REINFORCEMENT DESIGN OF THE CATARACT WATER TUNNEL TO WITHSTAND LONGWALL SUBSIDENCE

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SUMMARY

This paper presents the procedures followed in the design of the reinforcement system for Cataract Tunnel. The Tunnel is a 19th century unlined tunnel under which a series of longwalls will be extracted at Appin Colliery. Based on the prediction of subsidence levels, three dimensional stress analyses were conducted to investigate the range of additional stresses which were likely to be induced around the tunnel. The impact of the induced stresses on the tunnel were then analysed in detail and a reinforcement system designed.

It should be noted that, as in most geotechnical situations it was not possible to define all the relevant variables. Stress variations outside those used in this analysis are possible. In recognition of this and the uncertainty of the actual stress redistributions, the design has been used as the basis for a minimum support pattern. It is envisaged that this may be supplemented by additional reinforcement such as additional bolts or cable tendons as determined by a planned program of monitoring and observation.

1. INTRODUCTION

The following paper covers the stages of the design of the reinforcement system for the Cataract Tunnel which is to be subjected to subsidence induced by mining a series of longwall at Appin Colliery, Figure 1 and 2. The following design process was specified by Sydney Water to ensure that the design was based on a systematic approach utilising best practice at all stages of the design.

2. SURVEY OF BEST PRACTICE

The initial purpose of the best practice study was to search for any relevant previous experience regarding the behaviour to unlined tunnels that have been undermined and subjected to subsidence. The survey also included methods of design on the unlined tunnels for subsidence impact.

An extensive search through the Internet, conference and symposia proceedings has failed to identify any publications of relevance.

Even extending the review to include the affect of subsidence on lined tunnels, no publications were noted. Although specific publications were not found, it is known that at least three common incidence of tunnel undermining have occurred.

- Historic metalliferous workings undermined by coal mining activity.
- Existing coal mine workings undermined by longwalls.
- Near surface structures undermined by longwalls.

Unfortunately, even in these areas, no specific written and quantitative data was found relating to the affect of subsidence on tunnels and most records are not commonly accessible.

3. ASSESSMENT OF PRESENT STABILITY OF THE UNSUPPORTED TUNNEL

The Cataract Tunnel is an unlined 19th Century tunnel mined primarily in Hawksbury Sandstone with occasional layers of interbedded sandstones and siltstones. The depth of cover of the tunnel varies up to a maximum of 70 metres.

3.1 INITIAL GEOTECHNICAL APPRAISAL AND MAPPING

3.1.1 State of the Tunnel

Over the majority of the length of the tunnel the rock quality is good, consisting of coarsely bedded sandstones. Throughout the majority of the length of the tunnel however it is clear that a significant level of shear failure has occurred. There is evidence that a proportion of this failure has occurred on driveage, both from the shape of the opening formed as well as the random orientation of the triangular chisel marks indicating a high level of 'barring down' of material during construction.

In addition to the material failure on driveage, continued deterioration through shear fracturing has occurred leading to additional debris on the floor of the tunnel. The conditions are summarised in Figure 1.

Buckling of the concrete invert in the floor has been noted again indicating deformation of floor strata subsequent to construction.

3.1.2 Stressfield

An assessment of the stressfield has been undertaken from mapping of rock failure along the tunnel and from an extrapolation of stress measurements made in surrounding mines.

The directions of the principal horizontal stresses, which are determined from the orientation of shear fractures within the roof strata, are presented in Figure 1 and 2. The direction of the stresses observed is consistent with those measured in nearby collieries at the Bulli Seam level.

Using extrapolation of the stress magnitudes measured in the mines it is possible to establish potential values of stress, which may be acting at the tunnel horizon.

The horizontal stress magnitude can only be expected to fall within a range, and variation is possible. The average stresses anticipated at various depths from the data available is presented in Table 1. Vertical stress is based on a rock density of 2.5g/cc.

