What happened to the mechanics in rock mechanics and the geology in engineering geology

by P.J.N. Pells*

Transaction Paper

Synopsis

A good thing is becoming a bad thing. Rock mass classification systems, that are so excellent for communications between engineers and geologists, and that can be valuable in categorizing project experience, are emasculating engineering geology and rock mechanics. Some engineering geologists have been sucked into thinking that Q and RMR values are all that is needed for engineering purposes, and seem to have put aside what can be learned from structural geology and geomorphology. Many rock mechanics engineers seem to have forgotten the scientific method. This paper attempts to redress the situation by showing how mechanics can be used in rock engineering to design with a similar rigour to that used in the fields of structural engineering, hydraulics and soil mechanics. It also attempts to remind practitioners of what can be achieved by skilled engineering geology.

Introduction

The genesis of this paper lies in a disquiet that has gradually built up over the last decade about the practice of rock engineering. I, and I am not alone, have perceived a breakdown in the sciences of both rock mechanics and engineering geology. I have perceived, at an increasing rate, hypotheses taken as laws, and practice that constitutes no more than cookbook application of ill-founded recipes. So, while I have always written papers properly in the third person, this one is personal.

My starting point for the mechanics in 'rock mechanics' is a statement by one of the fathers of rock mechanics:

'Rock mechanics is one of the scientific disciplines in which progress can only be achieved by means of interdisciplinary team work. ... As a branch of mechanics rock mechanics cannot prosper outside the general fundamentals of the science of mechanics' Leopold Muller, 1974

My starting point for geology and geomorphology in 'engineering geology' may seem strange at first sight. It is a single sentence from the introductory chapter to the book *Geomorphology for Engineers:*

'Problems have to be identified before they can be solved'. Peter Fookes and Mark Lee, 2005

I think the first phrase in this sentence covers the essence of geology and geomorphology for engineering, and the second covers geotechnical engineering.

There is a temptation in a paper of this nature to be negative, to pour scorn on what I term cookbook rock mechanics and blinkered engineering geology. But that teaches nothing. So I will attempt to illustrate the importance of good applied mechanics and, by case study, the value of good geology. Then I will present just one case study that encapsulates some of what is wrong in current practice.

Mechanics of rock socketed piles

Given ultimate end bearing and side shear values, the design of a rock socketed pile, as illustrated in Figure 1, would appear to be a trivial matter. Surely the allowable load should be given by adding the allowable end bearing load to the allowable side shear?

Is not the equation as follows?

$$P_{allowable} = \frac{A_{base} \times q_{bult}}{FOS_1} + \frac{A_{side} \times t_{ult}}{FOS_2} \quad [1]$$

where

A_{base} = area of base
A_{side} = sidewall area
q_{bult} = ultimate base resistance
τ_{ult} = ultimate side shear
FOS = Factor of Safety

Nice and simple, but the applied mechanics is wrong. To obtain an appropriate answer we have to find recourse in the theory of elasticity. As stated by two other fathers of rock mechanics:

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What happened to the mechanics in rock mechanics?

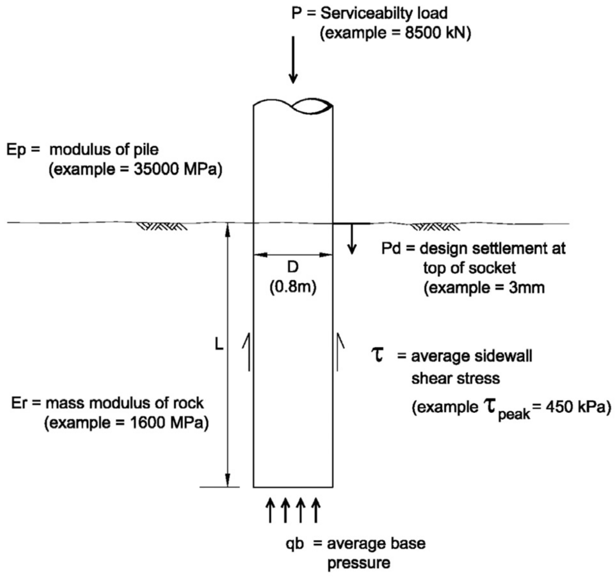

Figure 1—Rock socketed pile

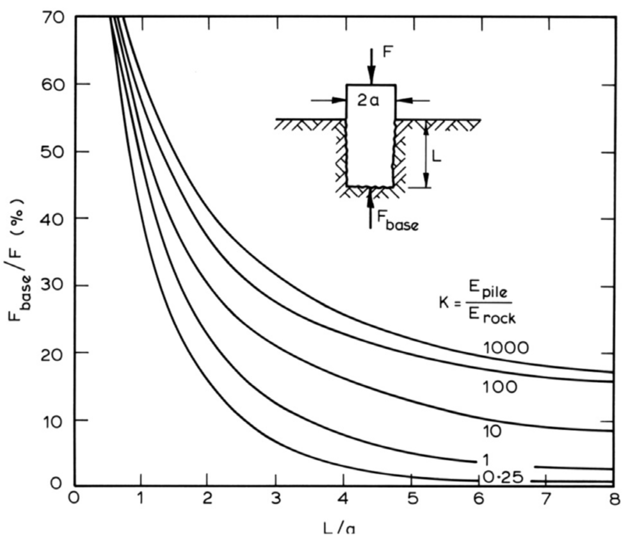

Figure 2—Load distribution between base and sidewall

'The science of rock mechanics is a whole composed of parts taken from a number of different subjects. Much of the theory of elasticity is continually needed ...' J.C. Jaeger and N. G W Cook, 1969

Elastic analysis of a rock socket indicates that the applied load is shared between sidewall and base, as shown in Figure 2. This figure shows us that the sharing of load between base and sidewall is not a matter of prescribed end bearing and side shear values, but a matter of the relative stiffness of pile and rock. Furthermore, the theory allows settlement to be calculated through the solutions given in Figure 3.

If we want to mobilize a greater proportion of base resistance than Figure 2 would allow, we have to go beyond the theory of elasticity, and allow side slip to occur. Rowe and Armitage (1984) have provided solutions for this scenario, an example of which is given in Figure 4. There are similar figures for other ratios of pile to rock stiffness, and rock sidewall to rock base stiffness. These figures can be used in a neat way to design a socketed pile, as set out below for the example shown in Figure 1.

