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Developments in the design of tunnels and caverns in the Triassic rocks of the Sydney region

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Abstract

Details are given of the analytical methods used to design rockbolt and shotcrete support for tunnels and large span caverns under relatively low cover in the near horizontally bedded Triassic sandstones of the Sydney region.

The paper provides a concise description of the engineering geology of the Sydney sandstones because it is fundamental to tunnel support design that a valid geological model be the basis of any analytical design. Equations are provided which allow calculations of the lengths, density and capacity of rockbolts for support in this geological environment. The paper also discusses the design approach adopted for support of tunnels and caverns which are at sufficient depth to generate compressive and shear failure of the rock mass, so-called True Rock Pressure.

1. Introduction

This paper traces what the writer and his co-workers have learned in regard to the design and construction of caverns and tunnels in the Sydney region over the past two decades. This work is quite specific to the near horizontally bedded Triassic sandstones and shales of the area, and comprises the construction of some of the world's widest span near surface caverns, all without passive support. These include the donut shaped cavern for the Sydney Opera House underground car park, with a span of 17 m and only 6 m of rock cover (see Figs. 1 and 2), and the 24 m wide section of the double deck Eastern Distributor tunnel (see Figs. 3 and 4).

This paper contains no truly new concepts but seeks to show how facets of the science of rock mechanics have been appropriately linked to a particular geological environment to provide a design process which is primarily scientific and not simply educated guesswork (often called 'art'). In this regard the writer subscribes wholly to the following quote from Petroski [1]

> The conception of a design of a new structure can involve as much a leap of the imagination and as much a synthesis of experience and knowledge as any artist is required to bring his canvas or paper. And once that design is articulated by the engineer as artist, it must be analysed by the engineer as scientist in as rigorous an application of the scientific method as any scientist can make.

The first step in this process is to present the geological setting, because a valid geological model is the starting point of tunnel design.

2. The geological setting

2.1. Regional

Rock engineering works in the Sydney region relate largely to the Triassic sandstones and shales that underlie most of the metropolitan area. The Triassic stratigraphy comprises four major divisions, namely:

Wianamatta Group	maximum thickness 300 m, consisting mainly of shales and siltstones
Mittagong Formation	a thin horizon (<20 m) of interbedded shales and sandstones
Hawkesbury Sandstone	maximum thickness 290 m, predominantly near-horizontally bedded sandstone but with some laminite beds

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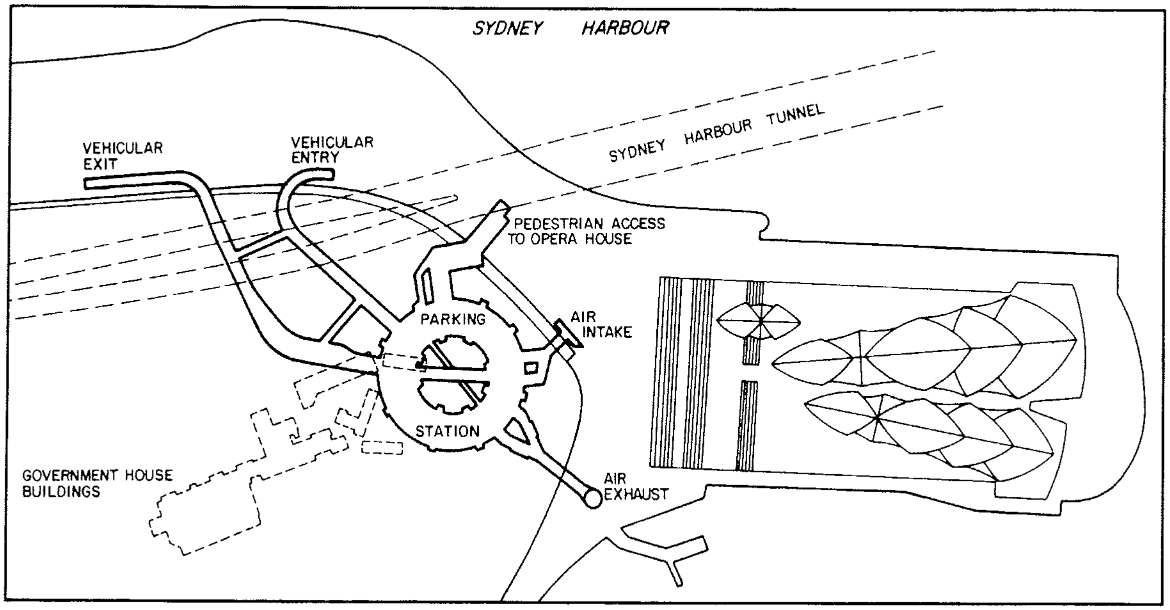


Fig. 1. Location plan of Sydney Opera House underground parking station.

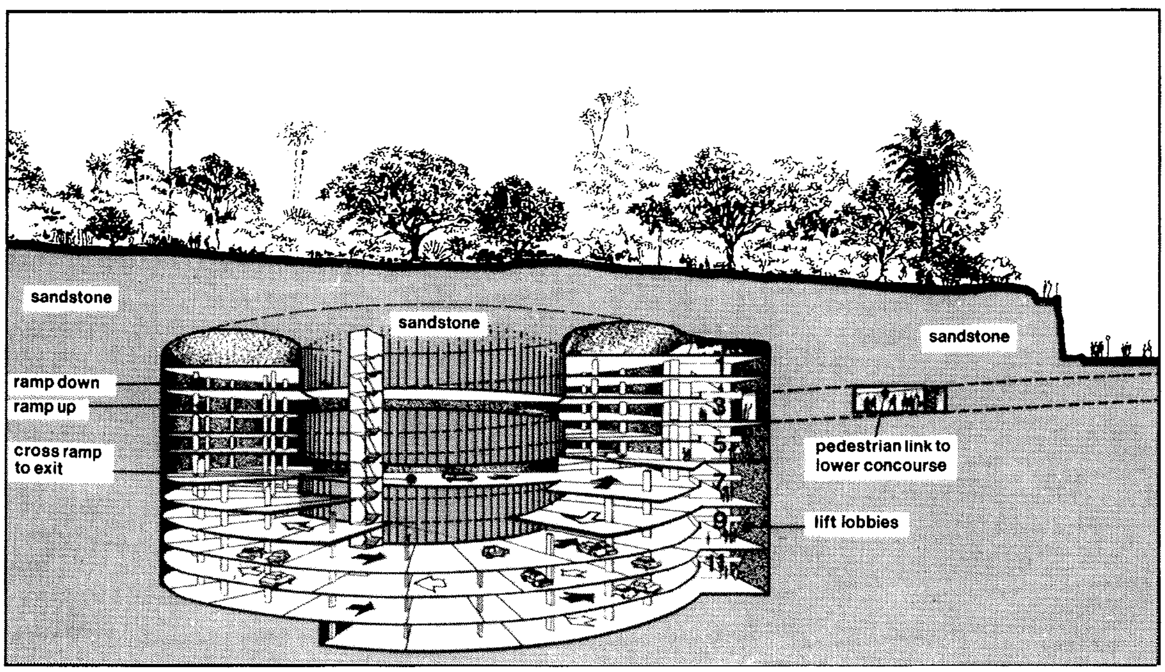


Fig. 2. Artistic impression of Opera House underground parking station.

Narrabeen Group	maximum thickness 700 m, comprising sandstones with claystone horizons

As shown in Fig. 5 the Wianamatta Group shales and the Hawkesbury Sandstone are the dominant near surface rocks, and because the shales form an eroded cap it is in the Hawkesbury Sandstone that most of the structures discussed in this paper are located.

2.2. *Engineering geology of the Hawkesbury Sandstone*

When viewed in vertical section the Hawkesbury Sandstone may be divided into three facies, namely:

sheet facies	
massive facies	
mudstone facies	$<5\%$ of formation

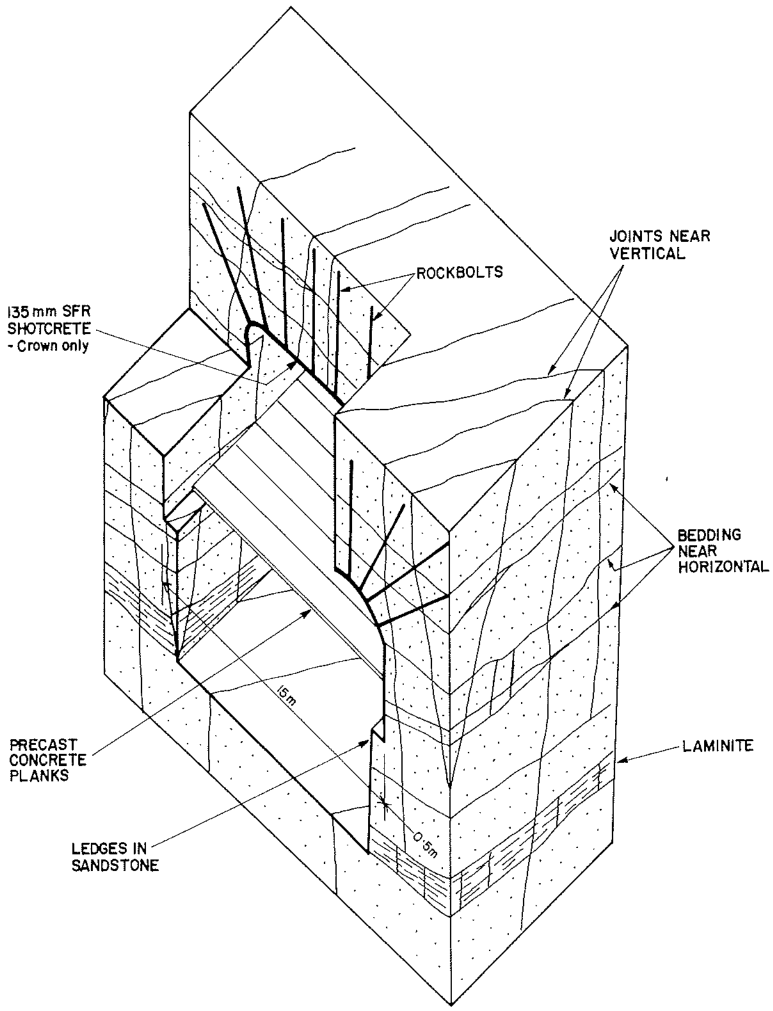


Fig. 3. Double deck tunnel of the 3 km long Eastern Distributor.

