An update of conditions in the Donkin-Morien tunnels

R.W. Seedsman
Seedsman Geotechnics Pty Ltd, Mt Kembla, Australia

ABSTRACT: In 1984 and 1985, two 7.6m diameter TBM tunnels were driven from the Donkin-Morien peninsula in Nova Scotia to access the Harbour coal seam. The design of the ground support was based on identifying zones of rock failure through the application of the Hoek Brown failure criterion and the early use of finite element analyses. In 1992, the mining project was abandoned and the tunnels allowed to flood. Monitoring data from the tunnel was subsequently used as a case study in the development of the concepts of brittle rock behaviour. In 2007, the tunnels were pumped out as part of a feasibility study into the reintroduction of longwall mining into the coalfield. There is a good match between observations of the collapse zones in the roof and sides of the tunnel and a simple elastic analysis using brittle parameters, a spalling limit of 5, and a low independent shear modulus. The brittle analyses form the basis of the design for the resupport of the tunnels.

1 INTRODUCTION

Up until the end of the last decade, Cape Breton Development Corporation (CBDC), a Canadian federal crown corporation, operated the coal mines in the Sydney Basin of Nova Scotia. In 1984 and 1985, CBDC drove two tunnels to access the Harbour Seam under the North Atlantic Ocean offshore from the Donkin-Morien peninsula (Marsh et al, 1986). Initially, drill and blast techniques were used and subsequently the first Canadian rock tunnel-boring machine (Lovat M-300) was deployed. Despite the successful completion of the tunnels, the mining project was abandoned for a number of reasons, including a downturn in the coal market. A decision was made in 1992 to seal the tunnels and allow them to flood.

In December 2004, the Province of Nova Scotia issued a call for proposals for the exploration and development of the Donkin coal resource. Xstrata Coal Donkin Management (XCDM) was formed in a joint venture partnership between Xstrata Coal (75%) and Erdene Gold Inc. (25%) and proceeded to investigate the project. In December 2005, the Donkin Coal Alliance was granted rights to apply for a special licence for the Donkin coal resource block. The duration of the special license was three years and enabled Xstrata Coal Donkin Management (XCDM) to determine the business case for coal mining options.

XCDM decided to dewater the tunnels so as to collect a large sample of the Harbour Seam coal for testing and to allow for some in-seam drilling. Pumping began in early 2006 and by September 2007 access was available to the Harbour Seam at the bottom of the tunnels. As of February 2009, the project is in feasibility stage with a continuous miner soon to be installed to drive exploration headings down-dip to confirm some fault structures, to test seam gas permeability and desorption, and to assess extended cut mining on cross grades.

This paper discusses the conditions of the roof and sides of the two tunnels as have been revealed since dewatering.

2 BACKGROUND

2.1 *Site characterisation and initial design*

The late Carboniferous coal seams in this area of the Sydney Basin dip to the north east at about 11^{o}. The target seam subcrops under the North Atlantic Ocean. The tunnels were located in a thick sandstone unit below the McRury Seam for as long as possible at a grade of - 20% and then the grade was flattened to -1% to pass upwards through a number of seams until the Harbour Seam was encountered. The maximum depth of the tunnels is approximately 180m below sea level, and their length is 3.58km.

The Lovat TBM advanced by thrusting against full circumferential steel rings which were also used for ground support. The rings were installed about 6m behind the face. The geotechnical design work was done by Golder Associates in 1982 (Yuen and Boyd, 1987). Golder recognised 7 rock types in the tunnels (Table 1) recognising that this was, by necessity, based on limited site investigations given the desire to minimize coal sterilization. For the initial design in 1982, Golder considered a thin coal layer (rock type VIA) occurring under a mudstone (rock type VI) as representative of the worst case loading condition. The average strength of the type VI mudstone was revised downwards once information from the tunnel was available – 33 MPa to 16.6 MPa. The initial design assumed a horizontal:vertical stress ratio of 3, which was higher than the 2:1 subsequently measured.