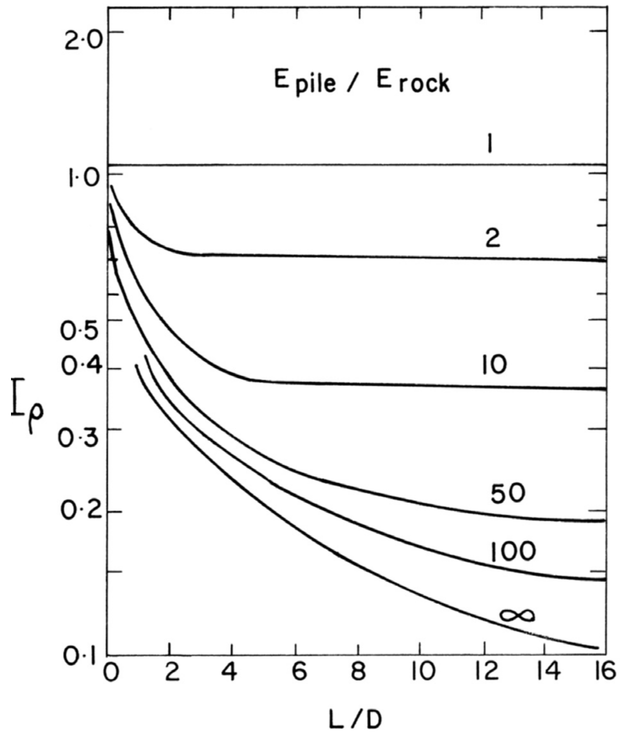

Figure 3—Settlement influence factors. Settlement = $\frac{F}{E_r D} I_P$

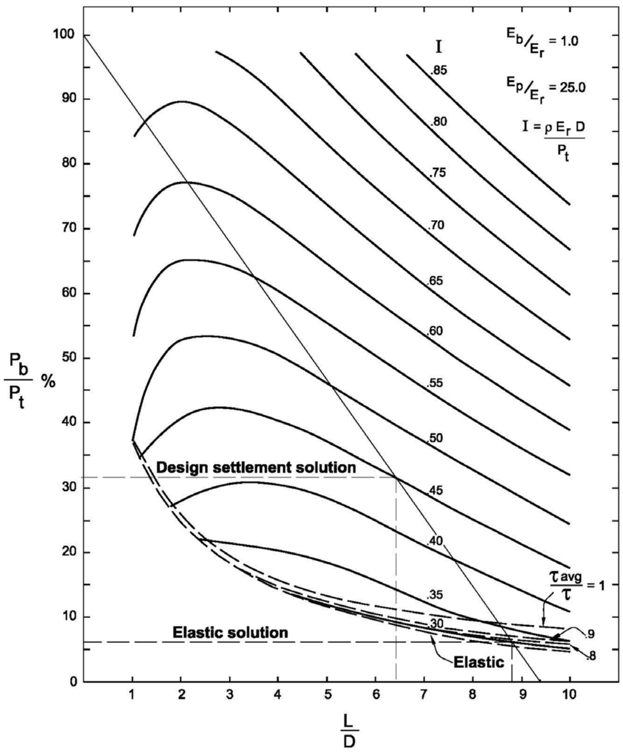

Figure 4—One of the Rowe and Armitage graphs for socket with side slip

- *Step 1*—Calculate the length of socket as if all the load were taken in side shear.

$$\text{Length} = \frac{8500}{\Pi \times 0.8 \times 450} = 7.5 \text{ m}$$

$$\text{Hench } L/D = \frac{7.5}{0.8} = 9.4$$

- *Step 2*—Plot this point on the x-axis of Figure 4 and draw a straight line to the 100% mark on the y-axis. This line represents all solutions that obey the requirement of an average side shear value of 450 kPa.
- *Step 3*—Calculate the settlement influence factor.

$$I = \frac{\rho E_r D}{Pt}$$

$$I = \frac{0.003 \times 1600 \times 0.8}{8.5}$$

$$=0.45$$

- *Step 4*—Where the straight line intersects the influence factor line for 0.45 is the design solution, if you are prepared to accept side slip. The design would be:

Socket length = 6.4 x 0.8 = 5.1 m

$$\text{End bearing pressure} = \frac{0.32 \times 8.5 \times 4}{\Pi \times 0.8^2} = 5.4 \text{ MPa}$$

If 5.4 MPa is considered too high for end bearing pressure then other points can be chosen along the straight line, as far down as the elastic solution. For these solutions settlements will be less than 3 mm. The limit is the elastic solution where

Socket length = 8.8 x 0.8 = 7 m

$$\text{End bearing pressure} = \frac{0.06 \times 8.5 \times 4}{\Pi \times 0.8^2} = 1.0 \text{ MPa}$$

Thus proper applied mechanics gives an elegant design method.

Mechanics of support design in horizontal bedded strat

Scope of application

In many parts of the world there occur near horizontally bedded sandstones and shales in which to a depth of several hundred metres the natural horizontal stresses are higher than overburden pressure. Examples include the Karoo beds of South Africa, the Bunter Sandstone of the UK and the Triassic strata of the Sydney region.

Fundamentals

Analytical studies have shown (Hoek and Brown, 1980, Pells, 1980) that in such strata, and in such stress fields, stress concentrations in the crown are smaller with a flat crown shape than with an arch (see Figure 5). Furthermore, cutting an arch-shaped crown in this type of rock is counter-productive because this creates unnecessary cantilevers of rock and fails to utilize positive aspects of a relatively high horizontal stress field (see Figure 6).

The simple piece of applied mechanics published by Evans in 1941 showed that spans in excess of 15 m can readily be sustained in a typical horizontally bedded sandstone having unconfined compressive strength greater than about 20 MPa provided the effective bedding spacing is greater than about 5 m. For strata of other strengths, stiffnesses and natural stress fields the requisite thickness can be calculated using an updated version of Evan's linear arch theory as published by Sofianos. The problem is that bedding spacings are typically much less than 5 m, so the trick is to make the rock mass function as if there is a 5 m thick bed overlying the excavation. To do so one has to use reinforcement to reduce bedding plane shear displacements to those that would occur in an equivalent massive beam.

To implement this procedure two initial sets of calculations have to be made:

(i) Calculation of the bedding plane shear displacements 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. If only horizontal bedding discontinuities are considered then an approximate closed form solution can be used, as discussed by Bertuzzi and Pells (2002).

(ii) Calculate the shear stresses that 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. An example of this procedure is given by Pells (2002).

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.

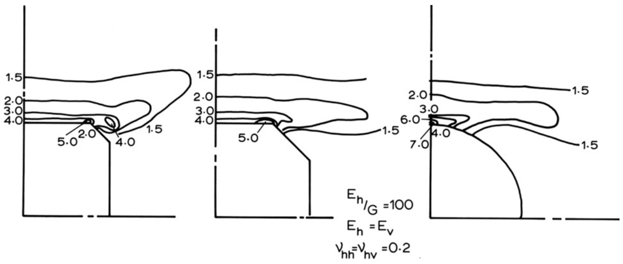

Figure 5—Contours of major principal stress as a function of the virgin horizontal stress field

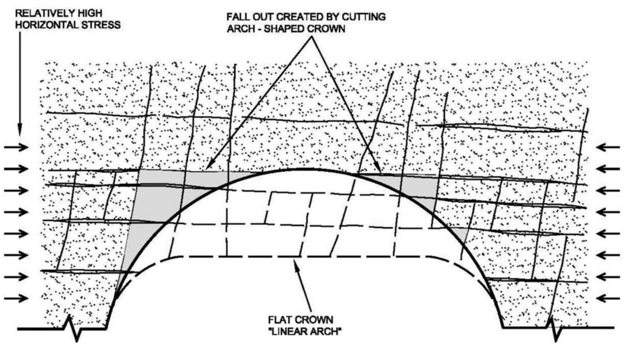

Figure 6—Negative impacts of excavating an arch shape in certain horizontally bedded strata

Calculation of rockbolt capacities

Forces

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 should be used only for local support of isolated loosened blocks of rock.

Figure 7 shows the case of a single rockbolt crossing a discontinuity. The reinforcement acts to increase the shear resistance of the joint by the mechanisms summarized below.

- ➤ An increase in shear resistance due to the lateral resistance developed by the rockbolt via dowel action—force R_1.
- ➤ An increase in normal stress as a result of prestressing of the rockbolt—force R_2.
- ➤ An increase in normal stress as a result of axial force developed in the rockbolt from dilatancy of the joint—force R_3.
- ➤ An increase in normal stress as a result of axial force developed in the rockbolt from lateral extension—force R_4.
- ➤ An increase in shear resistance due to the axial force in the rockbolt resolved in the direction of the joint—force R_5.