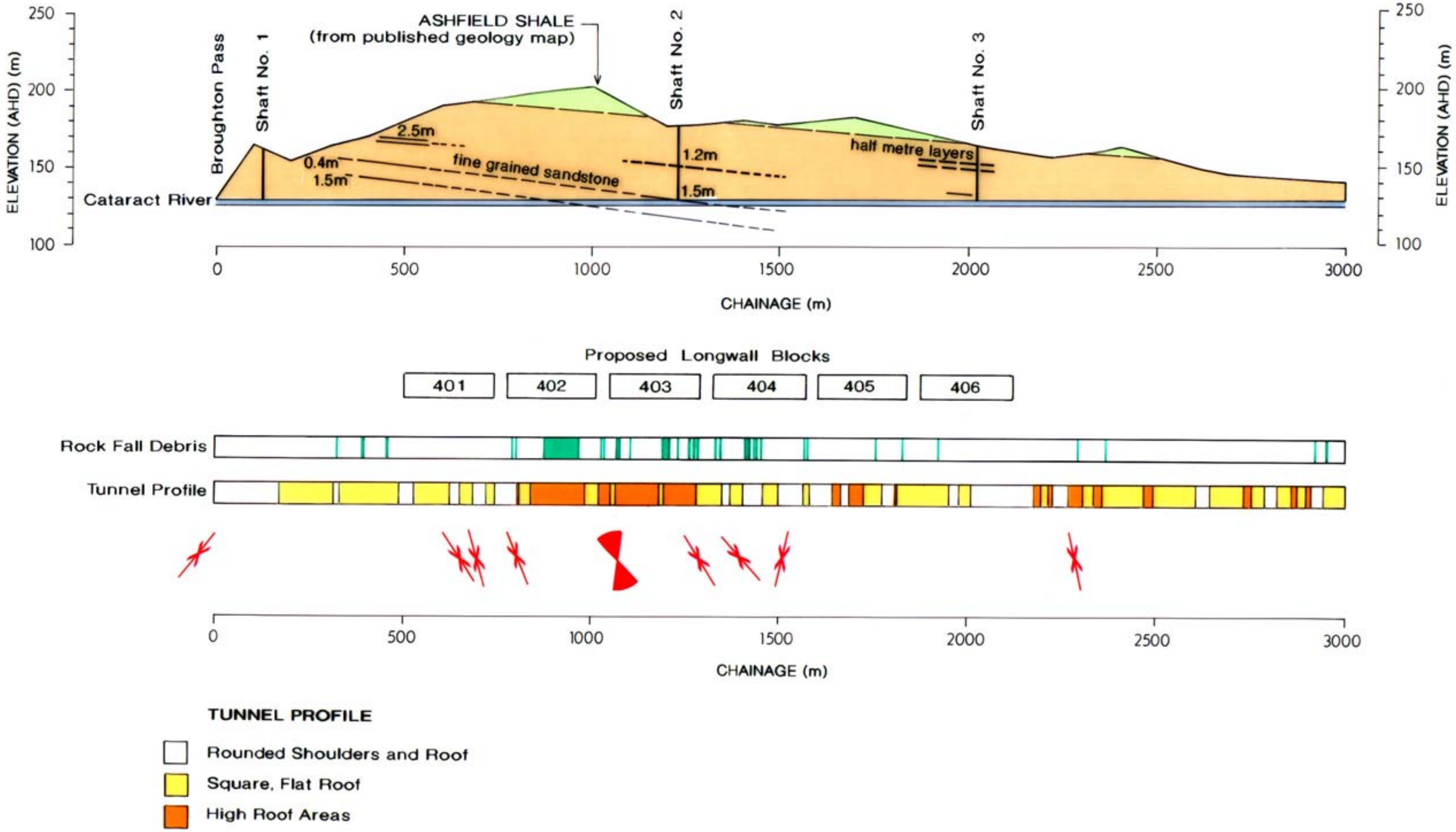


Figure 1 – CATARACT TUNNEL

Mapped stress directions and roadway conditions.

(Roadway conditions from Golder Assoc. drawing for Project No 96621135).

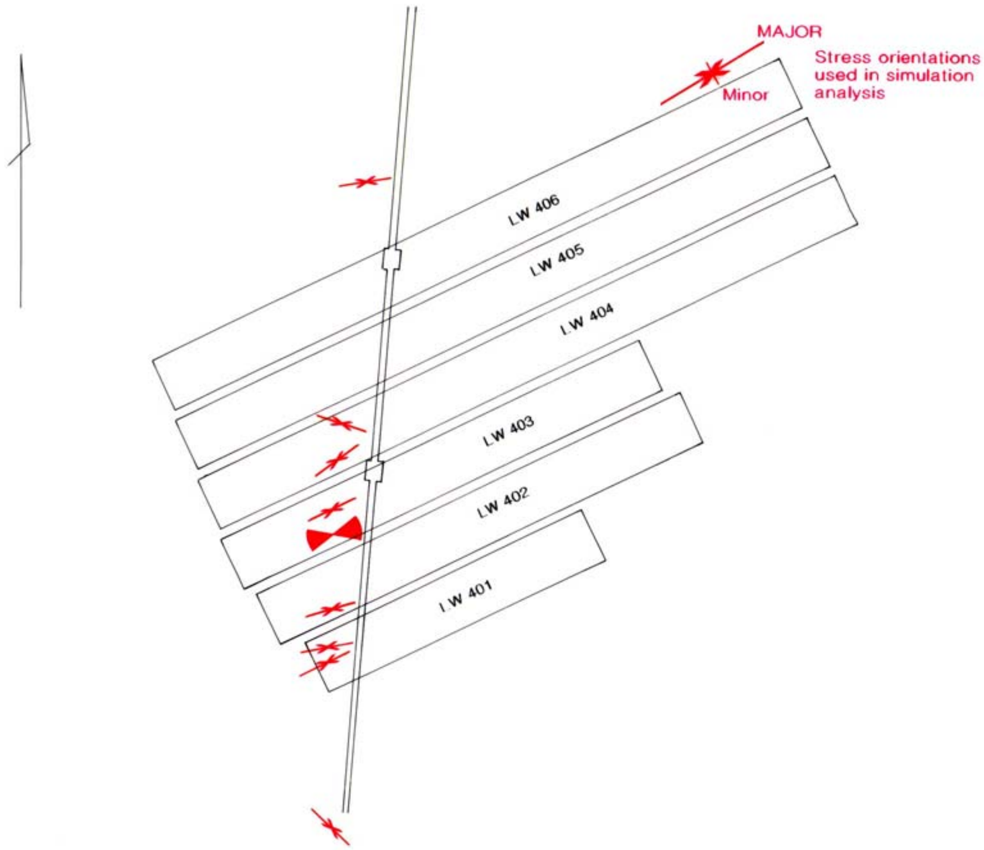
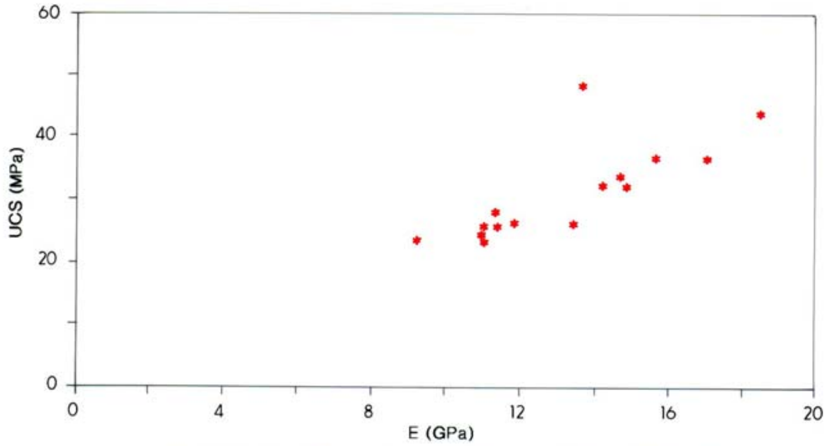
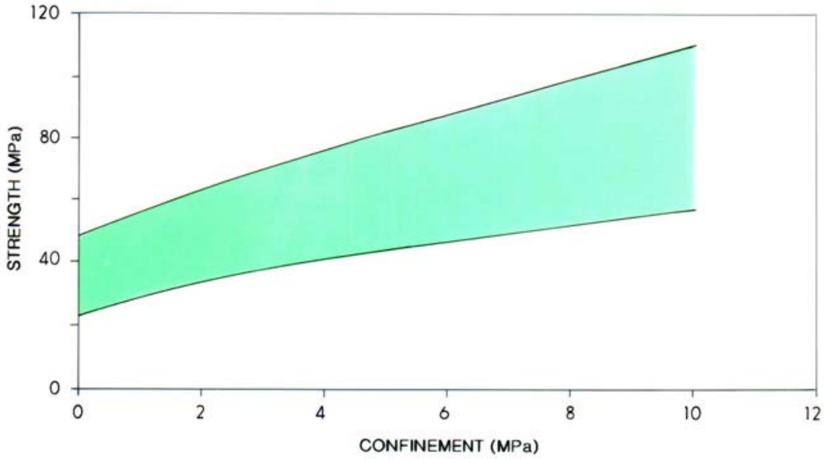