Golder used finite element analyses to predict the maximum loading on the steel rings. Isotropic elastic properties were assumed. The presumed worst case loading condition resulted in a maximum load on the steel sets equivalent to 3.0m of loosened rock. The W150 x 23 steel sets were assessed as being capable of carrying a rock load of 3.8m and 5.7m for 1.5m and 1.0m spacings respectively.

Table 1 Changes in the Hoek-Brown strength parameters of the various lithologies between the planning and implementation phases

Type	Lithology	1982		1987		
		UCS (MPa)	Range	UCS (MPa)	m	s
I	Sandstone	50	29-93	92	5	0.1
II	Interbedded sandstone and siltstone	56		121	5	0.1
III	Siltstone	62	35-99	53	5	0.1
IV	Interbedded siltstone and mudstone	54	18-85	36	5	0.1
V	Mudstone	39	10-58	36	5	0.1
VI	Carbonaceous mudstone	33	19-47	16.6	0.05	0.00001
VIA	Coal	16.6	13-20	16.6	0.05	0.00001

2.2 *Tunnelling conditions*

It is understood the tunneling conditions were good and that there were no serious problems with the TBM. The highest weekly advance was 122m and the best daily performance was 30.5m

Geotechnical delays were encountered when the first coal seam (McRury Seam) was encountered. In this area the roof was reinforced with 2.4m and 3.5m long fully grouted roof bolts. Subsequently, similar areas immediately below any coal seams were adequately supported by reducing the ring spacing to 1.0m (Marsh et al, 1986). In their last inspection in 1989 prior to flooding, Golder reported slight flattening of some of the rings and assessed the tunnels as stable.

Two opinions were sought in on the possible corrosion of the steel rings prior to the tunnels being allowed to flood in 1992. One opinion was that any damage would be minimal and would be seen as side spalling in the mudstones and carbonaceous mudstones. The other opinion was4 that significant repairs would be required.

2.3 *Previous studies.*

Pelli et al (1991) analysed the deformation monitoring conducted at a number of sites in the tunnels. The measured “loosening zone” was not well predicted by the Hoek-Brown m and s parameters used by Golder. Significantly in the context of the ground support, the loosening heights were greater than predicted. Martin et al (1999) referred to the Donkin-Morien tunnel in their work on brittle failure. They reported that the zone of loosening was better predicted with parameter values of m = 0, s = 0.11.

2.4 *Discussion*

In Table 2, the maximum heights of failure (defined by the strength factor = 1 in Phase2) above the centreline of the tunnel for type IV and type VI rock types are compared using the original Golder strength and stress assumptions (1982), Golder strength assumptions and the measured stress field (1987), and brittle parameters and the measured stress field (1999).

Table 2 Comparison of failure heights based on 1982, 1987, and 1999 parameters

	Interbedded mudstones and siltstone Type IV			Carbonaceous mudstones Type VI		
	1982	1987	1999	1982	1987	1999
UCS (MPa)	36	36	36	16.6	16.6	16.6
σ_H (MPa)	24	10	10	24	10	10
σ_V (MPa)	8	5	5	8	5	5
m	5	5	0	0.05	0.05	0
s	0.1	0.1	0.11	0.00001	0.00001	0.11
Height	0.73m	0.28m	1.46m	>16m	>16m	9.6m

The brittle parameters and the measured stress field predict higher failure zones when compared to either of the 1982 or 1987 data sets. The earlier data sets for type VI rock predict overall failure around the tunnel. This possible situation was not identified in 1982 as the maximum thickness for type VIA rock was only 1.8m in the geology model. This paper discusses an alternative estimate of the failure zone in low strength rock.

3 OBSERVATIONS IN 2007

Once access to the tunnels was obtained, a range of conditions were observed. The driveage in the sandstone units is in very good condition. At the Emery Seam, a large roof fall to about 6m above the seam had developed. A sketch of the shape of the fall cavity is shown in Figure 1. A photograph of the side of the fall shows the closely spaced bedding that is characteristic of the carbonaceous mudstones. There was also a similar large roof fall close to the Bouthillier Seam Seam intersection at the base of the tunnel.