Forces R_1 and R_5 can be considered as increasing the cohesion component along the joint. The other three components increase the frictional component of joint shear strength by increasing the effective normal stress on the interface. If the rockbolts are at a spacing S, so that each bolt affects an area S^2, the equivalent increases in cohesion, Δ_c, and normal effective stress, $\Delta\sigma_n$, are as follows:

$$\Delta c = \frac{R_1 + R_5}{S^2} \qquad [2]$$

$$\Delta\sigma_n = \frac{R_2 + R_3 + R_4}{S^2} \qquad [3]$$

Therefore, the equivalent strength of the joint, s_j will be as follows:

$$s_j = \left(c_j + \Delta c\right) + \left(\sigma_{n0} + \Delta\sigma_n\right)\tan\phi_j \qquad [4]$$

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 (Equation [2]) and $\Delta\sigma_n$ the equivalent increase in effective normal stress (Equation [3]).

Force R_2 is created by the initial pretension in the bolt, as too is most of the force R_5. Methods of calculating forces R_1, R_3, and R_4 are set out below.

Calculations of dowel action: force R1

Calculations of dowel action is based on laboratory test data and theoretical analyses presented by Dight (1982). The experimental data showed that:

- ➤ plastic hinges formed in the fully grouted rockbolts at small shear displacements (typically <1.5 mm); these plastic hinges were located 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 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, with corrections by Carter (2003), are:

$$R_1 = \frac{D^2}{4}\sqrt{1.7\sigma_y P_u \Pi\left[1 - \left(\frac{T}{T_y}\right)^2\right]} \qquad [5]$$

where

$$P_u = \sigma_c\left[\frac{\delta}{K\left(\Pi D + 2\delta\right)}\right]^{A/2} \qquad [6]$$

$$A = \frac{2\sin\phi}{1+\sin\phi} \qquad [7]$$

$$K = \sigma_c\left(\frac{1-v^2}{E}\right)\ln\left(\frac{\sigma_c}{2p_o - \sigma_t}\right) + \frac{\sigma_c}{2P_0 - \sigma_t}\left[\frac{2v\left(P_o - \sigma_t\right) - \sigma_t}{E}\right] \qquad [8]$$

and where

σ_y, T_y = yield stress and yield force in the bolt
σ_c = unconfined compressive strength of the rock
σ_t = tensile strength of the rock
ϕ = internal angle of friction of the rock
v, E = elastic constraints of the rock
P_O = initial stress in the rock in the plane
δ = shear displacement of the joint
T = initial bolt pretension

The term in the square brackets in Equation [5] allows for the effect on the plastic moment of the tensile force in the bar. Strictly R_2 should be modified by R_3 and R_4 but this is a second order effect.

Calculation of axial forces due to joint dilation (R3) and due to bolt extension caused by shearing (R4)

If the assumption is made that in a fully grouted rockbolt the

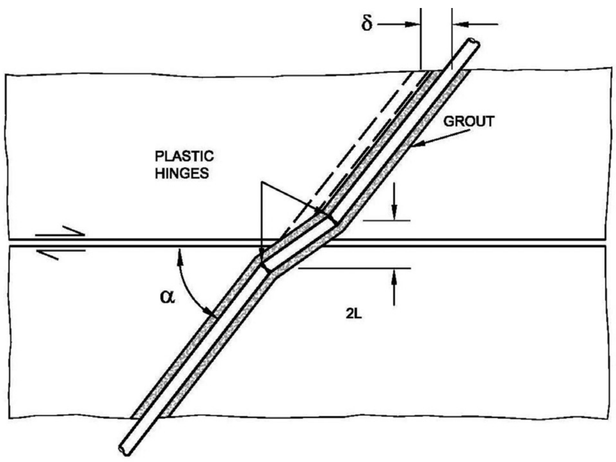

Figure 7—Grouted rockbolt in shear (after Dight 1982)

incremental axial strain in the bolt is dominantly between the two plastic hinges (see Figure 7) then the normal force generated by dilation is:

$$R_3 = \frac{\pi D^2 E_s}{4}\left(\frac{\delta \tan i}{2L}\right)\sin^3 \alpha \qquad [9]$$

where

α = angle between bolt and bedding plane

i = dilation angle

The axial force due to lengthening is:

$$R_4 \Lambda = \frac{\left(\frac{2L}{\tan\alpha}+\delta\right)\sin\alpha}{2L\cos\gamma} - 1 \qquad [10]$$

$$\gamma = \tan^{-1}\left(\frac{2L\tan\alpha}{2L+\delta\tan\alpha}\right) \qquad [11]$$

Components due to bolt prestressing

If a bolt is prestressed to a force P_{st} prior to grouting then the normal force on the joint is:

$$R_2 = P_{st}\sin\left(\alpha - \tan^{-1}\left(\frac{\delta}{2L}\right)\right) \qquad [12]$$

and the force along the joint is

$$R_5 = P_{st}\cos\left(\alpha - \tan^{-1}\left(\frac{\delta}{2L}\right)\right) \qquad [13]$$

Equations [9] to [13 presume that rockbolts are inclined so that movements on bedding planes increase their effectiveness.

Relative Importance of the forces R_1 to R_5

Figure 8 shows the contributions of the different rockbolt actions to the shear strength of a typical joint or bedding plan in Hawkesbury sandstone. The figure shows clearly that at shear displacements of about 2 mm the contributions from prestress and dowel action are of similar magnitude. The contribution due to elongation is quite small but the contribution from joint dilation can completely dominate the load in the bolt, and with rough joints will rapidly lead to bolt failure.

At this time I have not explored the relative contributions in strong rock; it could be an illuminating exercise.

Design of rockbolt layout to create the requisite linear arch

Rockbolt 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.

Rockbolt density

The design process is iterative because of the following variables for the bolts alone:

- bolt capacity—a function of diameter and bolting material (typically either 400 MPa reinforcing steel, or 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. Bolts inclined across bedding against the direction of shearing can be ineffective. Therefore, given the uncertainty about this, it is considered appropriate that only those bolts located over the tunnel abutments should be inclined, the central bolts are installed vertically. Figure 9 shows the support used for the wide span section of the Eastern Distributor tunnels in Sydney.

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

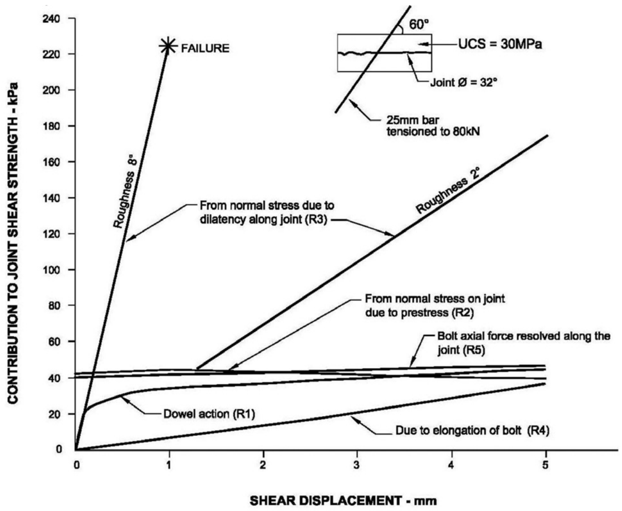

Figure 8—Contribution of the different bolt actions to joint shear strength

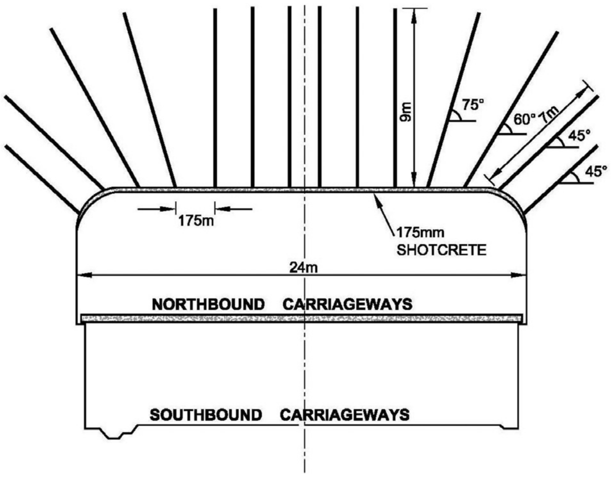

Figure 9—Rockbolt support for 24 m span of double decker Eastern Distributor

- *Step 1*—The tunnel crown is divided into patches at each bedding horizon with each patch intended to cover one rockbolt. It should be noted that the first major bedding horizon above the crown usually controls design.
- *Step 2*—From the jointed finite element analysis the average shear displacement and the normal stress within each patch are calculated.
- *Step 3*—A rockbolt type (diameter, material, inclination) is selected for a patch and the forces R_1 to R_5 are calculated as per the equations given above.
- *Step 4*—Using the values of R_1 to R_5 and the normal stress from Step 2, the shear strength of the bolted patch is calculated ($\tau_{strength}$).
- *Step 5*—The average shear stress ($\tau_{applied}$) in the same patch is computed from the elastic finite element analyses.
- *Step 6*—The 'factor of safety' against shearing within each patch is defined as