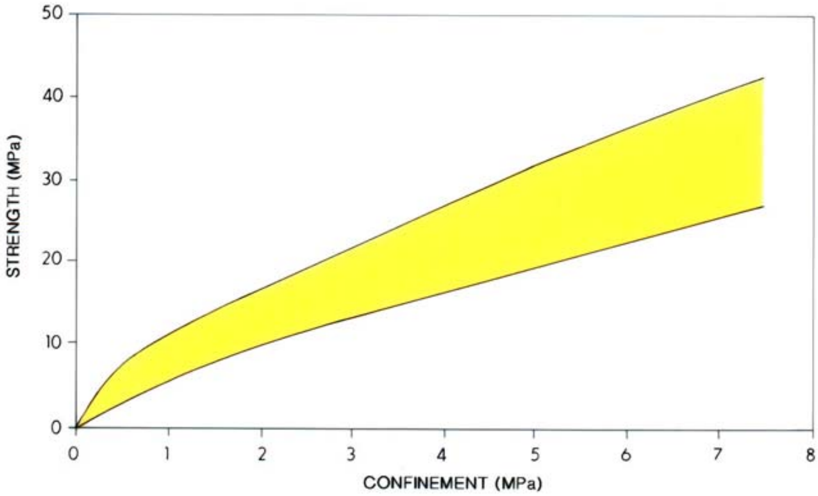
Figure 2 – CATARACT TUNNEL
Stress directions as determined from shear fractures in the Cataract Tunnel with respect to proposed longwalls.



a) Range of UCS and Youngs Modulus of tested samples.



b) Range of intact triaxial strength properties.



c) Range of residual triaxial strength properties.

Figure 3 – CATARACT TUNNEL
Strength properties of rocks tested.

TABLE I: STRESSFIELD DATA USED IN THE ANALYSIS

Depth (m)	Major Stress (MPa) Horizontal (Northeast)	Minor Stress (MPa) Horizontal (Southeast)	Vertical Stress (MPa)
20	3.2	1.75	0.5
40	4.5	3.9	1.0
60	5.7	4.5	1.5
80	7.0	5.5	2.0

3.1.3 Deformation Mechanisms

Observations of the tunnel during the site inspection have defined the rock failure modes about the tunnel. Bedding planes and joints are the main geological structures present in the tunnel. The occurrence of jointing does not appear to be the controlling factor in the tunnel deformation.

The principal controlling factor on the deformation process of the tunnel is the failure of the strata through shear as a result of overstressing of the rock about the tunnel. The level of shear observed is a function of the depth of cover of the tunnel and the local lithology. The position in the roof of the finer bedded sandstones and siltstone units has a significant role in the location of the falls, as well as observed level of deformation in the immediate tunnel roof. The location of roof falls and the inferred roof lithology is presented in Figure 1.

4. ROCK PROPERTIES

4.1 Sampling Program

A series of 3 boreholes were drilled from the surface to below the tunnel horizon to confirm the lithology, recognising the significant role played by the weaker fine grained sandstones and siltstones on influencing behaviour of the tunnel.

4.2 Rock Strength

The range of Unconfined Compressive Strength (UCS) and Young's Modulus of the samples tested are presented in Figure 3a. The range of strength values, obtained from extrapolation from the multi-stage triaxial tests, ranged from 23 to 48MPa. Young's Modulus values ranged from 9 to 18GPa.

The triaxial intact and residual strengths of the samples are summarised in Figures 3b and 3c respectively.

Visual observation during the tunnel inspection indicated that there where sections of roof consisting of heavily listricated siltstones. These siltstones were apparently not intersected during the testing program and strength properties for these units were extrapolated from previous studies.

4.3 Rock Strengths Used in Analysis

The unconfined compressive strength of the rock units was presented in Figure 4. The strength of these units when confined in situ is determined by the triaxial strength factor, which indicates the increase in strength of the rock for every MPa of confining stress.

The triaxial strength factor in the confining stress range of 0-2MPa was approximately 5.

Bedding plane properties were not obtained during the testing, however relationships obtained from other test programs and our database was used to estimate the bedding plane properties.

The range of data used is presented in Table 2.

Table II ROCK PROPERTIES USED IN THE ANALYSIS

Rock Unit	UCS (MPa)	E (GPa)	Bedding Friction (degrees)	Bedding Cohesion (MPa)
Sandstone	20	9	34	3.5
Sandstone	25	12	34	4
Sandstone	40	15	34	6
Sandstone	32	14	34	4
Shale	20	6	30	2.5
Listricated Siltstone	3	6	10	0.25

5. BACK ANALYSIS OF ROCK STABILITY DURING CONSTRUCTION

5.1 Initial Stability Calculations of the Tunnel Crown

An initial estimate of the stability of the rock about the crown of the tunnel both during and subsequent to construction has been obtained from two methods. These were:

i) a simple calculation of the normal tangential stresses in the crown relative to various rock strengths,

ii) a computer simulation of a rock section under a range of horizontal stresses.

5.1.1 Simple Analysis Based On Kirsch Equations

This analysis was done to obtain an initial overview of the potential stresses about the tunnel and the effect of variation in rock strength along the tunnel.