The sheet facies comprises sets of cross-bedded strata bounded by planar near horizontal surfaces, see Fig. 6. The units range in thickness from fractions of a metre to greater than 5 m but are typically of the order of a metre. The near horizontal bedding planes give this facies a sheet-like appearance when viewed from a distance. The cross-beds typically dip to the northeast.

The term massive facies was coined to covey the gross aspect when viewed from a distance and does not mean wholly structureless at closer inspection. Frequently, sandstone bodies of this facies have a discordant erosional lower surface and a planar concordant upper surface. Mudstone (or shale) breccia commonly occurs within troughs at or above the basal surface, but clasts, and in particular mudchips and mudflakes, can occur dispersed throughout.

Petrographic analyses indicate that typically the Hawkesbury Sandstone has the following composition:

detrital quartz grains	50–75%
lithic fragments	2–4%
clay matrix	15–30%
secondary quartz	4–10%
siderite (iron carbonate)	2–4%

The average composition of the matrix clay is 55–75% kaolinite, 20–30% illite and the balance mixed-layered clays.

2.2.1. Strength

The sandstone comprises medium grained quartz grains; quartz overgrowth of the grains frequently provides an interlocking structure and the development thus of crystal faces imparts a glistening effect in fractured faces.

Substance strength properties (nominally 50 mm diameter core) of fresh or slightly weathered sandstone are typically:

$$\text{Saturated unconfined compressive strength (UCS)} = 25-45\ \text{MPa},$$

$$\text{Saturated Brazilian tensile strength} = 2-3\ \text{MPa}.$$

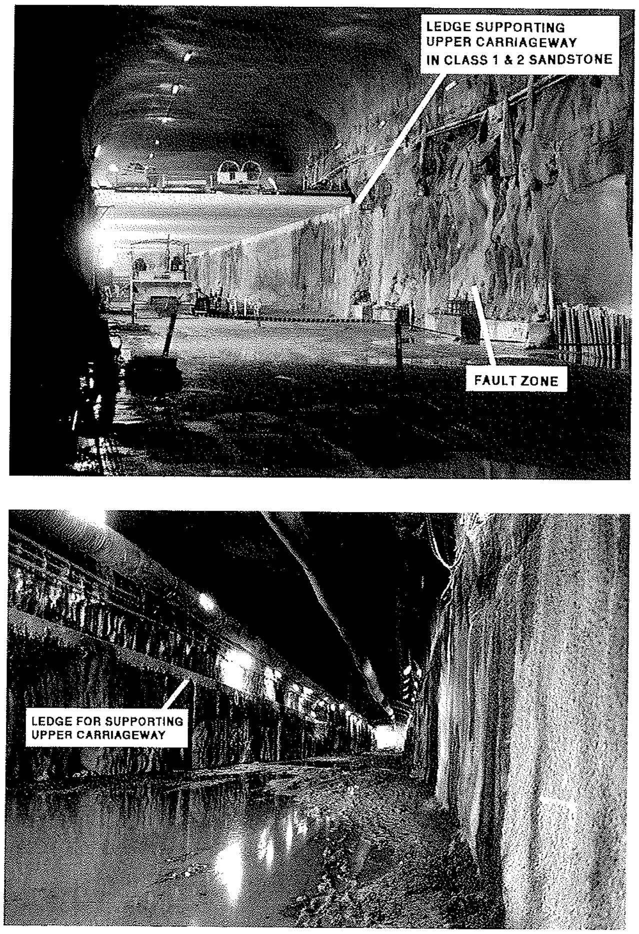


Fig. 4. The Eastern Distributor double deck tunnel during construction.

The strength of the material oven-dried is about 1.5–2.0 times the saturated strength. Intact Young's Modulus values range from about 2.5 to 8 GPa, indicating a low to average modulus ratio.

An important question is what substance strength to use at the "tunnel scale" when evaluating the potential for stress induced failures. The approach taken by the writer is based on the view that around excavations the rock volume potentially subject to high induced compressive stresses is of the order of bedding plane and joint spacings, i.e. 0.5–2 m. Based on the data on size effects on rock strength summarised by Hoek and Brown [2] a reduction factor of about 0.6 is adopted with respect to 50 mm diameter core strength. This means that in practice a field scale strength of about 20 MPa is adopted for the fresh Hawkesbury Sandstone.

2.2.2. Bedding discontinuities

Two main forms of bedding discontinuities are present (see Fig. 6).

Facies bedding. Major depositional horizons represented by near horizontal, undulating, bedding discontinuities which have a typical spacing of between about 1 and 2.5 m. These may be continuous for hundreds of metres and are marked by continuous partings, clay seams or petrographic changes. Local increase in the dip of the facies bedding occurs where sand has been deposited in channel structures.

Clay seams, typically between 5 and 25 mm thick, are very common within the sequence. The origin of these

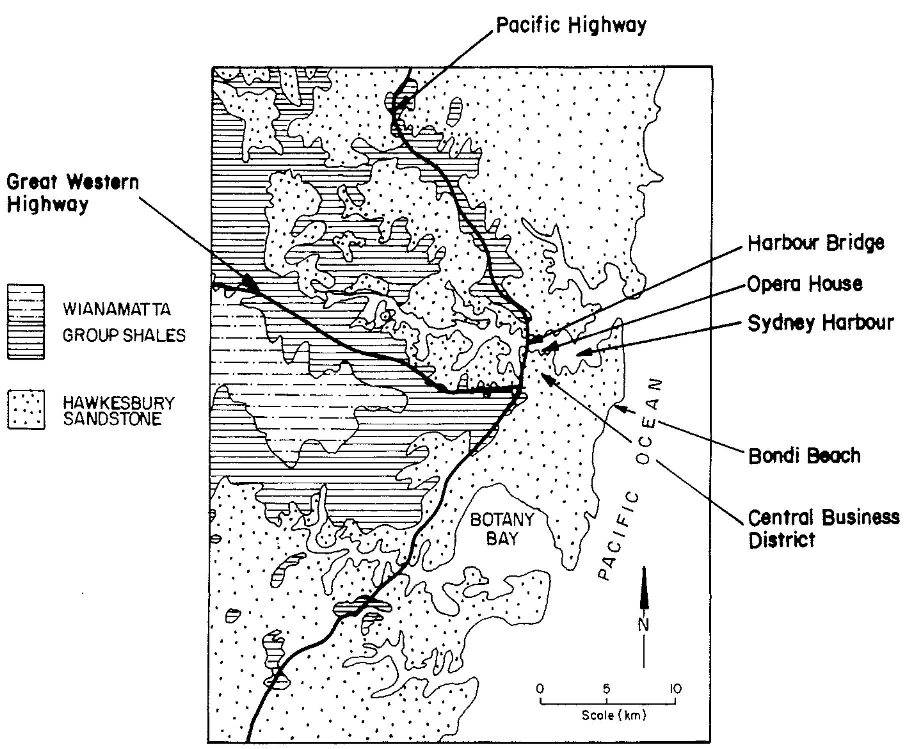


Fig. 5. Triassic geology of the Sydney region.

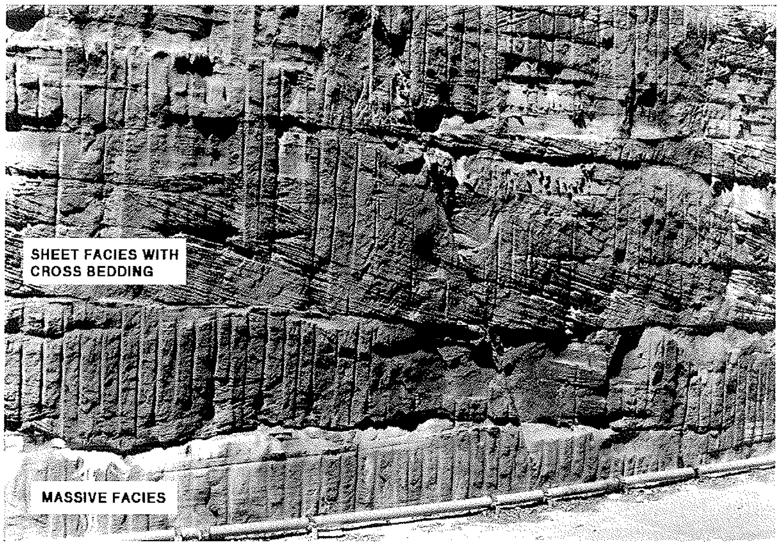


Fig. 6. Typical exposure of Hawkesbury Sandstone.

seams is not clearly understood but they provide the major weakness within the Hawkesbury Sandstone sequence.