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

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

Calculating shotcrete requirements

The basic principle behind the design of shotcrete, in the loosening pressure environment, is to support and contain the rock between the rockbolts. The size of the rock blocks that 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 preplanned, 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.

Structural design of shotcrete for block loading is discussed in many texts, is summarized in Pells (2002), and need not be repeated here. Suffice it to say that design can either consider the shotcrete spanning between rockbolt ends, or can adopt the concept of adhesion. I caution against the latter approach in closely bedded or highly fractured rock, because the shotcrete may be adhered to something that is not adhered to anything else.

Summary

We have here a situation where support for a tunnel or cavern can be designed with similar rigour to that used by structural engineers for bridges, or hydraulic engineers for pipeline systems. I contend that this is true for most tunnel and cavern design work—but the design work cannot just be done on one page.

Applied mechanics of rockbolts

Rockbolt magic

Rockbolts are sometimes ascribed abilities that verge on magic. For example they are said to prevent stress induced failure, or said to interlock a rock mass-like aggregate in an upside-down bucket (Tom Lang's famous bucket that was outside the Snowy Mountains Authority buildings for many years—see Figure 10.) I am afraid that simple applied mechanics shows that these concepts are not valid.

I know that by questioning Lang's bucket I might have my citizenship revoked but let's have a look at the applied mechanics. But firstly, for those who know nothing about this bucket, here is Lance Endersbee's (1999) version of the story.

'One demonstration which was quite convincing to the workmen was to install model rockbolts in a bucket of crushed rock, and then to turn the bucket upside down. The crushed rock remained in place, and did not fall out of the bucket. That was surprising. While still upside down, a heavy weight would then be applied to a middle rockbolt, and still the crushed rock remained in place. That was amazing. The workmen would then be reminded that all this was possible because there was a pattern of rockbolts, and that the bolts worked together.'

The mechanics of the problem is shown in Figure 11.

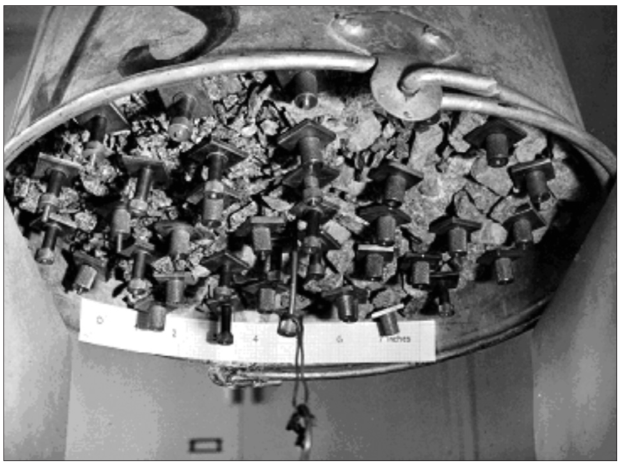

Figure 10—Lang's bucket (Endersbee, 1999)

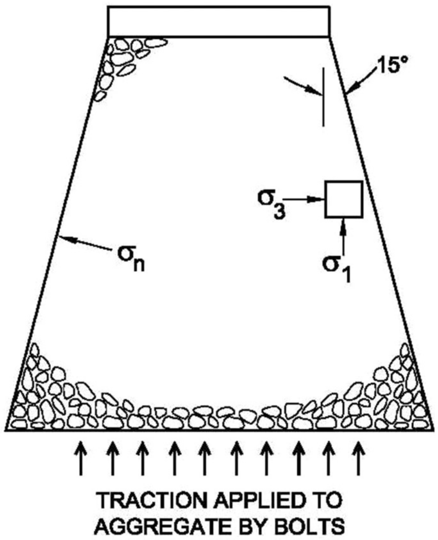

Figure 11—Stresses in Lang's bucket

The tensioned bolts create a vertical major principal stress, σ_1, in the crushed rock. The minor, horizontal, principal stress is given by

$$\begin{aligned}\sigma_3 &= K_0\sigma_1\\ &= (1-\sin\phi)\ \sigma_1\\ &= 0.75\ \sigma_1\end{aligned} \qquad [14]$$

where K_O = earth pressure coefficient at rest
ϕ = internal angle of friction, assumed to be 45°

The sides of the bucket are at 15°, so the normal stress against the side at any point is

$$\begin{aligned}\sigma_n &= \sigma_1 \sin^2 30 + \sigma_3 \cos^2 30\\ &= \sigma_1(\sin^2 30 + 0.3\cos^2 30)\\ &= 0.48\ \sigma_1\end{aligned} \qquad [15]$$

The shearing force stopping the stones falling out of the bucket is

$$T = \int \sigma_n \tan\varphi_i dA \qquad [16]$$

where
σ_n = friction angle between crushed rock and Australian galvanized steel, say ≈ 30°

Now let's put some numbers to the equations.

The bucket shown by Endersbee (Figure 10) has 36 bolts. Let us say each bolt is tensioned to 1 kg force, a moderate load.

Hence

$$\sigma_1 = \frac{36 \text{ x } 1 \text{ x } 9.81}{\text{average area}} = \frac{36 \text{ x } 1 \text{ x } 9.81}{0.05} = 7 \text{ kPa}$$

Hence
σ_n = 3.4 kPa (from Equation [15])
and
T ≈ 3.4 x tan 30° x 0.236 (from Equation [16])
≈ 0.46 kN

The crushed rock in the bucket weighs 0.32 kN, so Lang could hang another 0.14 kN (a 14 kg bag of sugar) on his hook before the whole lot fell out.

What is the relationship of this experiment to the action of rockbolts around a tunnel? I would suggest very little, because the stress scale is all wrong. Stresses of 3.4 kPa, or 34 kPa, for that matter, mean nothing in relation to rock mass stresses around a tunnel. I think that Lang's bucket is best thought of as just a demonstration of Terzaghi's silo theory.

However, Lang's bucket was extended to the concept of a 'ring of compressed and strengthened rock' by Pender, Hosking and Mattner (1962), as shown in the reproduction of their diagram given as Figure 12. This figure has been reproduced in many texts, but analysis shows that it is not valid.