If the arch shaped tunnel profile during construction is approximated as semi circular, the maximum stress in the crown area is approximated by:

Maximum stress = 3 * maximum horizontal stress - vertical stress

Similarly, if we assume the shafts to be elliptical and elongated in the southeast - northwest direction in the ratio of 1.38:1, the maximum stress about the shaft is approximated by:

Maximum stress = σ Northeast (3.76 - $\sigma_{\text{Southeast}}/\sigma_{\text{Northeast}}$)

The confinement about the immediate tunnel or shaft is considered to be negligible and the strength of the rock will be its UCS * a field strength reduction factor. The strength reduction factor selected is 0.55 which is consistent to our back analyses of other sites and also with the Hoek and Brown (1980) relationship of (50/d) 0.18 where d=1cubic m.

Using these relationships the rock strength expected to fail in the crown of the tunnel and in the sidewall of the shaft at various depths is presented in Table 3. Units having the tabulated strength or lower would be expected to fail in shear.

Table III: ROCK UNITS EXPECTED TO FAIL IN THE CROWN AND SHAFTS

Depth (m)	Crown UCS (MPa)	Northwest/ Southeast Axis Shaft UCS (MPa)	Northeast/ Southwest Axis Shaft UCS (MPa)
20	N/A	<19	<2
40	<23	<24	<6
60	<28	<31	<10
80	<35	<38	<12

The table indicates that the weaker sandstone units would be expected to fail in shear in the crown during and subsequent to construction. The inclusion of weak shale units and listricated bands would increase the potential for failure. Similarly, areas of higher stress would fail stronger units

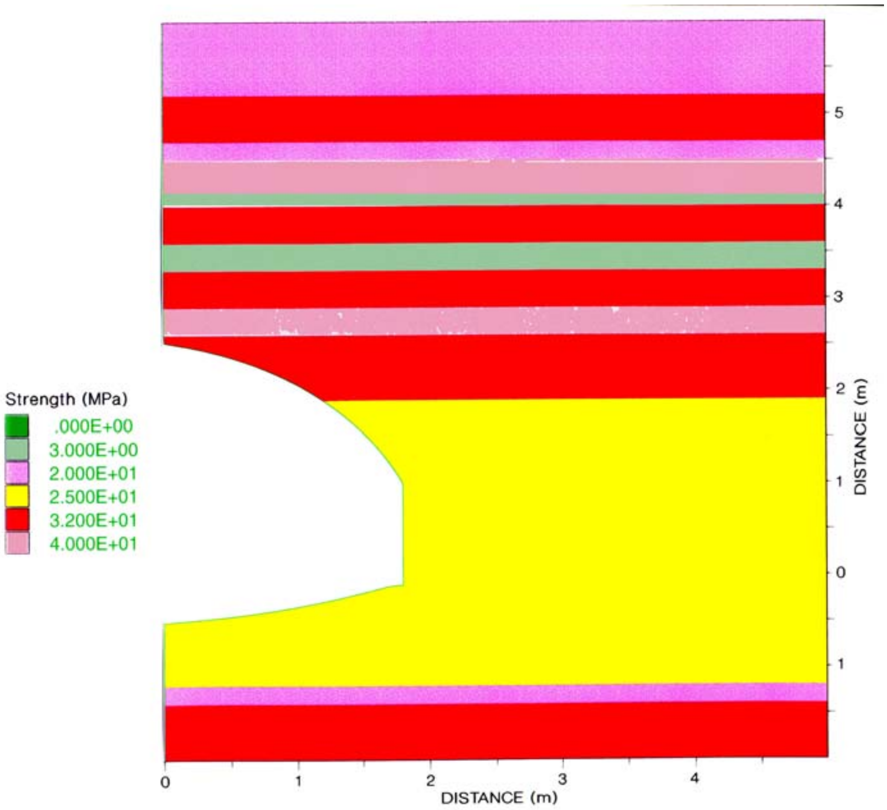


Figure 4 – CATARACT TUNNEL
Rock strength section used for the tunnel area.

5.1.2 Computer Simulation of Rock Failure about the Tunnel

A more detailed study was undertaken using two dimensional modelling of a rock section generated on the basis of the rock testing data. The rock section is presented in Figure 4 on the basis of the units UCS. The section is considered to represent a conservative situation for which two weak listricated siltstone units exist 1 and 1.5m respectively into the roof.

The computer code is FLAC (Version No. 3.04) and mohr-coulomb rock failure routines utilising the triaxial rock strength properties for the intact and failed rock at various confining pressures is used. A rock strength reduction factor of 0.55 has been used to derive the in situ equivalent UCS of the intact material and bedding plane strengths. The UCS, Young's Modulus and bedding plane properties of the units used are presented in Table 2 and the triaxial properties are presented in Figure 3. The lower bound residual rock properties were used in the model. A half symmetry model was used for the tunnel. The model was 40m in the horizontal dimension and 90m in the vertical dimension. The output of data from the model shown in this paper are windows about the tunnel within the larger model grid.

The modelling approach was to initialise stresses in the ground and then excavate the tunnel in an existing stressfield. The stresses in the ground were based on the relationship presented in Table 1, whereby the maximum horizontal stresses were oriented across the tunnel. The depth for the simulation was 65m and the vertical stress was

1.625MPa. The geological section was developed to be generally consistent with the core log at Shaft No. 2, and the ground expected in the region of 850-1000m section close to Shaft No. 2, which has experienced significant roof failure. The tunnel was unsupported.

The results of the simulations are presented below which indicates the deformation expected at various horizontal stresses within the reference unit adjacent to the tunnel. This unit is a 25MPa sandstone having a Young's Modulus of 12GPa. The results indicate that failure would occur in the crown at a horizontal stress of approximately 4.5MPa, and that failure up to the first listricated siltstone unit would occur at a stress of 5-6MPa. The areas of fractured ground are presented in Figure 5.