Cross bedding. Cross bedding (also termed current bedding) is an ubiquitous feature of the sheet facies. Cross bedding planes are often marked by the deposition of flakes of mica, graphite, and carbonaceous matter. The cross bedding usually does not represent planes of weakness in fresh or slightly weathered sandstone. However, in moderately to highly weathered sandstone the cross beds can form surfaces of relatively low tensile and shear strength ($\phi' \approx 30-35°$).

2.2.3. *Jointing*

The dominant joint set strikes NNE with dips ranging from 65° to 90° east or west, depending on location across the city. A secondary, orthogonal, set comprises near vertical joints.

The joints have substantial horizontal continuity (typically greater than 20 m). Their vertical continuity is variable. Many of the joints terminate on bedding horizons and may have vertical continuities of the order of 5 m. However, about 30% of the joints transgress several bedding horizons and have vertical continuities of between 10 and 30 m. Spacings of the dominant NNE joint set range between less than 0.3 m to about 5 m, with an average of about 1.5 m. A pattern frequently observed within the Sydney area is that the joints may occur in swarms. This means that anywhere between three and ten joints may occur over a distance of a few metres, with there then being a substantial gap before encountering further joints of the same set.

The orthogonal joints (ESE) have similar continuities to the main set but with spacings in the range 6–20 m.

RQD values in the fresh or slightly weathered rock typically lie in the range of 75–100%.

2.2.4. *Faulting*

Faulting in the Hawkesbury Sandstone is relatively rare [3] but there are at least three 20–40 m wide zones comprising high angle normal faulting, oriented NNE, running through the central business district. The zones are spaced at between 300 and 600 m. In addition, low angle thrust faults, substantially confined to bedding horizons, are also found.

The above structures play little part in the rock mechanics discussed in this paper but serve as a warning that the Hawkesbury Sandstone is not always a benign medium.

2.2.5. *Regional stress field*

Measurements of the natural stress field have been made on many projects in the Sydney region using hydrofracture, rock slotter and strain cell overcoring techniques. The results of these measurements have been presented in several papers including Enever et al. [4], Enever [5] and McQueen [6]. Unfortunately, many of the stress measurement compilations combine results from different geological units and include results obviously affected by topography.

The writer considers that, within the Hawkesbury Sandstone to a depth of about 150 m, the following equations represent an appropriate expression of the natural total stress field:

$$\sigma_1 = \sigma_{NS} = 1.5 + 1.2\sigma_v \text{ to } 2.0\sigma_v \text{ MPa}, \quad (1)$$

$$\sigma_2 = \sigma_{WE} = 0.5\sigma_1 \text{ to } 0.7\sigma_1 \text{ MPa}, \quad (2)$$

$$\sigma_3 = \sigma_v = 0.024H \text{ MPa}, \tag{3}$$

where σ_v is the vertical stress, σ_{NS} the horizontal stress oriented approximately 20° East of true north, σ_{WE} the orthogonal to σ_{NS}.

Enever [5] has proposed a step function in the stress field at a depth of 20 m. This does not make sense from a geological viewpoint and is not adopted by the writer and his co-workers.

2.2.6. Hydrogeological parameters

The substance permeability of the sandstone is in the range of 10^{-9}–10^{-11} m/s. Mass permeability is governed by joints and bedding horizons. Analysis of the results of some 2240 m of Lugeon permeability tests for the Ocean Outfall Tunnels, the Eastern Distributor, the Chatswood–Parramatta rail tunnels, the Cross City Tunnel and four electric cable tunnels gives the following results:

Lugeon value	Cumulative length of borehole	Percentage of total length (%)
<0.1	620	28
0.1–1	800	36
1–3	277	12
3–10	208	9
10–25	200	9
25–100	78	3
>100	53	2

The log mean permeability from these data is 0.48 Lugeon (i.e. $k = 5 \times 10^{-8}$ m/s). Experience from monitoring seepages of tunnels and deep basements below the water table is that average mass permeability is in the range 3×10^{-7}–5×10^{-8} m/s. Occasional open joints produce moderate seepage flows (0.2–1 l/s), but typically only for a few days because of the low storage characteristics of the mass sandstone.

The regional groundwater table follows a muted reflection of the topography, being at sea level along the harbour and at a depth of 5–8 m below the ridges.

2.2.7. Effects of weathering

The parameters presented in Sections 2.2.1–2.2.6 apply primarily to fresh or slightly weathered sandstone. The horizon of extremely (EW) to moderately weathered (MW) rock typically extends to a depth of between 5 and 15 m. The following is typically apparent with increased degree of weathering:

- UCS decrease to <20 MPa for MW, <10 MPa for HW and <2 MPa for EW,
- RQD decrease to 40–70% for MW, 10–40% for HW and <10% for EW, and
- increase in the frequency and thickness of near horizontal seams of extremely weathered material along facies bedding horizons.

However, it should be noted that in some zones, weathering causes strength increase due to the conversion of siderite (iron carbonate) to iron oxide. Therefore geological categorisation of weathering is not always an indicator of engineering properties.

3. Classification systems and their limitations

3.1. The Sydney system for sandstone and shale

Before discussing experience with the Q-system and RMR system in the Sydney environment, it is appropriate to describe a classification system used in Sydney since 1978 [7,8]. This system was developed specifically for assessing design parameters for heavily loaded foundations on the sandstones and shales. It was never intended for use in tunnel design. However, with the passage of time it has proven to be a very valuable tool for rapid communication between investigators, designers and contractors of information on mass quality of the sandstones and shales. The widespread adoption of the system within the Sydney area has demonstrated that it encapsulates the key features which affect engineering performance of the rocks. It is a five-class system with Class I being the best quality. The system is set out in Table 1 and is based on:

- substance unconfined compressive strength,
- degree of fracturing, usually assessed from core, and
- the percentage cumulative thickness of sub-horizontal clay seams within the zone being assessed.

The lowest rating of any factor defines the class. Thus, for example, an 8 m horizon of sandstone selected as being reasonably consistent, with the parameters

- substance UCS = 10 MPa,
- slightly fractured (equivalent to RQD > 75),
- cumulative 50 mm of clay seams over 8 m (i.e. 0.6%)

would classify as Class III because the UCS controls. If the UCS were > 24 MPa the zone would classify as Class II because the degree of fracturing would control.

3.2. Classification using Bieniawski's RMR system

While every site is different, one can obtain a good idea of how the range of Sydney sandstones classify in the RMR system [9] by analysing each of the major classes given in Table 1. The results of this process are given in Table 2 for a north–south oriented tunnel at a depth of between 20 and 50 m (equivalent to the Eastern Distributor tunnels).

The data in Table 2 show that:

- the RMR system is very insensitive to the intact strength, a parameter which is very important in the engineering behaviour of the Sydney sandstone,
- the best sandstone classifies at the bottom of Bieniawski's Class 2, indicating a fair to good rock mass—which is very conservative given the spectacular unsupported vertical cuttings and unsupported openings made in this rock, and
- the system does not discriminate well between the sandstone grades encountered in the Sydney Basin.

Table 1
Engineering classification of shales and sandstones in the Sydney region—a summary guide

Class	Unconfined compressive strength q_u (MPa)	Defect spacing (mm)	Allowable seams (%)[a]
Classification for sandstone			
I	>24	>600	<1.5
II	>12	>600	<3
III	>7	>200	<5
IV	>2	>60	<10
V	>1	NA	NA
Classification for shale			
I	>16	>600	<2
II	>7	>200	<4
III	>2	>60	<8
IV	>1	>20	<25
V	>1	NA	NA

[a] Allowable seams: seams include clay, fragmented rock and highly to extremely weathered zones, usually sub-horizontal. The limits suggested in the tables relate to a defined zone of influence. For pad footings, the zone of influence is defined as 1.5 times the least footing dimension. For socketed footings, the zone includes the length of the socket plus a further depth equal to the width of the footing. For tunnel or excavation assessment purposes the defects are assessed over a length of core of similar characteristics.
The classification system is based on rock strength and defects using three parameters as set out below. The lowest rating of any one factor defines the class.

3.3. Classification using the Q-system

In accord with the approach set out in Section 3.2 for the RMR system, Table 3 gives the Q-system classification [10,11] for the range of Sydney sandstone, again assuming a north–south oriented tunnel at a depth of 20–50 m.

Unlike the RMR system, the Q-value classification provides good discrimination of the range of mass properties of the sandstones.

3.4. Limitations for support design

A 1997 review [12] of the adopted support in five major tunnelling projects in the Hawkesbury Sandstone, compared with that indicated by the RMR and Q-systems, reached the following conclusions:

1. The RMR system provides poor discrimination of the range in rock mass quality, and only provides support guidelines for nominally 10 m span tunnels at shallow depth; hence the system has little practical value for tunnel design in the Sydney region.
2. The Q-system predicts a significantly lesser level of support than actually adopted in the five cases studied.

At the time of the 1997 study all that could be said about the Q-system guidelines was that either the designs adopted in the Hawkesbury Sandstone had been conservative or the guidelines were potentially dangerous. Since then information has become available from crown collapses in a storage cavern project, which suggests the latter to be true. For legal reasons full details of these collapses cannot be given at this time. Suffice to say that the span was about 13 m, overburden cover about 125 m and the Q-value designated from the site investigation boreholes was 24. The original Q-system recommendations [10], for ESR = 0.8, would place this structure in Support Category 14 and would require tensioned bolts at 1.5–2.0 m spacings at lengths of 3, 5 and 7 m plus chain link mesh. The updated (1993/1996) recommendations [11] would require systematic

Table 2
Classification of typical range of Sydney sandstone using RMR

Item	Parameter	Sandstone class according to Sydney system				
		I	II	III	IV	V
1	Intact strength	2	2	2	1	0
2	RQD	18	17	13	5	3
3	Discontinuity spacing	15	15	12	10	8
4	Discontinuity condition[a]	25	22	20	10	10
5	Groundwater	10	8	8	8	8
6	Adjustment for orientation	−5	−5	−5	−5	−5
	Total RMR	65	59	50	29	24
	Class	2	3	3	4	4
	Description	Good	Fair	Fair	Poor	Poor

[a] This is difficult to assess because of the need to define the relative priorities of continuity, roughness, joint infill and joint wall condition.