Figures 13a and 13b show the major and minor stresses generated around a tunnel by a typical pattern of pre-

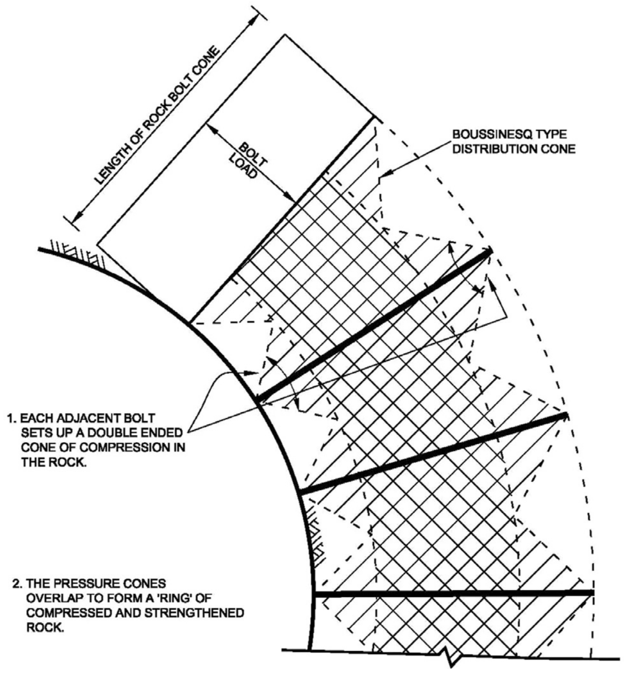

Figure 12—Another myth? (reproduced from Pender *et al.*, 1963)

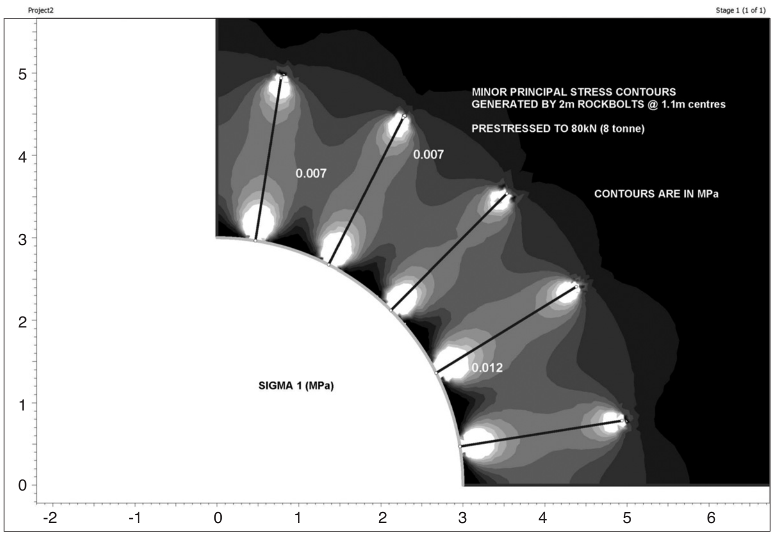

Figure 13a—Contours of major principal stress—2 m rockbolts at 1.1 m centres pretensioned to 80 kN

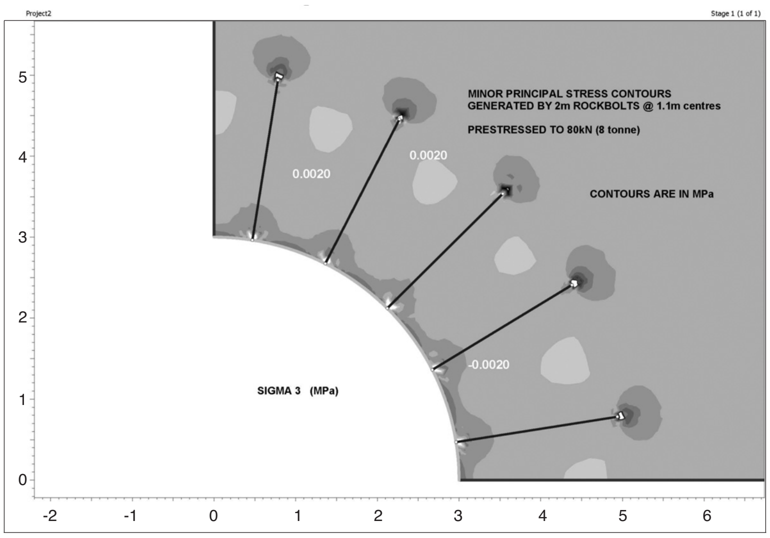

Figure 13b—Contours of minor principal stress

tensioned rockbolts. The pattern of major principal stress looks somewhat like Figure 12. The problem is the magnitude of the stresses. Away from the bolt ends the induced major principal stress is about 7 kPa and the induced minor principal stress about 2 kPa. These are too small, by several orders of magnitude, to have any effect on the rock mass strength by 'confinement'.

How do rockbolts really work?

I conclude on the basis of the applied mechanics presented above, that in most cases rockbolts do not serve a significant support function by modifying the general stress field around a tunnel. As stated in 1974 by Egger, of the University of Karlsruhe, rockbolts function as structural elements serving to transmit tensile forces within the rock mass. In this way they are not very different from reinforcing steel in concrete, except that most concrete does not include pre-existing joints. The provision of an element that is strong in tension can prevent, what Egger called, 'disintegration'. In his words, 'a tensioned anchor holding a rock block in its original position acts as a preventive measure against the disintegration of the rock'.

It is my conclusion that where the form of loading on tunnel support is 'loosening pressure' (as defined by Lauffer in 1958) rockbolts have to sustain only small tensile and shear movements and bolting requirements can be determined quantitatively by statics and keyblock analyses.

For 'true rock pressure' (again as defined by Lauffer) the function of rockbolting is different. Bolting cannot prevent stress induced fracturing and yielding, and the only purpose is to maintain the geometric integrity of the rock mass, so that stresses can redistribute and the fractured rock mass itself provide the requisite support. In this situation, bolts must be able to accommodate large movements across joints and new fractures, and the ability to deform 'plastically' is very important in selecting the types of bolts. A consequence of this 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 (galvanizing, epoxy coating and HDPE sheathing) cannot be assured.

Spaces does not permit presentation of all the structural approaches available for proper design of rockbolts. An earlier section of this paper gave one of these methods for linear arch design. There are many others, but within this category I do not include design by rock mass classification systems. Why? Because the classification system approach provides little or no idea of the loads the reinforcement is supposed to carry, or the shear and tensile displacements the bolts are expected to encounter.

Mechanics of far field subsidence

The collated wisdom of those involved in predicting and measuring subsidence above coal-mine longwalls is that settlements and surface strains are substantially confined to a 'subsidence bowl'. The limits of this bowl are defined by angles of draw measured from the edges of the area of extraction. The classic diagram illustrating this view is that of the National Coal Board, as reproduced in Figure 14.

Therefore there was growing surprise when, in the 1990s data started coming in from the NSW southern coalfields (2 hours' drive south of Sydney) showing significant horizontal ground movements well outside the expected subsidence bowl.

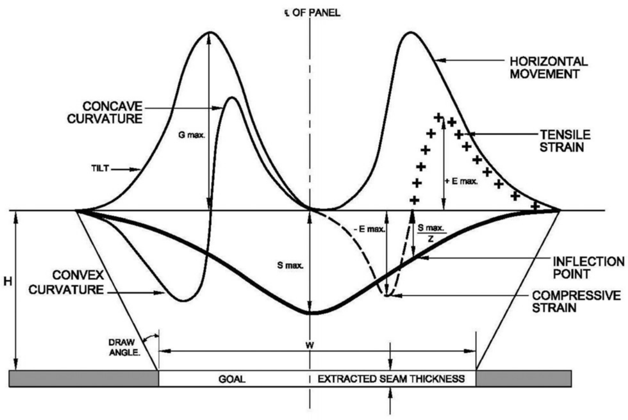

Figure 14—Standard subsidence model

The longwall workings in these coalfields are at depths of about 400 m to 500 m, and movements were being measured 1 km or more away from an active longwall. Figure 15 shows one example of these measured movements and it can be seen that in this case lateral movements of about 40 mm were measured about 1.5 km away from a longwall panel being extracted at a depth of about 480 m. There were no measurable vertical movements at this distance. These lateral movements have been termed 'far field subsidence movements'.