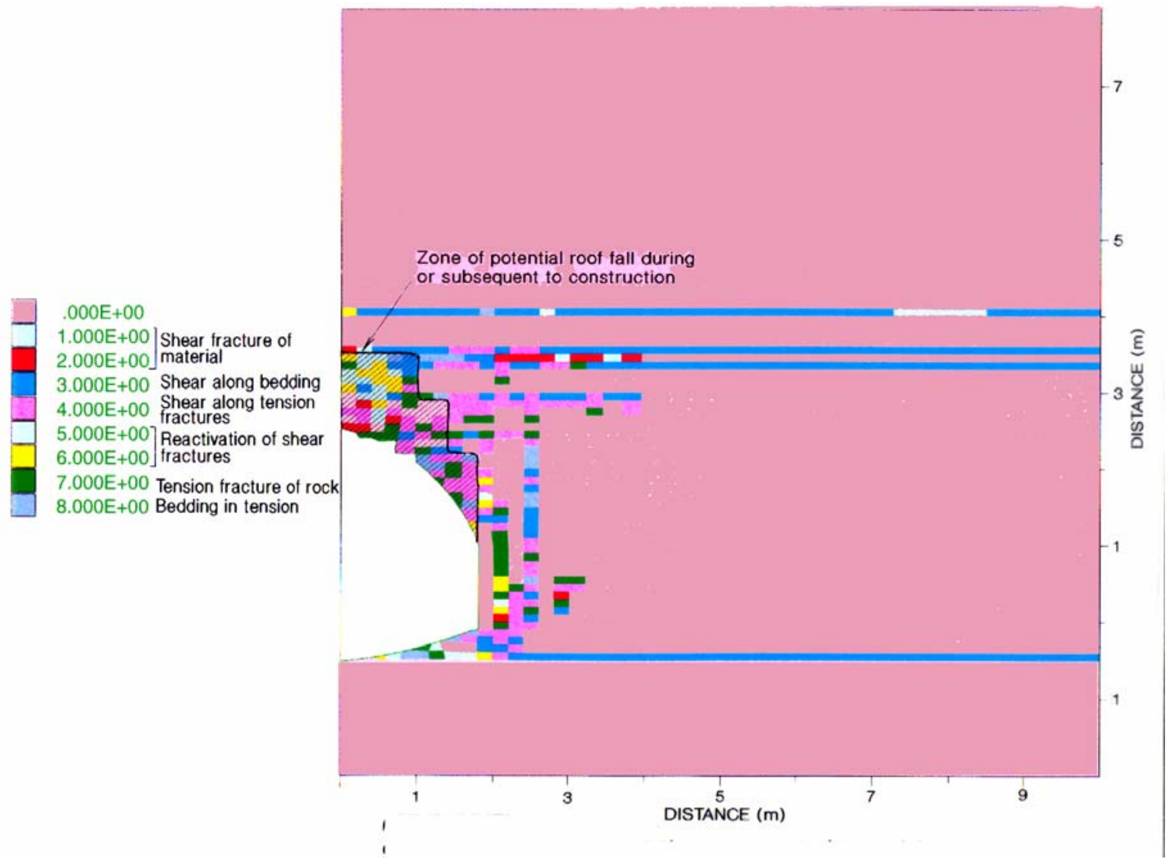


Figure 5 – CATARACT TUNNEL
Areas of fractured ground about the unsupported tunnel for 6MPa horizontal stress

The model also indicates failure of floor strata and lateral slip along bedding in the floor. Buckling and cracking of concrete inverts would be expected where deformation and rock failure has occurred subsequent to construction.

These results are consistent with the simple initial analysis and also with the actual tunnel sections observed. The results indicate that the model input and modelling method is simulating the behaviour of the material under the stress conditions expected to have existed during construction of the tunnel.

The initial analyses indicate that the stability of much of the tunnel after construction would have been marginal where the weaker units exist and rock falls subsequent to construction would be expected. The observed condition of the tunnel is considered to be consistent with this initial review.

In areas where the sandstone is stronger the tunnel stability would be greater.

These results indicate that re-activation of fallen areas and fracture of new areas of the tunnel would be likely with increased stresses developed about the tunnel due to subsidence.

6. ASSESSMENT OF THE POTENTIAL STRESS CHANGES CAUSED BY SUBSIDENCE OF THE TUNNEL

6.1 Stress Conditions Anticipated from the Three Dimensional Modelling

The subsidence and strains due to the undermining were determined by Waddington and Associates. Strains will be both tensile and compressive at various stages of the mining. Also, the orientation of the strains will vary in direction and elevation during mining. The tunnel (or sections of it) is expected to be subsided approximately 1.4-1.5m and surface strains in the range of +1.11 to 1.50mm/m are anticipated (Waddington and Associates).

The variation in stresses caused by undermining was assessed by M.A. Coulthard and Associates using three dimensional modelling of the ground subsidence profiles provided. A computer model of the initial 100m of overburden was developed. The modelled strata was then deformed into the shapes expected from the subsidence information provided. Initially only transverse subsidence strains were considered in the project scope, however, this was subsequently extended to include longitudinal strains due to undermining. The stresses obtained from the ground distortions at various mining geometries were evaluated to obtain an estimation of the stress magnitudes to be expected, and an understanding of the factors which may cause variation in those stresses.

On the basis of the information provided, the ground was considered to act as a mohr-coulomb material in the three dimensional modelling. Sensitivity studies conducted to assess the effect of bedding plane slip of certain units in the overburden indicated that the stresses may vary significantly should such conditions arise.

An overview of the potential field stresses expected for various extraction geometries are presented in Table 4. The stresses are those outside the immediate influence of the tunnel. These represent the maxima or minima along a traverse of the tunnel in the area of 403 and 404 Panels. It is at this stage that maximum subsidence would be expected.

Table IV: STRESS CONDITIONS EXPECTED

Panel Geometry	Stress Across Tunnel	Stress Parallel To Tunnel	Maximum Principal Stress	Minimum Principal Stress	Vertical
A. Case 1.	Final Transverse Subsidence The Overburden In A Uniform Mohr-Coloumb Material				
403 extracted	4.5-5	5.5	11	-4	-4
404 extracted	5	5	11	-4	-4
Case 2.	The overburden has discrete weak bedding planes in strata about the tunnel.				
403 extracted	7	9	10	0	0
404 extracted	7	9	10	0	0

Panel Geometry	Maximum Principal Stress	Minimum Principal Stress
B. Case 1.	Transient Stresses During Longitudinal Subsidence The Overburden In A Uniform Mohr-Coloumb Material	
403 being extracted	9	-4
404 being extracted	10	-4

It is noted that there are no systematic long term tensile stresses formed normal to the tunnel in the horizontal plane, and that the vertical and minimum principal stresses may vary significantly if bedding planes fail during transient undermining.