Table 3
Typical classification of range of Sydney Sandstones using NGI Q-system (<50 m depth)

Item	Parameter	Sandstone class according to Sydney system				
		I	II	III	IV	V
1	RQD	90	80	65	25	5
2	J_{n^*}	2	4	4	6	12
3	J_r	3	3	1.5	1	1
4	J_a	1	1.0	2.0	3.0	6.0
5	J_w	0.8	0.8	0.8	0.66	0.66
6	SRF[a]	2.5	2.5	5.0	5.0	5.0
Q-value		43	19	2	0.18	0.009
Description		Very good	Good	Fair	Very poor	Exceptionally poor

[a] Practitioners in Sydney vary in their assessment of this parameter.

bolting at 2.6 m spacing with bolt lengths of 4.5 m (no mesh, no shotcrete). The actual support was similar to the original recommendations and heavier than the updated guidelines. The first collapse comprised about 300 tonne of crown, the second about 20 tonne. After the collapses the support was increased to substantially greater density and capacity than would be indicated by the Q-system.

It may be argued that the Q-value adopted from the site investigation boreholes was wrong. The writer has reviewed this matter using logging of the actual excavation. This suggests a Q-value of about 13 (i.e. half the original value). However, the support recommendations would be the same.

In early 2000 a major system of TBM driven tunnels was completed on the north side of Sydney Harbour to store peak sewage flows. The scheme included about 6.5 km of 3.8 m diameter tunnel, 3.7 km of 6.0 m diameter tunnel and 3.5 km of 6.3 m diameter tunnel. All the tunnels were in Hawkesbury Sandstone, with depths of cover ranging from about 20 m to about 80 m. The initial primary support, comprising rockbolts and mesh, was designed using the Q-system. The actual density of rock bolting (bolt per metre) which proved necessary to install, following inadequate performance of the initial design, ranged between 5 and 9 times the initial design densities.

Based on the information given above it is the practice of the writer, and co-workers, not to rely on the recommendations of the Q and RMR systems for support design in the Hawkesbury Sandstone. The approaches which are used are set out in the remainder of this paper.

4. Adopted support design methodology

4.1. Basis of design

Some 30 years ago when, as a young civil engineer, the writer joined Bieniawski's rock mechanics group at the CSIR to work on tunnel design, the most striking personal discovery was that there appeared to be no appropriate analytical design methods for rock tunnel support, certainly not as understood in the fields of structural engineering, hydraulic engineering and even soil mechanics. There was a lot of talk about the 'art' of tunnel design, but on close examination much of this seemed to be educated guesswork. This is probably why those working in rock tunnel support design enthusiastically, and somewhat uncritically, adopted the RMR and Q classification systems when they appeared in 1973 and 1974.

The writer accepts that tunnel design is different from many other engineering design processes. However, it can be performed on a scientific basis using an intimate blend of engineering geology, precedent, structural analysis and the observational method during construction.

Fortunately, the relatively simple geology of the Hawkesbury Sandstone has facilitated the development of analytical design methods over the past two decades, methods which have been demonstrated to work well in rail, road and water tunnels, and major caverns. Before proceeding to discuss the analytical methods currently in use it is critically important to reiterate Lauffer's [13] categorisation of tunnel support loadings, namely:

(i) *Loosening pressure*: forces or pressures caused by the weight of blocks or prisms of loosened or potentially loosened rock in the crown and sidewalls of a tunnel.
(ii) *Swelling pressure*: caused by volumetric increases of clays, claystones or other rocks due to exposure to the atmosphere under altered stress conditions.
(iii) *True rock pressure*: arising when compressive stresses generated in the rock around the tunnel are sufficient to cause compressive yielding and fracture.

Swelling pressures are not an issue in Sydney's Triassic rocks. Loosening pressures are the prime consideration at depths of less than about 50 m, and are discussed in Section 4.2. True rock pressure creates a much more difficult design problem and is discussed in Section 4.3.

4.2. Design for loosening pressures

Traditionally, loosening pressures have been calculated using the Terzaghi [14] support loadings, or some modification thereof. These have no meaning where support in jointed rock is provided by anchors, dowels, mesh and shotcrete. With these forms of support the key concept is to retain the integrity of the rock mass, which will then support itself. In applying this concept to the near horizontally bedded, steeply jointed Hawkesbury Sandstones it is appropriate to invoke the concepts of linear arch theory published by Evans [15].

Cutting an arch-shaped crown in this type of rock mass is counterproductive because this creates unnecessary 'cantilevers' of rock and negates the positive aspects of a relatively high horizontal stress field (see Fig. 7). Linear arch theory clearly shows that spans in excess of 15 m can easily be created in sandstone of Class III or better (see Table 1) provided the effective bedding thickness is greater than 5 m (see Fig. 8). The problem is that real bedding spacings are between 0.5 and 2 m (see Fig. 6). The solution of this problem is to provide internal reinforcing to the rock so as to artificially create the requisite effective bed thickness. The analytical process whereby this is achieved is set out below. In essence it is an extension of the theory of reinforcement of a laminated crown beam by friction effects as set out in Chapter 20.3 of Obert and Duval [16].

The requisite thickness of the linear arch can be determined using the modified version of the original Evans linear arch theory as given by Sofianos [17], or by a more sophisticated version as developed by Booker [22]. This requires a decision to be made on what is the allowable crown sag, and cognisance must be taken of:

- the mass modulus of the crown strata and the effective abutment stiffness, and
- the natural stress field.

Because, these variables are usually not known precisely it is appropriate to do a parametric study to develop an understanding of what factors are critical to design. Fig. 8 shows the typical results of such calculations for a bolted crown beam in Class II Hawkesbury. It can be seen that for a 15 m span (the example given in Fig. 9) an effective linear arch thickness of 5 m is required if crown sag is to be less than 20 mm.

4.2.1. Bedding plane shear

In linear arch theory the steeply dipping joints play no substantial role in a crown stability. If shear along near horizontal bedding planes can be limited to approximately that which would occur in a thicker elastic beam then the effective thickness of the linear arch is not controlled by physical bedding plane spacings. Hence the fundamental tenet of the analytical design process is to limit shear displacements on bedding horizons within the desired thickness of the linear arch.

In order to implement this procedure two initial sets of calculations have to be made, namely:

1. Calculation of the probable bedding plane shear displacement that would occur at an acceptable maximum crown sag, if the crown rock were unreinforced. This can be done using a jointed finite element model. The results of such an analysis are given in Figs. 9b and c for the example tunnel shown in Fig. 9a. If only horizontal bedding discontinuities are considered then a closed form solution can be

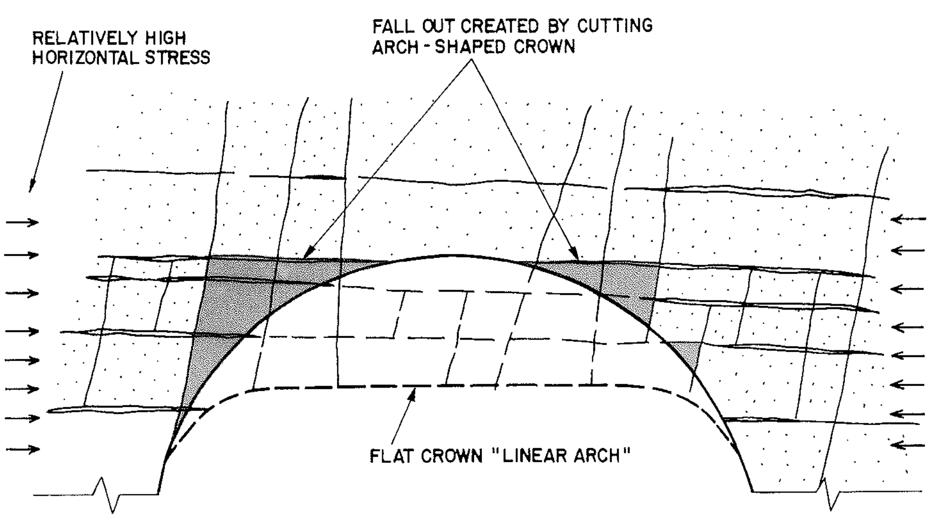


Fig. 7. Geometric disadvantages arising from cutting arch crown in Hawkesbury Sandstone.