Figure 16 gives a summary of far field movement in the Sydney Basin, collected by mine subsidence engineering consultants.

It has become apparent that the far field movements are due to redistribution of the high horizontal stress field in the Triassic sandstones and siltstones that overlie the Permian coal-seams. These horizontal stresses are typically 2 to 3 times the overburden pressure.

Over the past 150 years coal has been extracted over a huge area within the southern coalfields, and much of this has involved almost total extraction, either by pillar recovery or, over the past 30 years, by longwalls. This extraction creates goaf and sag zones extending 70 m to 120 m above the seams. The horizontal stress that was previously transmitted through this goaf zone now must be transferred over the top of the goaf, and around the mined area.

At first sight analysis of this phenomenon would appear to require a substantial 3D numerical analysis including jointed and fractured rock. However, there are two factors that suggest that such complexity may not be necessary.

The first is that the far field movements are pseudo-elastic movements a very long way from the goaf zone where the complex 3D non-linear fracturing is taking place, and St Venant told us long ago that when we are interested in stress or displacement well away from the point of action it really does not matter what is going on at that point, as long as we obey the laws of equilibrium and elasticity.

The second is that the coal-seam where extraction is occurring is a low stiffness horizon and it is not unreasonable to postulate that most of the regional stress redistribution occurs above seam level.

These two insights allow us to try a first pass analysis using a 2D, bird's-eye view, finite element model. The whole thickness above seam level is a plane stress plate. The ruined areas are modelled by reducing the stiffness of these areas according to the ratio of goaf height to total rock cover above the seam.

Figure 17 shows the results of the model prediction for the actual situation shown in Figure 15. It is remarkable how good a prediction is obtained from such a simple model, and how useful this model is in predicting incremental far field movements from future longwalls.

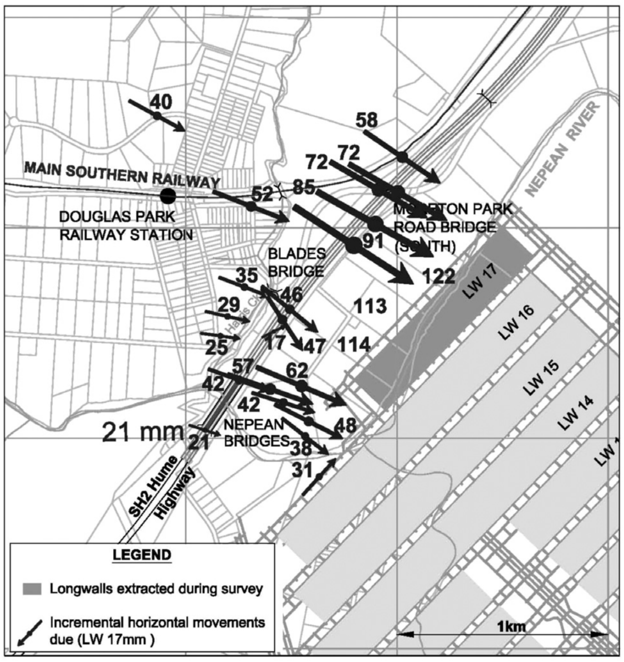

Figure 15—Far field movements, Douglas Park (mine subsidence engineering consultants)

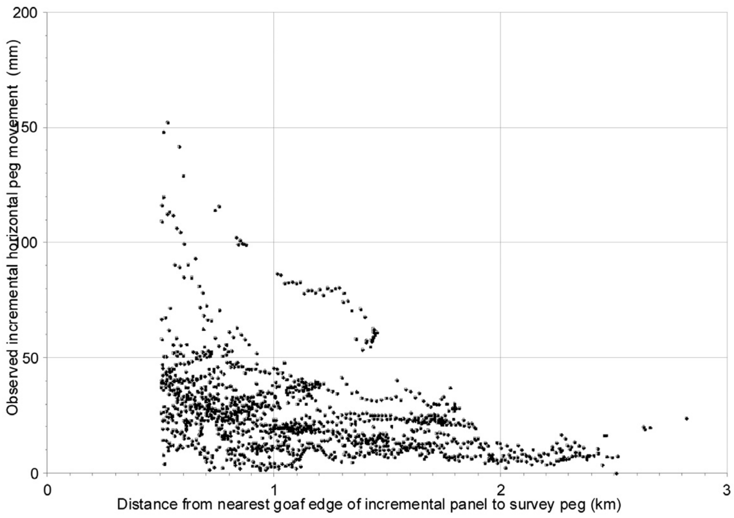

Figure 16—Far field horizontal movements in the NSW Southern Coalfield (from mine subsidence engineering consultants, 2008)

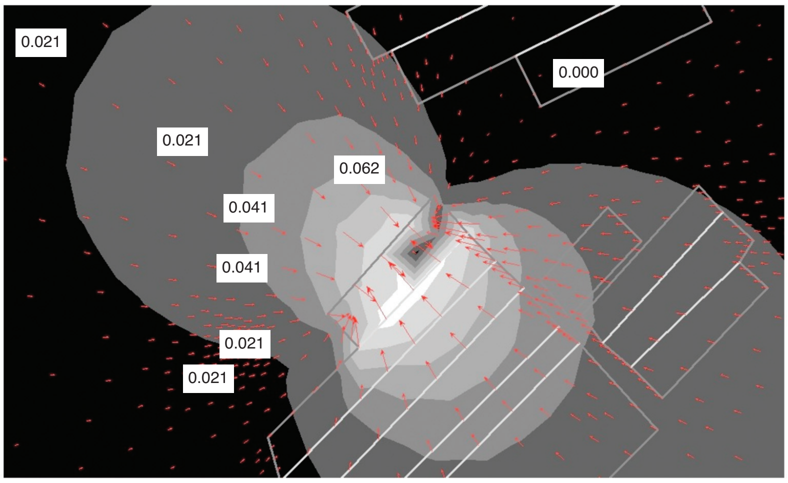

Figure 17—Predicted far field movements in metres

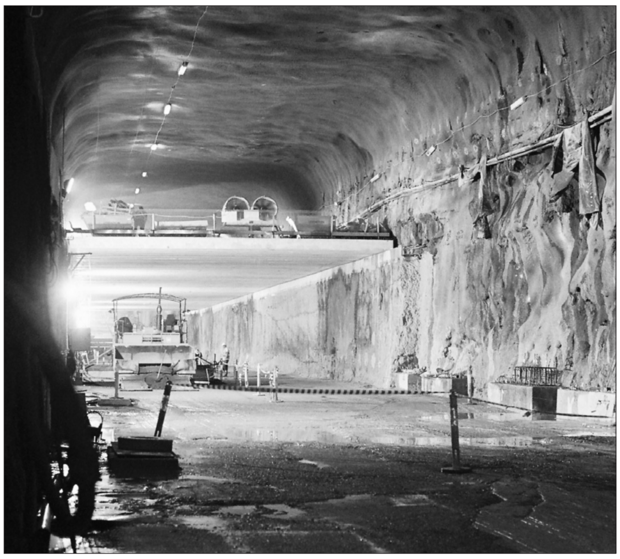

Figure 18—Eastern Distributor Tunnel. Concrete planks for northbound lanes supported on narrow ledge—approaching a fault zone in which the planks had to be supported on a column-beam structure

Stability of one-sided wedges

The problem

From time to time one comes across a issue of stability where a set of joints intersects a face at a moderately acute angle, but there is no second joint set to create a release plane for a wedge failure.Such a situation arose in connection with the design of the ledges supporting the upper carriageway of the Eastern Distributor in Sydney (see Figure 18). Near vertical joints of the dominant north-north-east joint set in the Hawkesbury sandstone intersects the ledges at oblique angles (see Figure 19). The joints are quite widely spaced, meaning that there are considerable lengths of ledge comprising intact sandstone. However, at joint locations it was clear that rockbolts would need to be installed to provide an approximate safety factor against bearing capacity failure. A simple method had to be developed to determine the bolting capacity.