It is also noted that during the active undermining stage, transient tensile stresses may develop in the horizontal plane. If bedding planes fail about the tunnel then the vertical stresses will be limited to a minimum of 0. Also, the horizontal stresses may vary in a similar manner to that for the transverse data.

The results of the three dimensional modelling indicates that the normal and shear stress patterns will be very complex and changing. It is not possible to evaluate all possible situations.

7. Assessment of the Reinforcement Patterns Expected to Maintain Stability under the Modified Stress Conditions

7.1 Background and Approach Adopted

The approach used for this situation has been to review various approaches based on civil tunnel experience and those from mining experience.

Review of the literature reveals that this proposal is unique and there are no standard approaches to designing a support system for such situations.

The approach used in this study has been to utilise a system which can reinforce the rock about the tunnel during the ground displacements and strains. The potential performance of such a reinforcement system has been assessed using computational modelling of the ground at a number of key stress conditions expected during mining.

The approach used for design has been to assess the stability of ground, with a reinforcement system placed in the tunnel, under the action of additional horizontal stresses elevated to that expected from the three dimensional modelling.

The grid size and material property descritisation used in the large scale three dimensional model and potential timeframe required, was not suitable to analyse the detailed behaviour of the reinforcement system about the roadway. In order to practically assess the potential effect of the stress changes during undermining on tunnel, the two dimensional model as used in the initial assessment of tunnel stability was utilised.

For this study the direction of the principal stresses were assumed to act across the tunnel and vertical stresses were associated with the in situ state.

Potential variation of shear stresses and local variation in vertical stress were not evaluated in this analysis. This was due to the wide range possible under various combinations of bedding plane failure which could not be fully evaluated in the three dimensional analysis.

It should be noted that, as in most geotechnical situations it has not been possible to define all the relevant variables. Stress variations outside those used in this analysis are possible.

In recognition of this and the uncertainty of the actual stress redistributions, this design has been used as the basis for minimum support which is envisaged to be supplemented by additional reinforcement such as cable bolts:

i) in more heavily deformed areas of the current tunnel,
ii) in areas of excessive deformation of the tunnel (noted during mining) resulting from stress variations more severe than anticipated.

Regular monitoring and inspection of the tunnel would be required to assess the actual performance and final reinforcement requirements for stability during and subsequent to mining.

7.2 Anticipated Action of Reinforcement

A reinforcement system is recommended in preference to a support system, as it can displace with the ground about the tunnel with significantly less damage to the system than would be the case with a standing support system.

A reinforcement system also allows maximum access and utility of the tunnel during the undermining process.

The reinforcement action about the tunnel is to provide confining stress across fractures to enhance their strength to form an interlocking mass capable of supporting the weight of the broken rock zone.

The contribution to the strength of the fractured rock provided by the bolting is estimated in general by the relationship of:

$$\sigma_1 = C_R + K\,\sigma_{conf.}$$

Where C_R = Residual cohesion of the rock and bolt shear capacity σ_{conf} is confining pressure developed by the bolting system.

The actual behaviour of the system is best assessed using computer simulation of the fracture geometries, bolt forces developed and surcharge loads placed on the bolted ground.

The bolt pattern suggested for this tunnel on the basis of mining experience is presented in Figure 6 which utilises a 2.44m long, 24mm nominal diameter X grade bolt in the roof and a 1.8m bolt in the ribside. The nominal capacity of the bolts is approximately 340kN. Mesh is to be used in the roof and on the sidewall to maximise the strength of the loosened material between bolts. The bolt pattern is expected to provide a confining stress about the crown area of at least 0.15MPa to reinforce any fractured rock from dislodging during undermining. The confining stress distribution for the bolting system is presented in Figure 7 on the basis of stresses developed in the rock at a bolt load of 28 tonnes. This pattern would be expected to provide greater reinforcement to the tunnel than that determined from other techniques.

The performance of the reinforcement system has been evaluated by placing the bolt pattern and bolt load transfer characteristics into the two dimensional computer model and varying the horizontal stresses progressively.

The extent and severity of rock failure about the tunnel was determined for various stress conditions.

The results of the modelling are summarised in Table 5.

Table V: RESULTS OF TWO DIMENSIONAL MODELLING OF MINIMUM ROCK BOLT PATTERN

Field Horizontal Stress (MPa)	Roof Displacement (mm)	Height of Failure (m)	Floor Displacement (mm)	Depth of Failure (m)
4.6	<3	<0.1		
7.4	<4	<0.1		
9.6	20-30	1.2	150	1.5
12	130	1.2-1.5	300	1.5
16	250-300	4-5	500-600	4-5

The modelling indicates that in the regular tunnel geometry the minimum bolting pattern would be expected to maintain roof stability for normal stresses up to 12MPa. The rock failure distribution at 12MPa is presented in Figure 8.

If stresses exceed this 12MPa or tunnel sections have weaker geology, rock failure would be expected to rapidly increase requiring the placement of secondary reinforcement.

On the basis of the three dimensional modelling and the simplifications to the stress variability made, this pattern should be suitable as a minimum pattern to satisfy the requirements outlined in 7.1. It should be noted, however that the actual stresses developed will be variable in magnitude and orientation during mining, and that regular monitoring and inspection during mining will be required to determine the final reinforcement system.