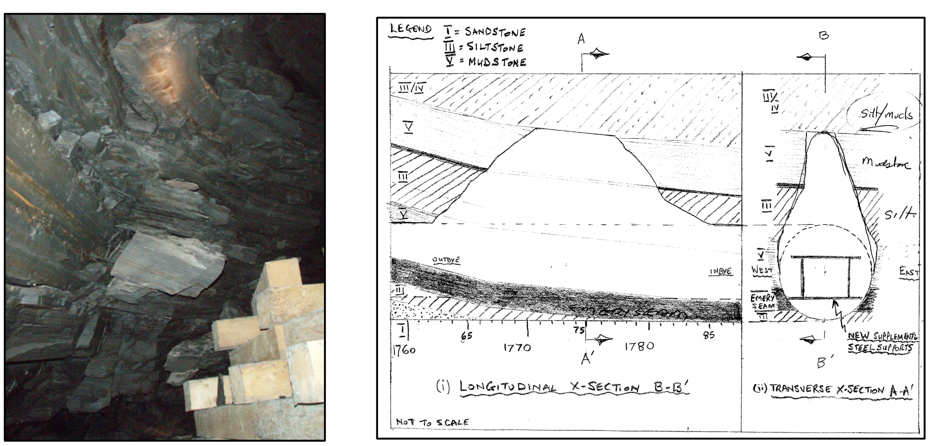

Figure 1 Photograph and sketch of the fall at the Emery Seam.

Elsewhere in the tunnel, there are areas where the steel rings have been distorted without the roof collapsing (Figure 2). In much of the tunnels located in weaker strata, the sides of the tunnel above the spring line have fallen (Figure 2). Where man holes had been excavated in the sides, there is evidence of concentric fractures (Figure 3a). There is no deformation of the rings in the type I and II sandstones. Different extents of corrosion are observed (Figure 3).

Figure 2 Typical condition of the tunnels in type V mudstones.

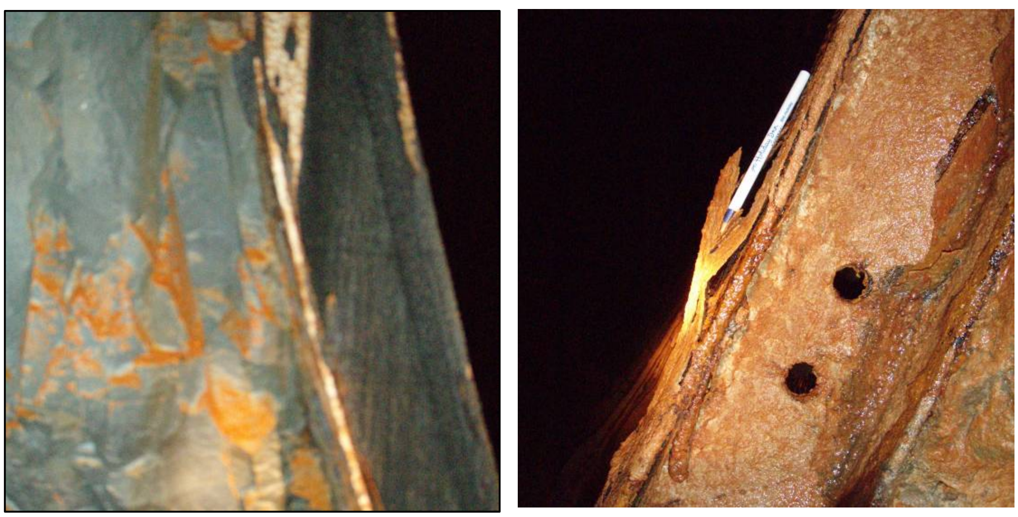

Figure 3 (a) Development of concentric fractures, (b) extreme example of corrosion.