Fudging of the 3D problem into a 2D analysis

Consider a simple 2D wedge on an inclined plane acted upon by a surcharge (Figure 20). A rockbolt, tensioned to a load T is installed at an angle to the plane.

The factor of safety of the reinforced wedge is defined as:

$$FOS = \frac{Resisting\ Force}{Disturbing\ Force} \qquad [17]$$

where:

$$Resisting\ Force = b\left(q + \frac{\gamma H}{2}\right)\cos\alpha\tan\phi' + T\sin(\alpha-\delta)\tan\phi' + T\cos(\alpha-\delta) + \frac{c'H}{\sin\alpha} \qquad [18]$$

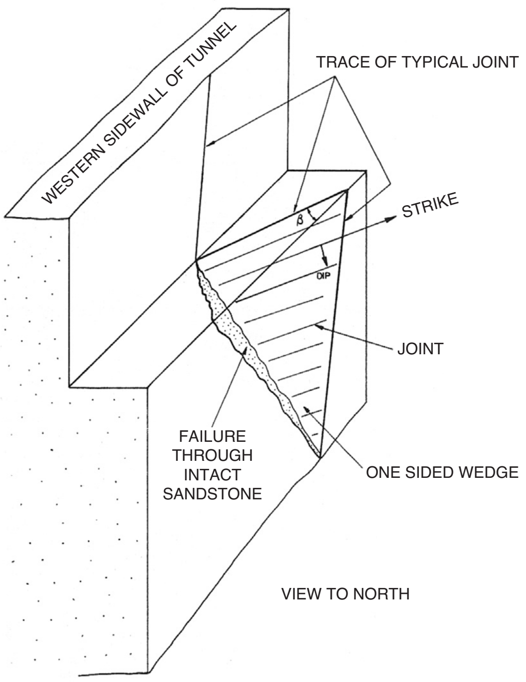

Figure 19—Geometry of rock ledge

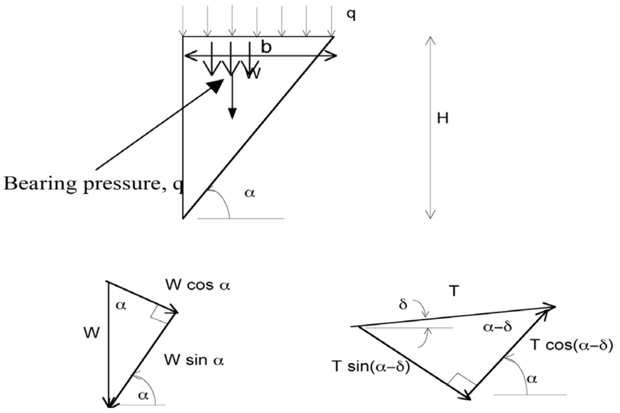

Figure 20—Two-dimensional wedge

$$Disturbing\ Force = b\left(q + \frac{\gamma H}{2}\right)\sin\alpha \qquad [19]$$

where:

b = width of ledge
α = dip of a sliding plane
H = height of the 2D wedge
T = bolt tension
c' = cohesion along the failure wedge

It should be noted that there is an alternative definition of the factor of safety wherein the effect of the rockbolt component $T\cos(\alpha\text{-}\delta)$ is taken as reducing the disturbing force.

The method of dealing with the real 3D, one-sided, wedge illustrated in Figure 19 is to assume that failure would have to create a planar fracture or shear surface through the intact rock, as illustrated in Figure 21 (Plane L). The strength from this failure through intact rock along Plane L is 'smeared' across Plane K as an equivalent cohesion (c'). A 2D analysis using Equations [17] to [19] is then performed to determine the required rockbolt force T, for an assumed FOS.

Assessment of equivalent cohesion

The equivalent cohesion is computed from the lesser strength considering shearing through intact rock, or tensile fracture by cantilever action, on Plane L.

Control by shear strength of intact rock

The assumed equivalent cohesion c'_τ from shearing through intact rock is:

$$c'_\tau = \tau \times {}^{A_L}\!/_{A_k} = \tau \sin\alpha \tan\beta \qquad [20]$$

where

α = dip of Plane K (joint)
τ = shear strength of rock along assumed vertical surface comprising Plane L

Control by cantilever action

The equivalent cohesion on the joint plane arising from tensile failure on the postulated fracture plane can be expressed as:

$$c'_\tau = \frac{qal}{2A_k} = \frac{\sigma_t}{2}\tan\alpha\ \tan\beta\ \sin\alpha\ \sin\beta \qquad [21]$$

It should be noted that the assumptions in Equation [24] are conservative because tensile failure is assumed to occur when the extreme 'fibre stresses' reach the substance tensile strength. In fact, collapse would occur only when tensile fracturing has propagated some distance into the rock.

Application

The upper heading was excavated first and during this process all the near vertical joints were accurately mapped. Thus when the ledges were exposed by subsequent excavation of the lower carriageway, it was possible to know exactly where one-sided wedges would occur. Given the measured strike and dip of a joint, the orientation of the tunnel and the known load on the ledge it was a simple matter to use a spreadsheet to compute the required numbers of rockbolts.

Geology

Up to this point we have considered how applied mechanics can be used in rock engineering. I would now like to look at how good geology can be used in engineering geology, how it can be used to correctly identify a problem.

In the early 1990s Coeur d'Alene Mines Corporation bought the Golden Cross gold mine in the Coromandel Peninsula of New Zealand. This was an old underground mining area, and was opened up with a new open pit, processing plant and tailings dam. The mine site is a beautiful, environmentally sensitive place and the mine infrastructure was developed with great care. Trout could be caught just downstream of the process plant.

As shown in Figures 22 and 23 the tailings dam was on a hillside about 1 km upslope of the access road.

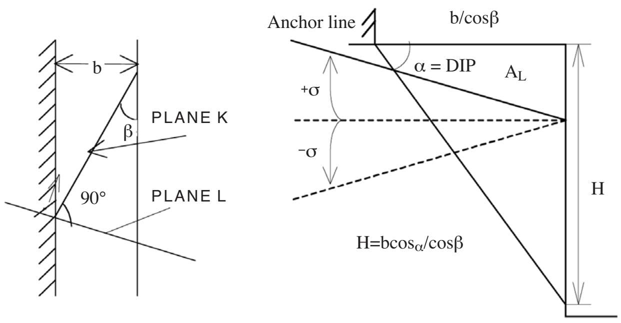

Figure 21—Assumed geometry

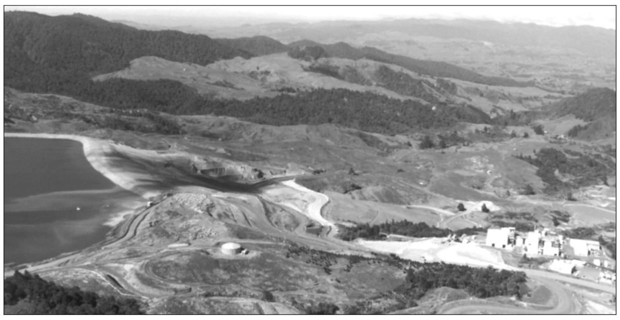

Figure 22—Golden Cross tailings dam on left, access road is along the valley floor on the right

What happened to the mechanics in rock mechanics?