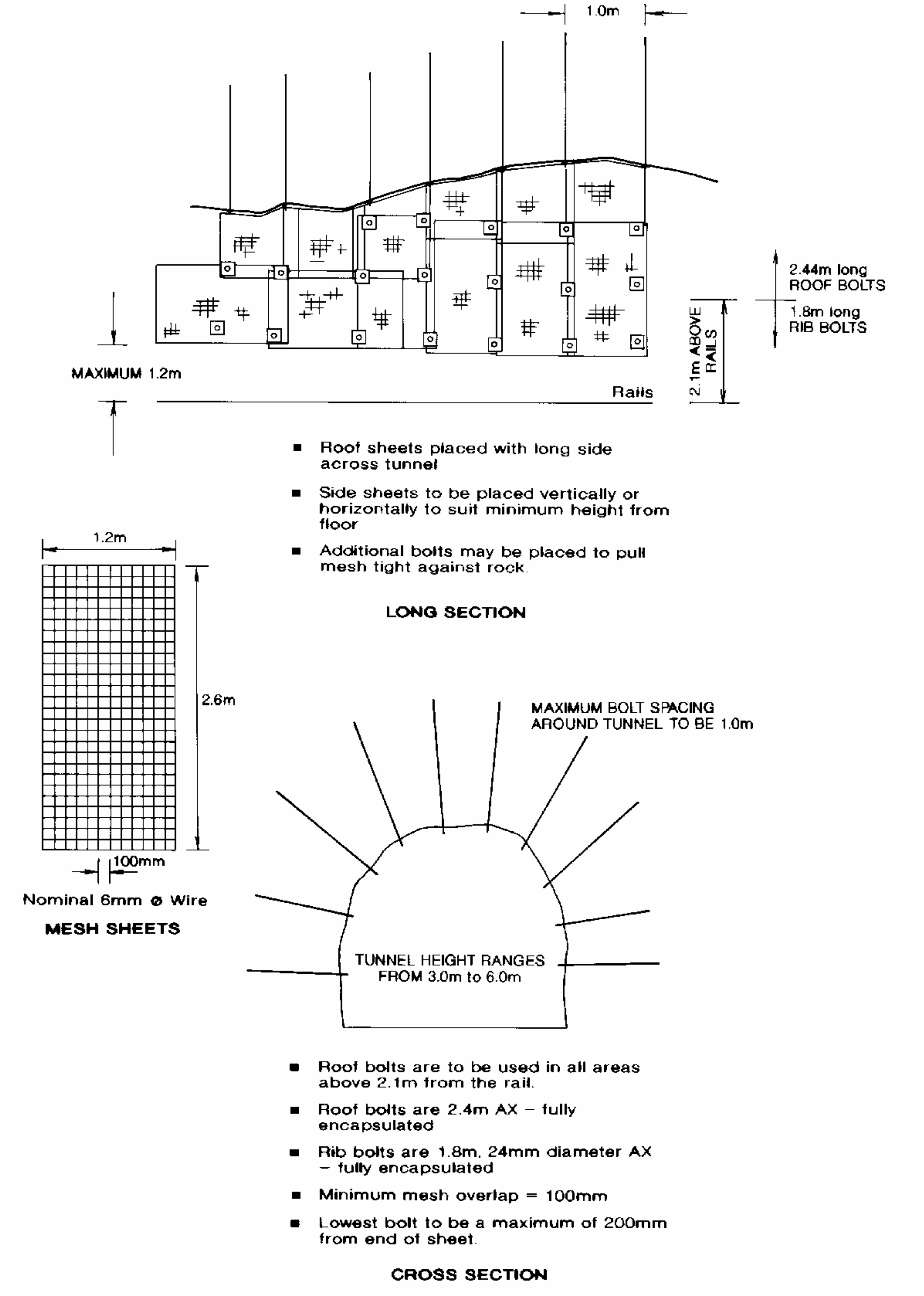


Figure 6 – CATARACT TUNNEL Bolting pattern.

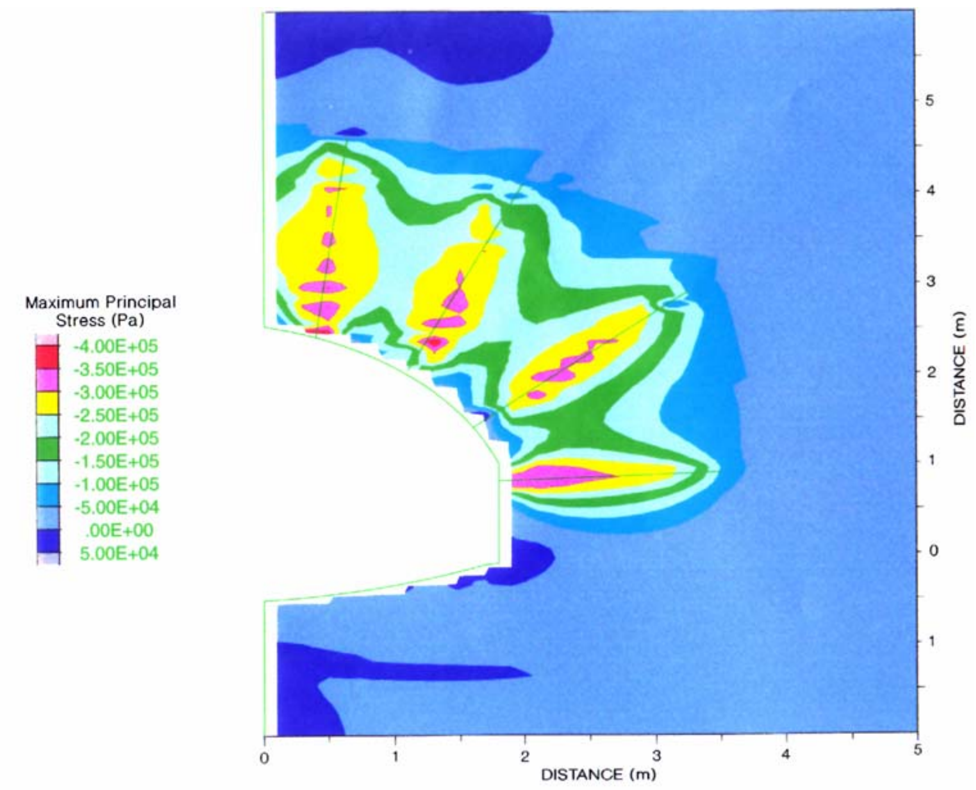


Figure 7 – CATARACT TUNNEL
Confining stress distribution developed by rock bolting system at 280kN force.