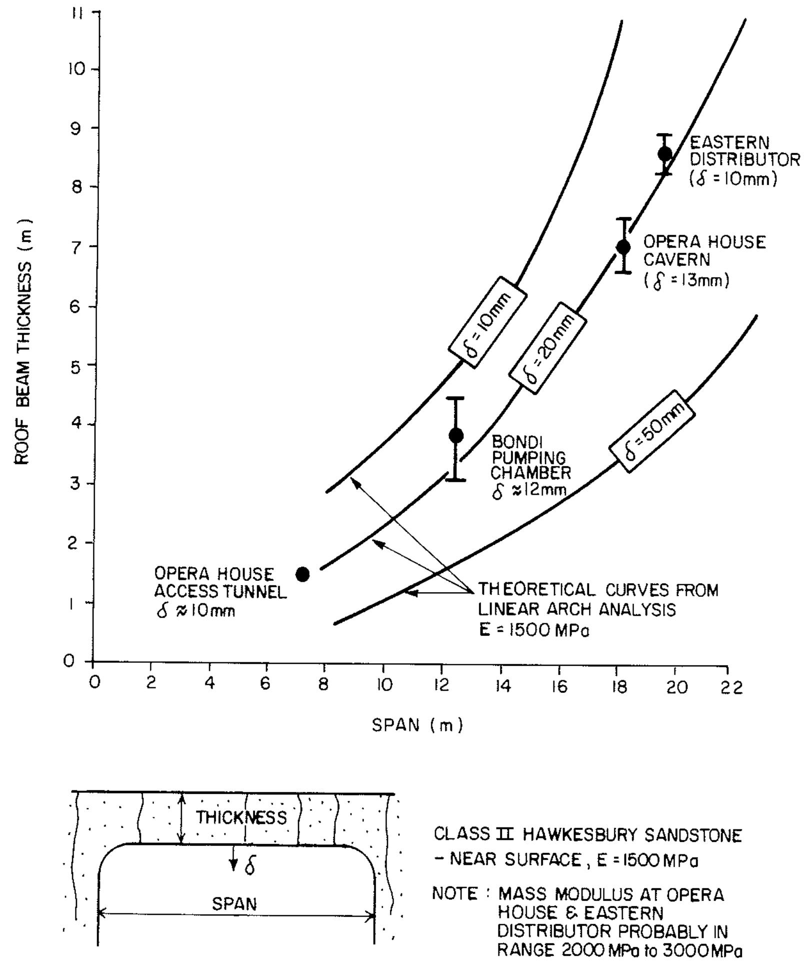


Fig. 8. Theoretical design curves from linear arch analysis with examples from Sydney projects.

used to estimate these shear displacements, as discussed by Bertuzzi and Pells [24].

2. Calculate the shear stresses which would occur at the locations of physical bedding horizons if behaviour were purely elastic. This can be done using the same finite element model but with elastic bedding plane behaviour. The results for the simple example are given in Fig. 9d. Again, if only horizontal bedding discontinuities are considered, a closed form solution can be used for these shear stresses [24].

Once the process of calculating the bedding plane shear displacements and shear stresses is completed as per (i) and (ii) above, attention can be turned to calculating the rock bolt capacities, orientations and distributions required to create the effective linear arch. This process is set out in Section 4.2.2.

4.2.2. Calculating rock bolting requirements

At the outset it should be noted that consideration is given here only to fully grouted rockbolts. These are typically so far superior to end anchored bolts in their influence on rock mass behaviour that the latter are only used for local support of isolated loosened blocks of rock.

Fig. 10 shows the general case of a single rockbolt crossing a discontinuity. The reinforcement acts to increase the shear resistance of the joint by the following mechanisms:

1. an increase in shear resistance due to the lateral resistance developed by the rockbolt via "dowel action"—force R_1,
2. an increase in normal stress as a result of prestressing of the rockbolt—force R_2,
3. an increase in normal stress as a result of axial force developed in the rockbolt from dilatancy of the joint—force R_3, and
4. an increase in normal stress as a result of axial force developed in the rockbolt from lateral extension—force R_4.

The first component can be considered as increasing the cohesion of the joint, while the other three components increase the frictional component of interface strength by increasing the effective normal stress on the interface. If the rockbolts are at a spacing S, so that

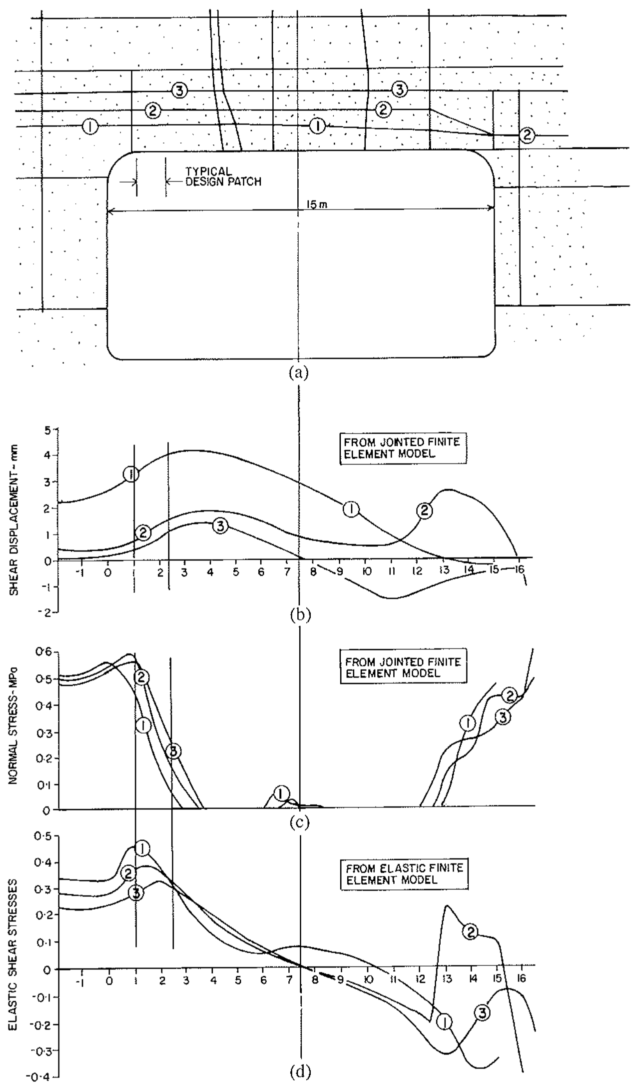


Fig. 9. (a) Example tunnel, (b) shear displacements, (c) normal stresses, (d) elastic shear stresses.

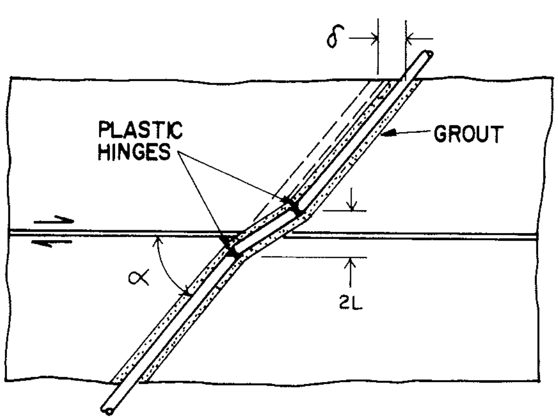


Fig. 10. Single full column grouted rockbolt across a joint.

each anchor affects an area, S^2, the equivalent increases in cohesion, Δc, and normal effective stress, $\Delta\sigma_n$, are as follows:

$$\Delta c = \frac{R_1 + R_5}{S^2}, \tag{4}$$

$$\Delta\sigma_n = \frac{R_2 + R_3 + R_4}{S^2}. \tag{5}$$

As a result of the anchors, the equivalent strength of the joint, s_j, will be as follows:

$$s_j = (c_j + \Delta c) + (\sigma_{n0} + \Delta\sigma_n)\tan\phi_j, \tag{6}$$

where c_j is the effective cohesion of joint, ϕ_j the effective friction angle of joint, σ_{n0} the initial effective normal stress on joint, Δc the equivalent increase in effective cohesion (Eq. (4)), $\Delta\sigma_n$ the equivalent increase in effective normal stress (Eq. (5)).

Methods of calculating forces R_1, R_3 and R_4 as a function of joint shear displacement are set out in Sections 4.2.2.1 and 4.2.2.2. Force R_2 is created by the initial bolt pretension.

4.2.2.1. Calculation of dowel action: force R_1. Calculations of dowel action is based on laboratory test data and theoretical analyses presented by Dight [18]. The experimental data showed that:

- plastic hinges formed in the fully grouted rockbolts at small shear displacements (typically <1.5 mm); these plastic hinges occur only a short distance on either side of the joint.
- crushing of the grout, or rock (whichever was the weaker) occurred at similar small displacements.

Based on his experiments, on plastic bending theory, and Ladanyi's [19] expanding cylinder theory, Dight developed equations for calculating the 'dowel' force R_1. For the simplified assumptions of grout strength equal to or less than the rock, and for the joint having no infill, the equations are

$$R_1 = \frac{D^2}{4}\sqrt{1.7\sigma_y}\,P_u\,\pi\left(1 - \frac{R_2}{T_y}\right)^2, \tag{7}$$

where

$$P_u = \left[\frac{\delta}{K(\pi D + 2\delta)}\right]^{A/2} \tag{8}$$

and

$$K = \sigma_c\left(\frac{1 - \nu^2}{E}\right)\ln\left(\frac{\sigma_c}{2P_0 - \sigma_t}\right) + \frac{\sigma_c}{(2P_0 - \sigma_t)} \times \left[\frac{2\nu(P_0 - \sigma_t) - \sigma_t}{E}\right] \tag{9}$$

and

$$A = \frac{2\sin\phi}{1 + \sin\phi}, \tag{10}$$

where σ_t is the tensile strength of the rock, σ_c the compressive strength of the rock, E the modulus of the rock, ϕ the friction angle of the crushed rock, T_y the yield strength of the rockbolt, ν the Poisson's ratio of the rock, R_2 the initial tension in the bolt, P_0 the initial stress in the rock in the plane of the joint (assumed equal all round the bolt), δ the shear displacement on the joint.

Eqs. (7)–(10) can be solved, using a MathCAD routine or a spreadsheet, to give a relationship between joint shear displacement and the dowel action resistance R_1.