4 ANALYSIS

In this section, the conditions revealed in the tunnel, particularly the fall of ground at the Emery Seam, are analysed in the context of the brittle failure criterion as applied to relatively low strength rock. The key features of the conditions are the height of the 2 falls (about 6m) and the depth of the damage at and above the spring line. These require consideration of the spalling limit and transverse anisotropy in coal measure rocks.

As a consequence of the well-developed bedding, coal measures are transversely anisotropic and this can lead to a concentration of stresses in the crown of an excavation (Pells, 1980). There is little guidance in the literature on the selection of values for the independent shear modulus. A series of analyses have been conducted to examine the potential maximum height of failure as a function of the UCS and the independent shear modulus, assuming a 10MPa/5MPa stress field, m=0.001, s=0.11, a horizontal modulus equal to twice the vertical modulus, a modulus/UCS ratio of 250:1, and Poisson’s ratio values of 0.2.

Figure 4a shows how the shape of the contour for the strength factor of 1.0 in 20 MPa rock changes from an isotropic elastic assumption through a range independent shear modulus values of 500 MPa to 100 MPa. It can be seen the height of loosening increases and the maximum width decreases as the shear modulus decreases.

Because it may control the height of failure for lower strength rocks, the spalling limit component of the brittle failure criterion also needs to be considered. The spalling limit (SL) is the ratio of σ_1 to σ_3 and hence is independent of strength Kaiser et al (2000) presents a plot that suggests that the range of SL is between 20 to 10, with a possible limit of 3.4 related to the strain weakening/ductile transition.

Figure 4b shows the distribution of a spalling limit of 5; contours of higher spalling limit values are closer to the excavation boundary. For this low strength rock, the maximum heights of the spalling limit contours are less than the respective contours for the strength factor.

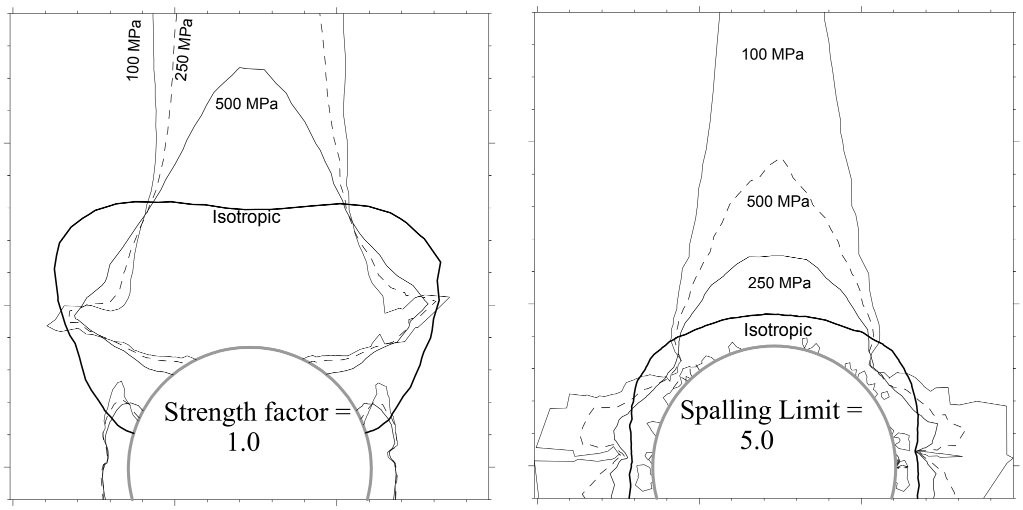

Figure 4 Contours of the strength factor and spalling limit for different independent shear modulus values

An assumption of isotropic conditions with brittle parameters does not explain the shape of the fall in the tunnels. Introducing a spalling limit of 5 provides a better fit. The maximum height of failure defined as the minimum of either a strength factor of 1.0 or a spalling limit of 5 is plotted against the independent shear modulus and UCS (Figure 5). The 3.8 m and 5.7m heights used in the specification of the rings is also shown.