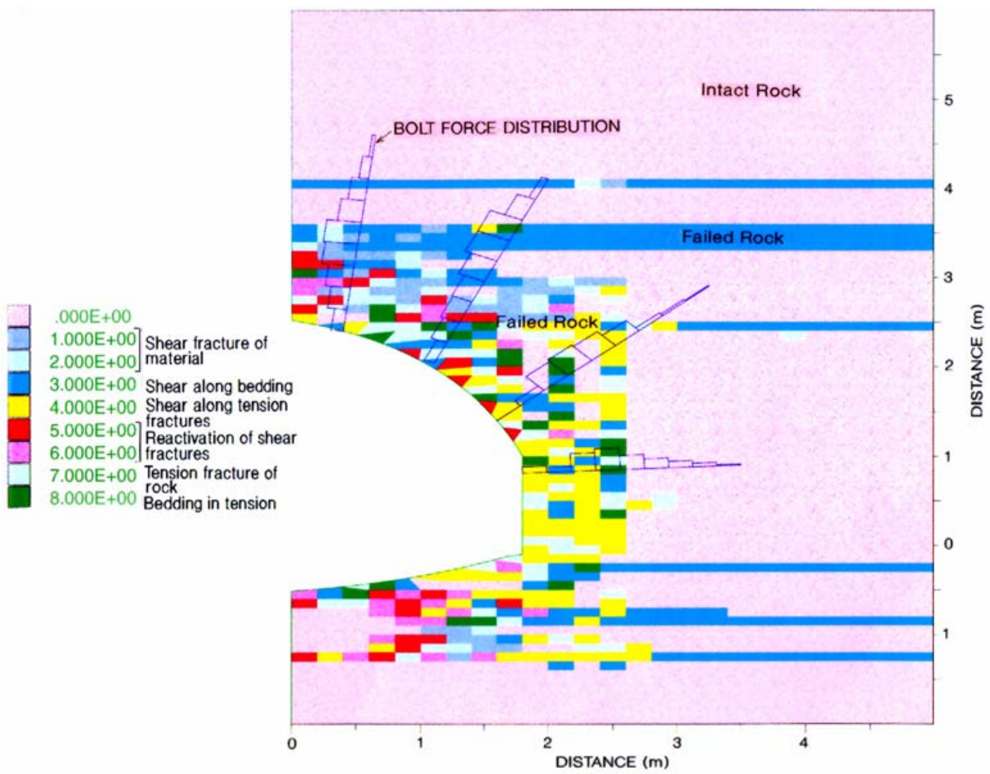


Figure 8 – CATARACT TUNNEL
Area of rock failure at 12MPa horizontal stress and coincident bolt force distribution.

7.4 Reinforcement Approach for Tensile Strain

The effect of tensile strain along and across the tunnel is likely to localise about pre-existing jointing. The potential effect of tensile strain was assessed on the basis of experience and the potential for loosening of blocks to fall. On the basis of the interim three dimensional modelling tensile stresses will only form perpendicular to the tunnel during the transient undermining.

Failure of the ground in areas of horizontal tensional strain would most likely involve collapse along pre-existing, steep, closely jointed zones for which the normal stress has been relieved.

Such zones would require jointing to be pervasive sub parallel to the tunnel and be of a spacing such that two planes bounding a potential block daylight into the tunnel.

The development of this type of failure would be considered to be unlikely in the majority of the tunnel where jointing is typically discontinuous.

7.5 Behaviour of High Roof Areas

An assessment of the stability of the areas for which rock falls had occurred subsequent to construction has been undertaken. These areas were overstressed during construction and the roof had fallen.

The behaviour of these areas was assessed in the same manner as the normal height areas, whereby horizontal stresses were increased above 12MPa.

The results indicated that additional roof failure initiated at approximately 11MPa. At stress levels above this, failure rapidly increased above the bolts and cable reinforcement would be required for stress levels above 12MPa.

The cable bolt pattern is recommended as a 2:1 pattern at 2m spacing.

Monitoring and regular inspection of these areas is required to assess this initial design, as stress variations outside those used in the analysis are possible.

8. FLOOR BEHAVIOUR

The strata in the floor of the tunnel was considered to have failed in a number of areas in the tunnel during the site inspections.

Buckling of the concrete inset was noted in site inspections and during undermining, additional failure is expected.

Shortening of the rail sections could cause local bucking in the compressive strain zones. These high compressive strain zones would be expected to require slotting or other methods to allow lateral shortening of the rail and ballast structures.

It is also possible that buckling of the floor strata may cause failure in the floor structure during active undermining. The effect on rail structures in the tunnel has not been assessed in detail.

9. MONITORING REQUIREMENTS

To confirm the adequacy of the proposed reinforcement in controlling the anticipated deformation levels, the following systematic program of monitoring is proposed. Analysis by M.A. Coulthard and Associates has indicated that it is not possible to predict the zones of highest compressive and tensile strains affecting the tunnel. This is due to the possibility of the subsidence deformation occurring in a series of layers in which the stress distributions induced by the subsidence could be complex.

The monitoring proposed covers the likely variability in performance by having a series of levels:

9.1 Large Scale Monitoring

- A series of convergence stations where the tunnel profile can be monitored will be established throughout the area of interest.
- Routine visual inspections of tunnel and tunnel reinforcement.

9.2 Local Intensive Monitoring

A series of seven sites have been selected along the tunnel in zones of expected highest deformation, either due to tunnel shape or position relative to expected subsidence. The locations of these sites selected to take into account:

- Results of the modelling.
- Results of the three dimensional modelling and its implications to the stress distribution.
- Influence of the transient longitudinal subsidence induced stresses.

10. REPAIR STRATEGY

Because of the complexity of the strata deformation due to the induced subsidence there is a possibility that the stress levels may locally reach levels which will induce high levels of deformation. Apart from the possible variations in the in situ stressfield around the tunnel beyond that used in the analysis, areas of adverse lithology or structure and atypical subsidence caused by undefined structural features above or below the tunnel horizon may result in localised areas of elevated stress.

With the proposed reinforcement, total failure of the tunnel is not anticipated, however failure around the skin of the roadway may lead to rupture of the restraining mesh and local block detachment. This type of failure may require selective scaling and re-bolting or in extreme cases further cable bolting.

The monitoring installed is designed to identify high deformation trends prior to reaching significant failure levels enabling local areas to have additional reinforcement placed. However, regardless of the level of reinforcement placed, it would be extremely difficult to prevent significant levels of rock failure occurring under the upper bound stresses which, could theoretically occur.

11. ACKNOWLEDGMENTS

The authors would like to recognise the contribution of Bruce Allen as well as Jim Hudson of Appin Colliery to the project. The contribution of Golder Associates for the geology, Mike Coulthard for the three dimensional modelling and Waddington and Associates for the subsidence predictions is gratefully acknowledged.

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