4.2.2.2. Calculation of axial forces due to dilation and lateral rockbolt extension: forces R_3 and R_4. Calculation of forces R_3 and R_4 is based on observations in experimental tests by Dight [18], Pells [20] and Pellet and Egger [21] that, for fully grouted rockbolts, axial bolt yield is attained at a joint after very small shear or joint opening movements (typically <1.5 mm).

The distance to the plastic hinges from the joint surface is given approximately by the equation

$$L = \frac{D}{4}\sqrt{\frac{1.7\pi\sigma_y}{P_u}}\left[1 - \frac{R_2}{T_y}\right]^2. \tag{11}$$

If the assumption is made that, under small shearing displacements, axial strain in the rockbolt is dominantly between the two plastic hinges, then from the geometry given in Fig. 10 the equations for R_3 and R_4 are, as a function of shear displacement:

$$R_3 = E_s\left(\frac{\delta \tan i}{2L}\right)\frac{\pi D^2}{4}, \tag{12}$$

$$R_4 = E_s\left(\frac{L - \sqrt{L^2 - (\delta/2)^2}}{L}\right)\frac{\pi D^2}{4}, \tag{13}$$

where E_s is the modulus of the rockbolt material (steel), i the joint dilatancy angle.

If the bolt is inclined at an angle ($\propto$) to the joint such that forces in the bolt resist shear movement, then the increased shear resistance due to R_3 and R_4 is

$$\Delta s = (R_3 + R_4)\sin(\propto + \delta/L)\tan(\phi_j + i) + (R_3 + R_4) \times \cos(\propto + \delta/L), \tag{14}$$

where $\propto$ is the angle of bolt to shear surface (radians).

Eqs. (7)–(10) and (14) can be used to calculate the total shear resistance provided by any rockbolt at any angle to a joint. The equations give good agreement with laboratory tests performed by Dight [18] and Pellet and Egger [21], and with calculations made by Poulos [22] using the analogy of a laterally loaded pile. Fig. 11 shows the calculated combined forces R_1, R_2, R_3 and R_4 using the equations presented above, and similar predictions made by Poulos.

Equations to calculate the effect of fully anchored rockbolts are also provided by Pellet and Egger [21]. They appear to give similar predicted load versus shear displacement curves to those obtained using the equations presented in this paper. The difficulty is that they include a "correction constant" which appears to have no objective method of quantification.

4.2.2.3. Rock bolt length. The bolt length is usually taken as the required linear arch thickness plus 1 m. This presumes there to be a physical bedding plane at the upper surface of the nominated linear arch and is intended to allow sufficient bond length for mobilization of bolt capacity at this postulated plane.

4.2.2.4. Bolt density. The design process is iterative because of the following variables in regard to the bolts alone:

- bolt capacity—a function of diameter and bolting material (typically either 400 MPa reinforcing steel or nominally 950 MPa steel associated with Macalloy/VSL/Diwidag bars),
- bolt inclination,
- bolt spacing across and along the tunnel.

Typically, for tunnels of spans up to about 12 m use is made of standard rockbolt steel (nominally 400 MPa). For larger spans some, or all, of the bolts comprise high-grade steel.

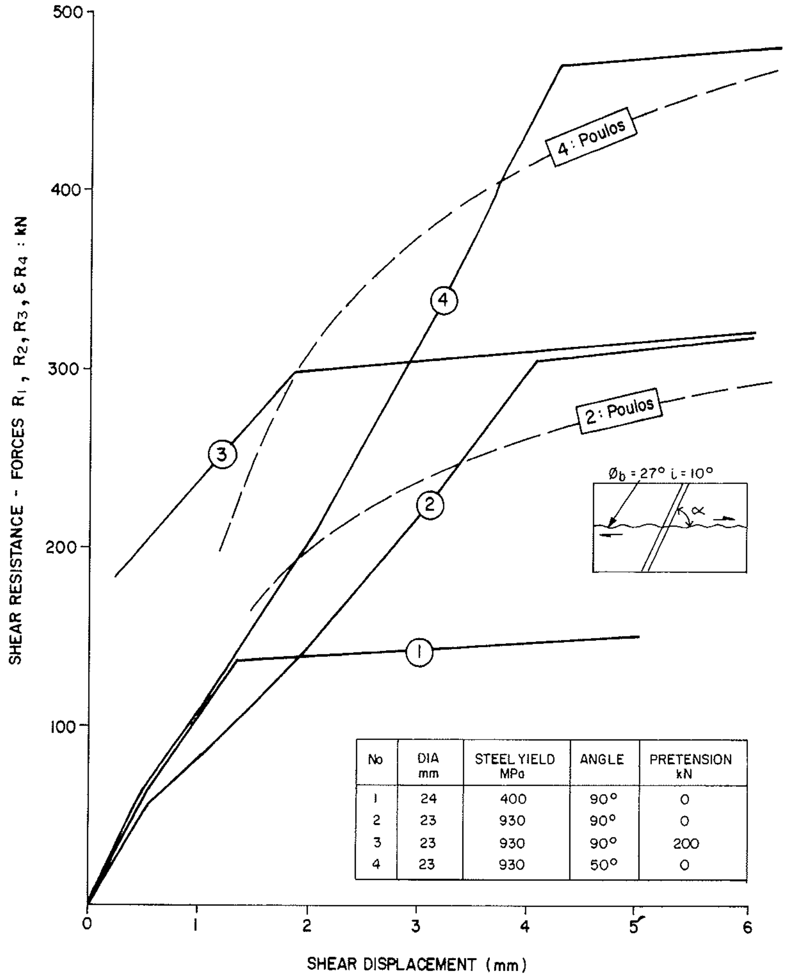
It is advantageous to incline bolts across the bedding planes provided one is certain as to the direction of shearing. However, as shown in the example presented in Fig. 9 such shearing is not always symmetrical about the tunnel centreline. Bolts inclined across bedding against the direction of shearing can be largely ineffective. Therefore given the uncertainty in this regard it is considered appropriate that only those bolts located over the tunnel abutments should be inclined, the central bolts are installed vertically. As an example, Fig. 12 shows the support used for the wide span section of the Eastern Distributor.

Having made the above decisions regarding bolt lengths and inclinations the process of bolt density computation proceeds, in principle, as set out below.

Step 1 The tunnel crown is divided into patches at each bedding horizon, as illustrated in Fig. 9, with each patch intended to cover one rock bolt. It should be noted that the first major bedding horizon above the crown usually controls design.

Step 2 From the jointed finite element analysis (see Fig 9b) the average shear displacement and the normal stress (see Fig. 9c) within each patch are calculated.

Step 3 A rockbolt type (diameter, material, inclination) is selected for a patch and the forces $R_1 - R_4$ are calculated as per the equations given in Section 4.2.2.



No	DIA mm	STEEL YIELD MPa	ANGLE	PRETENSION kN
1	24	400	90°	0
2	23	930	90°	0
3	23	930	90°	200
4	23	930	50°	0

Fig. 11. Calculated resistance forces R_1-R_4 for typical full column grouted rockbolts.

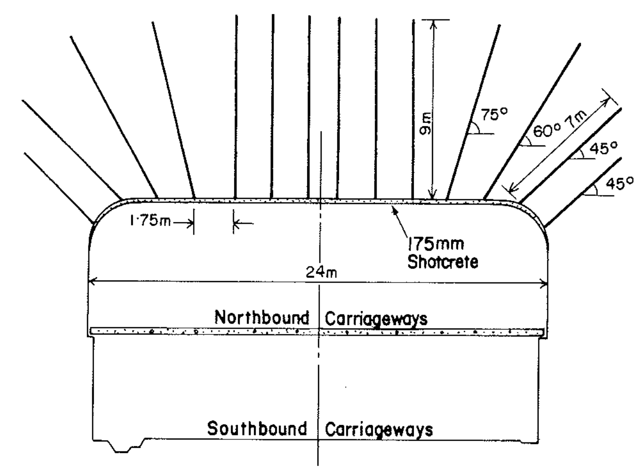


Fig. 12. Twenty-four metre span section of double decker Eastern Distributor.

Step 4 Using the values of R_1–R_4, and the normal stress from Step 2, the shear strength of the bolted patch is calculated (τ_{strength}).

Step 5 The average shear stresses (τ_{applied}) in the same patch is computed from the elastic finite element analyses (see Fig. 9d).

Step 6 The "factor of safety" against shearing within each patch is defined as

$$\text{FOS} = \tau_{\text{strength}}/\tau_{\text{applied}}.$$

It is required that each patch have a FOS$\geqslant$1.2 although it may be found that one or two patches on some joints may have lower factors of safety.

4.2.3. Calculating shotcrete requirements

4.2.3.1. Loading. The basic principle behind the design of shotcrete, in the loosening pressure environment, is that it is to support and contain the rock between the rockbolts. The size of the rock blocks which potentially have to be supported (the "design block") have to be assessed on a probability basis from the known geology. However, the point should be noted that there is no way of knowing, in advance of excavation, where exactly these blocks will be located. In reality they will occur at only a few locations in the crown of the tunnel, but because the shotcrete must be applied in a pre-planned, systematic manner, and because safety requirements dictate that not even a brick size piece of rock may be unsupported, it is necessary to assume that the 'design block' can occur anywhere. It comprises a patch of gravity load on the shotcrete.

4.2.3.2. Shotcrete action. The shotcrete may be designed to act in one of the two ways:

(i) as a membrane spanning between bolts, or
(ii) by adhesion to the rock immediately around the design block.

The two mechanisms are illustrated in Fig. 13.

Under membrane action the flexural strength of the shotcrete is the key material parameter and steel fibres, or mesh, are essential. In addition considerable attention has to be given to detailing a good structural connection between shotcrete and rockbolt heads. Spider plates, as illustrated in Fig. 14, were used for this purpose on the Eastern Distributor project.