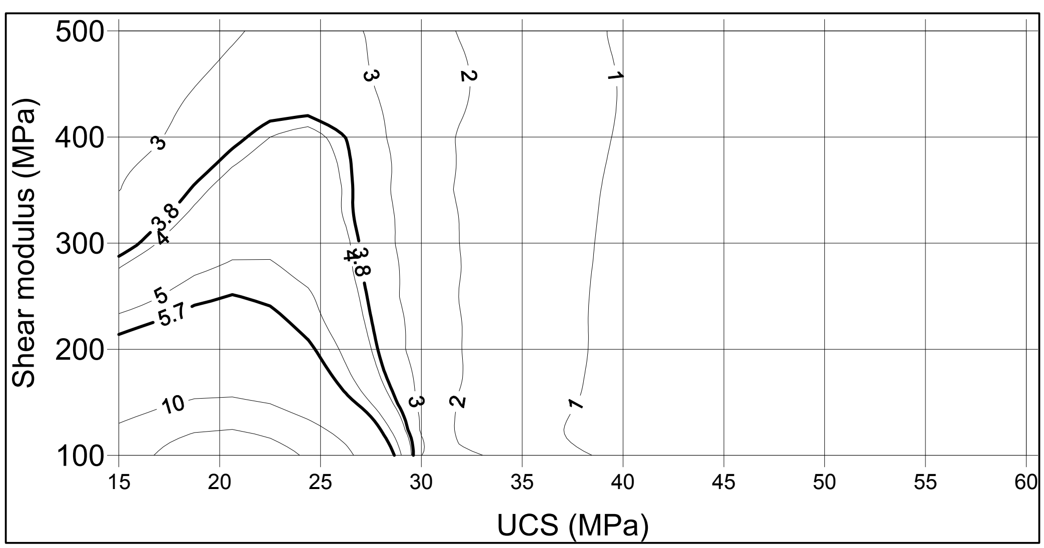

Figure 5 Maximum height of failure considering both brittle strength and a spalling limit of 5

5 ASSESSMENT

In this back-analysis, a reasonable fit to the range of conditions in the tunnels in rock types with UCS values less than about 30 MPa with a combination a independent shear modulus of approximately 250 MPa and a spalling limit of 5. This analysis suggests that the installed rings were inadequate for rocks with strengths less than about 25 MPa. Referring to Figure 1, this means all of type VI and VIA materials, and possibly some of V and IV may have not been adequately supported. It is not possible to determine if corrosion has played a major part in the tunnel deterioration.

It is considered that much of the support in the tunnels cannot be relied upon. A bolting strategy assuming suspension from un-failed material above the zone of brittle failure has been developed. It has been recommended that materials with a UCS between 30 MPa and 50 MPa be re-supported with roof bolts installed through the existing mesh panels between the steel rings (Figure 6). For rocks with a lower strength, a higher level of support will be used. A routine to assess the rock strength and to account for layering during the re-supporting program will need to be developed.

