear stresses shown in Figure 5 would have already developed.

The dashed lines in Figure 5 are the horizontal movements that were obtained when the excavation was remodelled to include the grey TIB failure zone. They may be interpreted to be the maximum amount of shear movement that was imposed onto the TIB failure zone. At heights less than 4.5 m above the roof line, the total amount of closure movement is in the order of 50 mm to 60 mm. From the Examine2D output it is possible to calculate the orientation of the principal stress and from that the orientation of the likely shear surfaces that would be induced. Cable 2 is exposed to a horizontal shear surface and cable 3 exposed to a shear surface dipping at 20°. At cable 3, 60 mm of horizontal movement would produce 64 mm of movement along such a shear plane with 22 mm of vertical displacement.

The predicted movement is sensitive to the modulus/shear modulus ratio – a value of 20 instead of 15 increases the movements by 16%.
DISCUSSION

In Figure 6 typical results for grouted and de-bonded cables based on McKenzie and King (2015) are plotted for the case of 5 cables installed every 1.5m across a 9 m wide roadway. The shear demand lines are taken from Figure 5 assuming there is a linear relationship between the elastic shear stress before failure and final displacements after failure (Table 1). Figure 6 is in the form of a ground reaction curve (Brady and Brown, 1985) for which the interpretation is that stability is achieved if the support line intersects the ground characteristic – in this case the shear characteristic. Inspection of the figure indicates that fully grouted cables would be ruptured 2.5 m above the roof line but not ruptured at 4.5 m above the roof line. De-bonded cables would not rupture at either location.

Table 1: Linear shear demand characteristic derived from the TIB failure model

<table>
<thead>
<tr>
<th>Height</th>
<th>Shear movement (mm)</th>
<th>Shear stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 m – 4.5 m above roof line</td>
<td>0</td>
<td>500</td>
</tr>
<tr>
<td>6 m – 2.5 m above roof line</td>
<td>0</td>
<td>1250</td>
</tr>
</tbody>
</table>
In terms of roof extensometry, the rupture of cables would result in the potential for unrestrained movement of the roof below. The actual movement cannot be predicted as it is possible that some sort of arching will develop in the bolted/meshed/possibly grouted immediate roof. For the de-bonded cable, a shear movement of 60 mm could produce up to 22 mm of vertical displacement at cable 3. However at the same time the cable will have been tensioned to approximately 30 tonnes. Given the cable density, 30 tonnes is equivalent to the dead weight load of 6.6 m of rock (6.6*1.8*2.5). It is therefore suggested that the result of the shearing of a de-bonded cable is the post tensioning of the cable and the possible closure of any dilated bedding surfaces below (Figure 7).

![Figure 6: Elastic shear characteristic/cable reaction plot](image)

![Figure 7: Thought experiment showing result of 60 mm of shear displacement with de-bonded or fully grouted cables](image)
CONCLUSIONS

The much lower roof deformations recorded in the case study using de-bonded cables may be related to the cables being post-tensioned but not ruptured during the onset of failure in the rock. The way in which this develops can be considered in a number of steps:

1. Cables 1 – 3 are installed in the first pass, prior to the second pass. There is a thin zone of failure in the immediate roof, the consequences of which are controlled by the primary bolts and mesh. The installed shear capacity of the cables is about 180 kPa. There is some minor bedding dilation in the roof before the extensometers are installed.
2. Stripping of the roadway during the second pass results in a large reduction in the minor stress in the roof above the first pass and an associated small reduction in the major stress. There are no material changes to the bonding capacity of either grouted or de-bonded cables. Shear stresses of up to 1.5 MPa are induced.
3. Brittle failure in the rock develops and cannot be prevented by the cables and the shear stresses are reduced to zero.
4. Low angle shear surfaces are created, coincident with the horizontal closure of the intact rock either side of the brittle failure zone.
5. At about 2.5 m into the roof, the shear displacements are sufficient to rupture fully grouted cables: de-bonded cables have lower shear stiffness and survive.
6. Higher in the roof the displacements are less and the fully grouted cables are not ruptured.
7. For grouted cables, the roof below 2.5 m is now unrestrained and the prevention of collapse requires arching within a bolted/meshed rock mass. Higher in the roof the grouted cables are not ruptured and suspend the rest of the brittle failure zone.
8. De-bonded cables do not rupture, and all of the failed rock is suspended by the cables. The grouting at the collars ensures the full load capacity of the cables can be mobilised.
9. Shear loading of the de-bonded cables effectively post-tensions the cables and any dilation in the immediate roof may be closed.

REFERENCES

Seedsman, R W, 2013. Practical strength criterion for coal mine roof support design in laminated soft rocks, Mining Technology, 122(4), 243-249.