The adhesion mechanism involves the shotcrete acting in shear, [23], and therefore flexural strength (and high dosage of steel fibres) is of little relevance. The key factor for 'adhesion' behaviour is that the rock surface is very clean.

Current practice in the Hawkesbury Sandstone is to use the adhesion mechanism when designing in Class I or Class II sandstone, and to use the membrane concept in poorer quality sandstone and shales.

Details of the methods of calculation are given by Bertuzzi and Pells [24] and Barrett and McCreath [23] and are not repeated here.

4.2.3.3. Typical shotcrete specifications. Current design specifications in the Sydney tunnels are typically as summarised below.

Shotcrete designed for membrane action

UCS	40–50 MPa
Residual flexural strength (ASTM)	2 MPa at 2 mm
Steel fibres	Dramix 30 or 40 mm at about 50–60 kg/m^3
Aggregate grading	EFNARC [25]
Microsilica	about 5% of cement content

Shotcrete designed for adhesion

UCS	35–45 MPa
Steel fibres	Dramix 30 mm at 20 kg/m^3
Aggregate grading	EFNARC [25] or finer
Microsilica	About 8–10% of cement content

4.2.4. Role of the observational method

The design process described above cannot be done for each metre of tunnel, and the practice is to develop designs for the Typical, Adverse and Special conditions expected in each Region of the geotechnical model prepared for the project, as described by Pells and Best [26]. Mapping and monitoring is essential during construction to check that geotechnical conditions are as expected and that crown deflections are consistent with design expectations.

The Opera House Carpark cavern crown, with its 6 m of rock cover, was designed using the principles described above and crown sag was carefully monitored as the 17.5 m span was progressively created by stripping from an initial 6 m heading. Fig. 15 shows how crown centreline sag increased as the span was progressively increased, compared with theoretical predictions made at design stage.

Maximum crown sag measurements from four different projects are shown in Fig. 8 compared with theoretical "linear arch" predictions.

An important observation which has been made from extensometers installed in the crowns of the Eastern Distributor and M5 tunnels is that it is impractical to install rockbolts sufficiently close to the advancing faces of initial headings to prevent significant opening of bedding planes which exist less than about 0.5 m above the crown. With opening of such bedding planes close to the crown it is not possible to generate the shear response equivalent to a pseudo-clastic beam, as required by the design method given in Section 4.2.2. To allow for this reality it is recommended that the

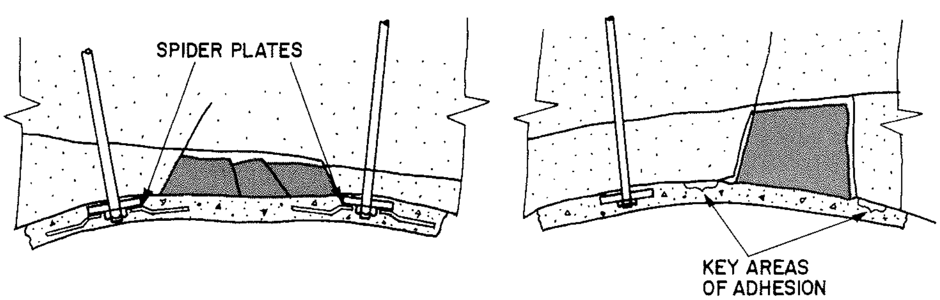


Fig. 13. Alternative structural actions of shotcrete.

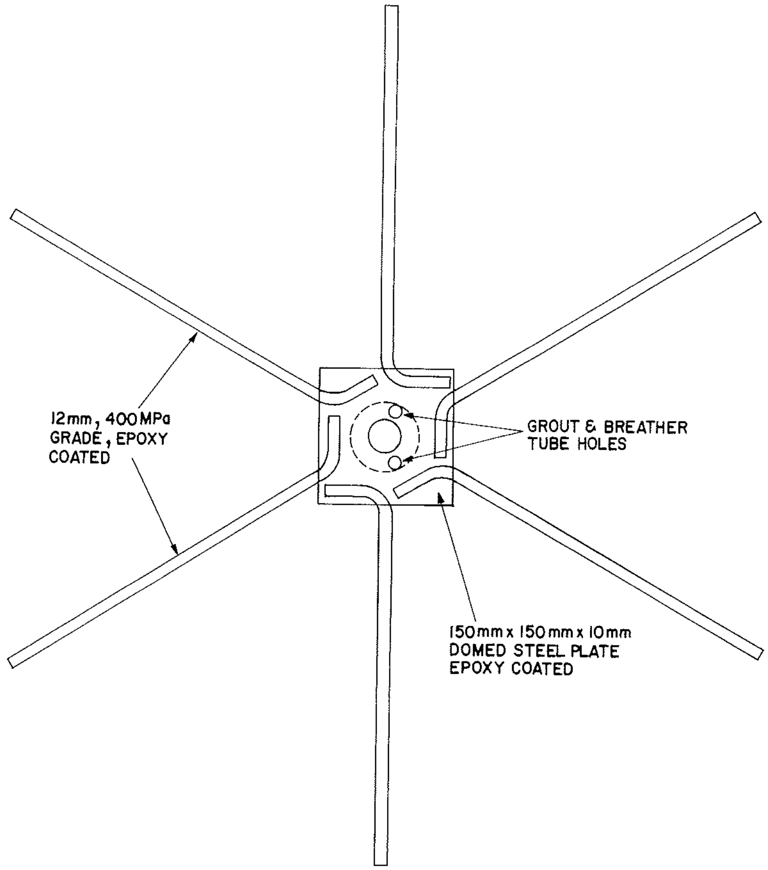


Fig. 14. Spider rockbolt plate used on Eastern Distributor to ensure adequate connection between shotcrete and rockbolts.

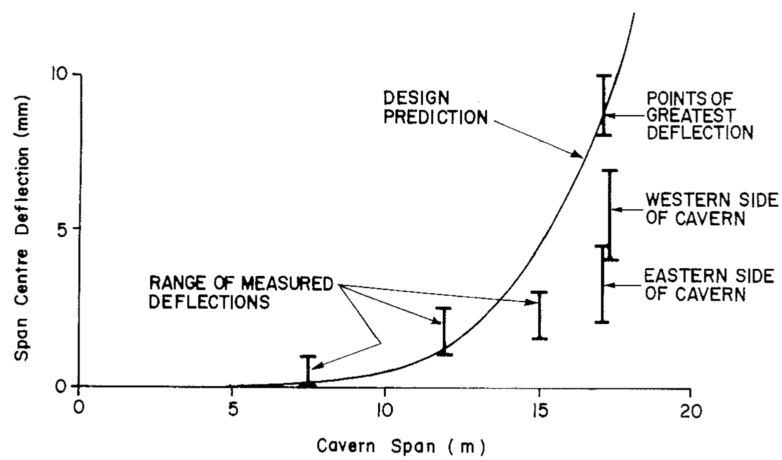


Fig. 15. Prediction & measurement of roof sag of the Opera House Carpark cavern.

lower 0.5 m of crown be treated as dead load (15 kN/m) and not be included as part of the design linear arch thickness.

4.3. Design for true rock pressure

Given the relatively high horizontal stress in the Sydney Basin (see Section 2.2.5), true rock pressure manifests as loading imposed on crown support by compressive, shear and tensile failure of the near horizontally bedded rock.

Design for true rock pressure is more difficult than for loosening pressure because a coherent analytical design method is yet to be developed. However, the principles of design are quite clear, and safe designs for deep tunnels and caverns in the Sydney Basin rocks can be developed as outlined below. The important steps are:

(i) Assess the likely locations and extents of rock mass yielding.
(ii) Take steps to reduce the extent of the problem by altering the position and shape of the excavation.
(iii) Recognise that no reasonable amount of support can prevent the rock mass yielding and that support is there to contain the failed material and maintain the geometric integrity.
(iv) Make use of the observational method by appropriate instrumentation, and implement the principles of NATM.

Steps (i) and (iii) are dealt with in more detail in the following sub-sections. Step (iv) is dealt with in many current texts and there are no special features applicable to the Sydney Basin rocks.

4.3.1. Location and extent of crown yielding

Usual practice is to assume that crown yielding initiates when the peak induced stresses at the tunnel periphery exceed the mass strength of the sandstone (typically about 20 MPa as discussed in Section 2.2.1). However, good evidence has been presented by Stacey [27] and Martin et al. [28] that the onset of, and the extent of, brittle rock mass failure can better be estimated using a tensile strain criterion. The writer's current practice is to use a stress versus strength criterion for assessment of failure likelihood, and for selecting an appropriate excavation shape. The tensile strain criterion is used to assess the likely volume of failure once it is known that true rock pressure cannot be avoided. In Class I and Class 2 Hawkesbury Sandstone it is reasonable to adopt a critical tensile strain of about 0.0005.

As discussed by Pells [29], crown stress concentrations under a high horizontal stress field are substantially increased by the presence of low shear strength and/or low stiffness near-horizontal bedding discontinuities. This is illustrated in Fig. 16 for the cases of a circular TBM tunnel.