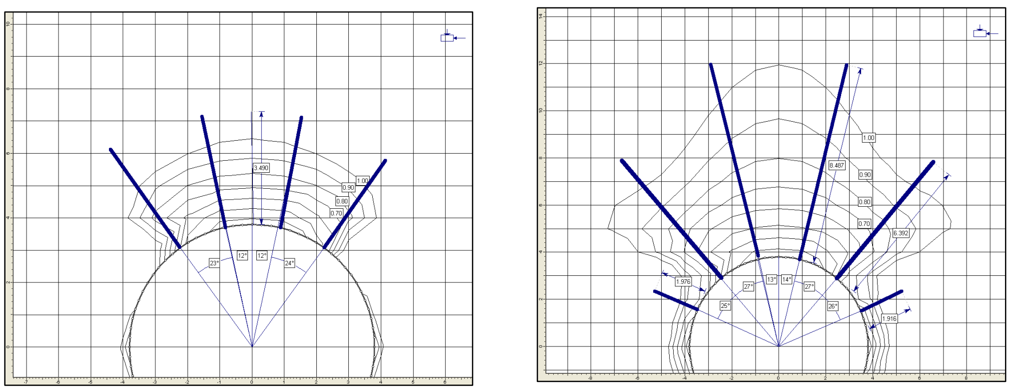

Figure 6 Candidate re-support for type III, IV, and V materials

6 CONCLUSION

Coal measure rocks can be analysed using the brittle failure criterion if both transverse anisotropy and low spalling limits are included. Critical spalling limits are lower for relatively low strength rocks. Using brittle parameters and the stress redistribution about a hole in an isotropic material, it is possible to use the height of failure at the centerline of the roof to place maximum limits on the Competence Factor (UCS/far field vertical stress, Muirwood (1972)) and the spalling limit (Figure 7). Transverse anisotropy increases the tangential stresses in the crown so that the values given in Figure 7 are maximums. Relatively lower strength rocks can have higher zones of failure, but these only develop if the spalling limit is comparatively low.

There is a need for more information to guide the selection of suitable spalling limit and independent shear modulus values. A spalling limit of 5 is indicated for the low strength mudstones in the Donkin-Morien tunnels. In other work in Australian coal mines, the author has found that a spalling limit of 5 is appropriate for similar strength materials. The Eh/G ratio of 50 implied by this study is within the range of values considered by Pells (1980).

For the Donkin-Morien tunnels, this case study also highlights the need to design on lower than average values of strength if the available data is from a limited site investigation. As mentioned, it is not possible to resolve the question of the contribution of corrosion to the deterioration of the tunnels.

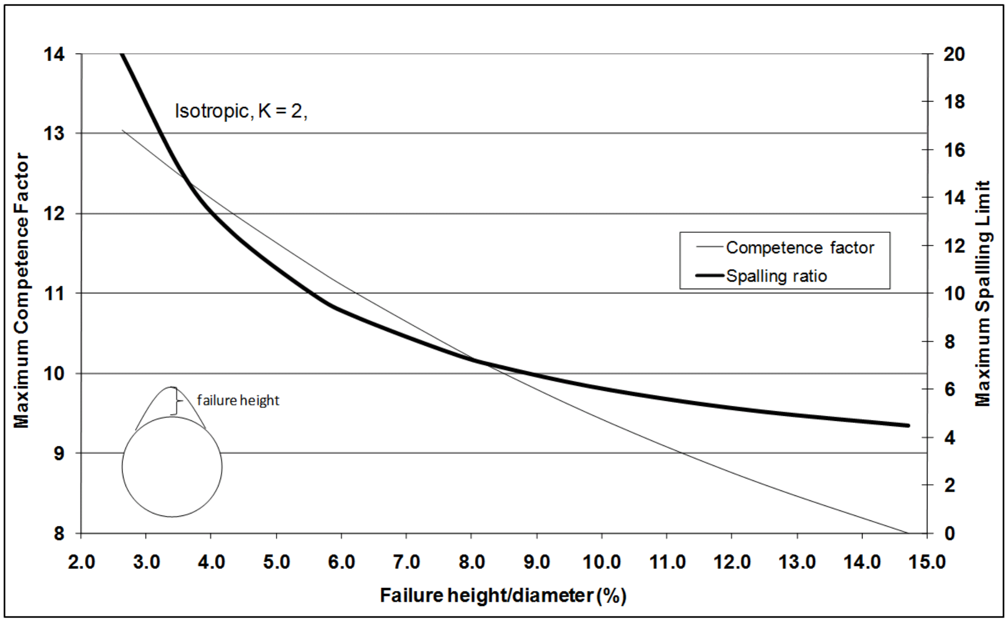

Figure 7 Limits to the Competence Factor and the spalling limit for isotropic materials about circular excavations

7 ACKNOLWEDGEMENTS

Permission of XCDM to present this case study is gratefully acknowledged. The views outlined in the paper are those of the author and not necessarily of XCDM. The assistance of Darren Nicholls, the XCDM's Project Manager on site, and Malcolm Cox and Kerry Whitby in Australia was invaluable. David Forrester mapped the conditions in the tunnels, provided technical support, and oversaw the safety hazard management plan during the subsequent drilling.

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