If, at excavation level, the natural stresses are

$$\sigma_h = \sigma_1 = K_0\sigma_v = K_0\gamma d,$$

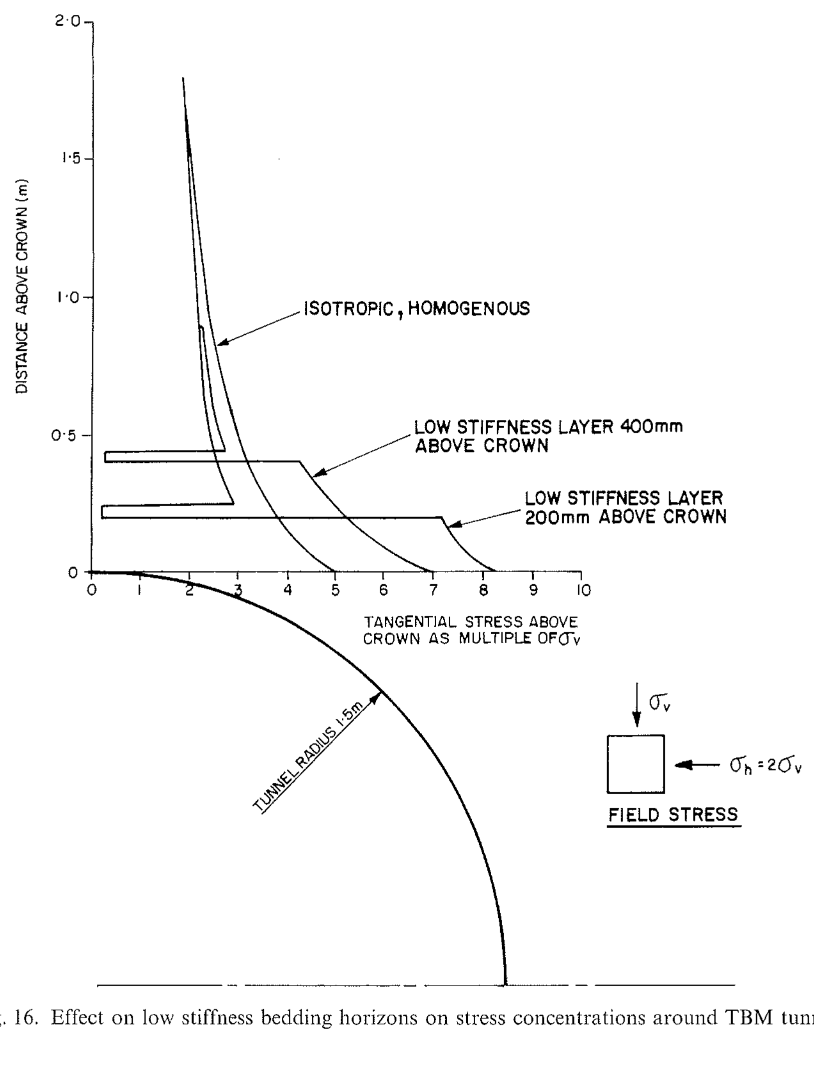


Fig. 16. Effect on low stiffness bedding horizons on stress concentrations around TBM tunnel.

where σ_h is the horizontal stress, σ_1 the major principal stress, σ_v the overburden pressure, K_0 the constant which is calculated from Eqs. (1)–(3), d the overburden depth, γ the unit weight.

Then, in general, the maximum compressive crown stresses can be expressed as

$$\sigma_0 = (S_f K_0 - 1)\gamma d,$$

where S_f is a constant depending on the shape of the tunnel and the anisotropy of the rock.

For a circular tunnel in isotropic rock, the value of $S_f = 3$. Therefore, for $\gamma = 0.024\,\mathrm{MN/m^3}$ the induced stresses (σ_0) would exceed a rock strength of 20 MPa at an overburden depth of 166 m. Actual experience in the Hawkesbury Sandstone is that crown and invert failure has developed around circular tunnels at depths of as little as 60 m. Invariably such failures are observed to be associated with low shear strength bedding surfaces just above crown or just below invert (see Fig. 17). There is no doubt that failure at such relatively shallow depth is due to the stress considerations being similar to those indicated by Fig. 16.

For rapid "back-of-the-envelope" assessments the writer uses a value of $S_f = 6$ for TBM tunnels, where low-strength bedding planes are likely to be present, indicating that significant true rock pressure problems may have to be dealt with below a depth of about 75 m. Clearly, particular site conditions affecting rock strength, bedding plane defects and topographic effects on the stress field must be taken into account in a detailed assessment, and it should be noted that lengths of tunnel below 100 m in massive sandstone have shown no signs of failure. For flat-crown tunnels the stress concentrations may be less than for a circular tunnel [29].

4.3.2. Bolting and structure requirements

The characteristics of rockbolts appropriate for true rock pressure are in some ways quite different from those required for loosening pressures. As already stated, there is no way bolting can prevent yielding of the rock. Therefore bolts must be able to accommodate potentially large shear movements along joints and new fractures, while at the same time providing high shear resistance. Again full column grouted bolts are far superior to end anchored bolts.

The following types of bolts are ineffective or, at best, inefficient:

- glass fibre reinforced plastic—because the long-term strength of this material is only about 30% of the short-term strength, and high local shear strains can easily result in the long-term strength being exceeded without the designer having any control of this matter.

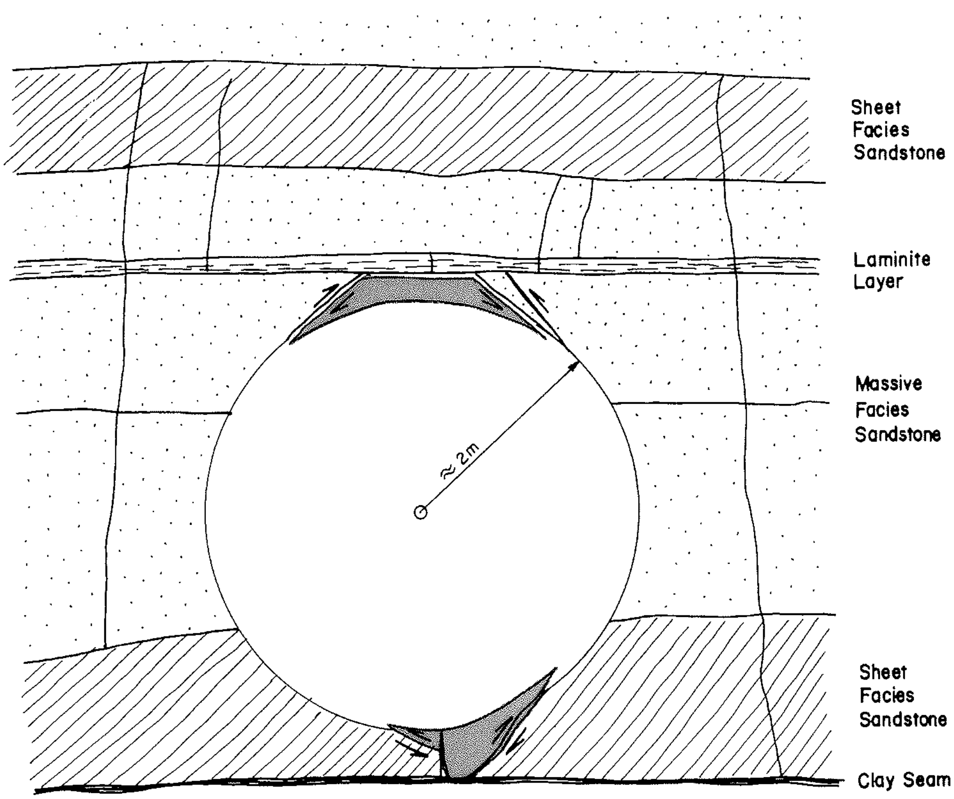


Fig. 17. Typical stress induced failure in TBM tunnel of North Side Storage Project.

- thin wall hollow steel bolts—because they provide much less shear resistance than solid bolts.
- high tensile strength steel bolts—because strain to failure may be as low as 5%.

Solid steel bolts composed of material with at least 20% strain to failure are appropriate. Furthermore, tests performed by Pells [20] and Dight [18] suggest that smooth sided bolts may perform better than rib profile bolts.

Bolt lengths are chosen so as to extend about 1 m beyond the yielded volume of rock calculated using the tensile strain criterion as discussed in Section 4.3.1. Total bolt capacity is preferably 1.5 to 2.0 times the weight of the estimated yielded volume of rock. This factor of safety reflects the uncertainty associated with predicting the volume of fractured rock.

One final point to make in regard to rockbolting for true pressure is that long term design life (greater than 25 years) is difficult to attain because the degree of local distortion of the bolts at joints and new fractures is such that the continuity of most corrosion protection measures (galvanising, epoxy coating and HDPE sheathing) cannot be assured.

Shotcrete is designed on the basis of membrane action because adhesion cannot be assumed when large rock mass deformations occur. The practice is to adopt a uniformly distributed load on the shotcrete, which is considered to be point supported by the bolts.

5. Conclusions

In the rocks of the Sydney Basin existing rock mass classification systems have value in rapid communication between professionals but are considered to have limited value in assessing support requirements.

Within the Hawkesbury Sandstone, with its near horizontal bedding and vertical jointing, it has been possible, using a combination of old and new rock mechanics concepts, to develop analytical methods for designing rock reinforcing for 'loosening pressures'. The methodology set out in the paper has been used successfully in several major projects where flat crown excavations with spans up to 24 m have no passive lining. Support comprises permanent rockbolts and a thin layer of steel fibre reinforced shotcrete. Monitoring of crown deflections and rock bolt loads has shown good agreement between theory and reality.

It is believed that the methodology proven in the Hawkesbury Sandstone can find application in similar rock masses elsewhere in the world.

At depths where rock mass yielding occurs, and designs have to deal with 'true rock pressure', the stage has not been reached that an integrated analytical design method is in place. However, there is good understanding of the important facets which must be covered by a design, and the paper provides guidelines in this regard for rock masses similar to the Hawkesbury Sandstone.

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