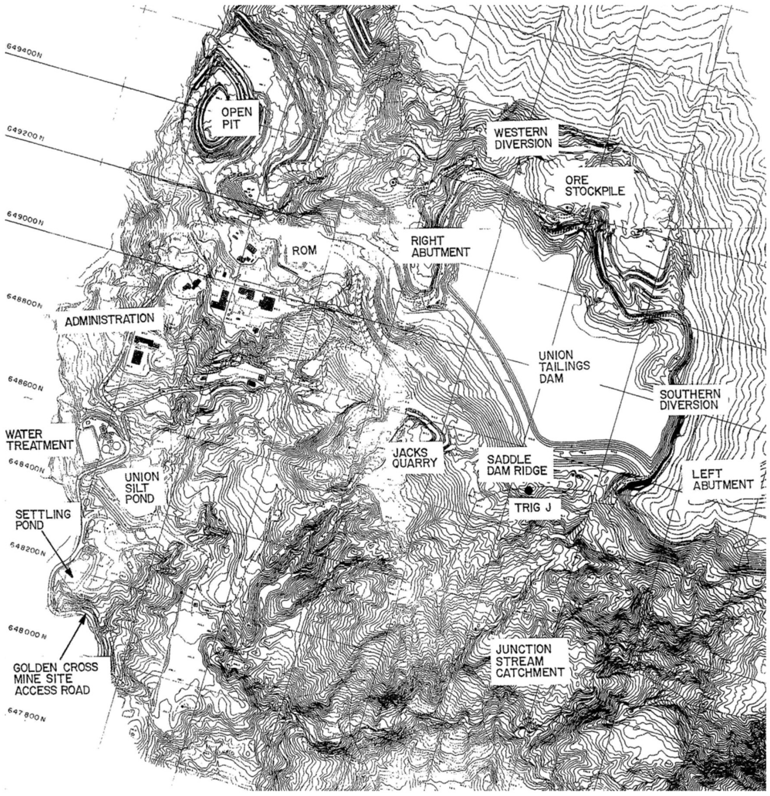

Figure 23—Plan of Golden Cross mine site

In April 1995, on a site visit to advise on the open pit, Tim Sullivan noted, at night, certain bumps ('two steps') in the access road, south-west and downhill from the dam. Some months later (August) he drove the same road and, almost subconsciously, noted that the bumps were different ('five steps').

In the meanwhile two groups of consultants were involved in design and monitoring of the dam. Concerns had arisen, and had not gone away, about cracking in the abutment areas of the dam. Investigations had been undertaken, piezometers installed, monitoring stations set up, and further fill had been placed against the, already gentle, downstream face of the dam. However, movements continued, and the whole issue heated up when a crack opened about 100 mm, with associated shearing, at Trig J near the left abutment of the dam (see Figure 23).

Shortly after this, Tim and I were retained to review the tailings dam area. It did not take him very long to put together the following observations and facts.

- The site geology comprises
  - ash (recent), overlying
  - alluvial/colluvial deposits, overlying
  - Omahia andesite, overlying
  - Coromandel volcanics (basement rock).
- It had been known for decades that the contact between the Omahia ardesite and the, smectite-rich, Coromandel volcanics played an important role in shaping the land. The contact dips at about 10° south.
- Four 'active faults' had been identified during investigations for the dam.
- An area of *tomos* (Maori word for sinkhole) was identified in the saddle embankment area during construction, and another within the reservoir area.
- A potable water bore to the west of the tailings dam was found to be blocked, or the pipe bent, at a depth of 34 m.
- A large slide had developed in the north wall of the open pit, controlled by sliding on the near horizontal Omahia-Coromandel contact.
- A monitoring bore, downhill of the saddle embankment ridge, was found to be blocked at 25 m.
- Bumps in the access road had changed.
- Cracking had been observed in the abutment ridge, and cracking and a *tomo* in the diversion drain around the tailings reservoir.
- An inclinometer near the underground mine vent shaft (west of the dam) had sheared off at 20 m.

On the basis of these observations Tim postulated that the mine could be dealing with a very large landslide (about 1.5 km downslope length, and 0.5 km width) and unknown depth, carrying the whole tailings dam along for the ride.

It need hardly be said that this view did not go across very well. Strong views were expressed that the observed events and features were unrelated and represented localized near-surface instability. Localized remedial measures continued.

To a large extent the argument was sealed when a 100 m deep inclinometer was installed downstream of the dam. After a few weeks or so there appeared the characteristic shear step, at a depth of 80 m.

By early 1996 the following conclusions had been reached (see Figure 24).

- The area containing and to the south of the tailings embankment was a large deep-seated, primarily translational landslide.

Figure 24—The approximate boundaries of the landslide at Golden Cross

- ➤ The head of the slide was located upslope of the tailings embankment, while there was some evidence to indicate a toe region near the access road, a distance of over 1.5 km.
- ➤ The basal shear surface was along the low strength, slickensided zone located at or about the base of Omahia andesite. This shear zone was up to 80 m.

This was not quite the end of the saga. Others refused to accept that the landslide extended across the access road, some 1.5 km from the dam. On the basis of this blinkered view it was decided to build a 'stabilizing' fill uphill of the road. After much expenditure the net effect was to accelerate landslide movements.

Within a year the mine was closed.

Issues with cookbook rock mechanics and engineering geology

General concerns

Before proceeding to discuss a case study that illustrates much of what is unsatisfactory in some current rock mechanics and engineering geology, I should come clean on the heart of the problem. This relates to what I consider to be the abuse of rock mass classification systems.

The publications of the RMR classification system by Bieniawski in 1973 and the Q-system by Barton, Lien and Lunde in 1974 were greeted with great enthusiasm by a large portion of the international rock mechanics fraternity, particularly those involved in support design for rock tunnels. Here appeared to be a systematic, if not truly scientific, procedures for designing primary support.

Over the past 30 years the two classification systems have been proposed as being design tools for a wide range of structures. The RMR was modified to the MRMR by Laubscher for underground mining. Hoek, Kaiser and Bawden presented a 'trimmed' version of the RMR system, called GSI (Geological Strength Index), to be used for calculating rock mass strength via the Hoek-Brown failure criteria. Q has been proposed as a means for estimating a whole suite of rock mass characteristics, including TBM productivity.

Like a good Jamie Oliver recipe, these classification systems are easy to apply, and they have now become so widely used that sight has been lost of some of their limitations.

Fortunately, a few papers are now appearing that question the validity of designing using classification systems. A recent paper by Palmstrom and Broch (2007) provides an excellent critique of many of the parameters used in determining Q values. They point out that, notwithstanding claims by Barton to the contrary:

- ➤ the ratio *RQD/J_n* does not provide a meaningful measure of relative block size
- ➤ the ratio *JW/SRF is not a meaningful measure of the* stresses acting on the rock mass to be supported.

They also point out that the Q system fails to properly consider joint orientations, joint continuity, joint aperture and rock strength.

In essence, Palmstrom and Broch (2006) consider that the classification systems (Q and RMR) provide good checklists for collecting rock mass data, and may be of use in planning stage studies 'for tunnels in hard and jointed rock masses without overstressing'. They do not support the use of these systems for final designs.

I was part of the team that worked with Bieniawski in developing the RMR system and I think such classification systems are very valuable in communicating rock mass quality. They also have a value as the basis of recording empirical data but only if the correlations reached are on a sound scientific basis. Unfortunately many of the experiences of myself and my colleagues have led us to conclude that they can be inappropriate, and sometimes dangerous, when used for quantifying rock mass behaviour.

Firstly, there is the direct use of the RMR and Q systems in determining tunnel support. In an article published in *Tunnels and Tunnelling* (April 2007) we presented data from nine tunnelling projects in Sydney where the design support determined from the Q system proved to be substantially less than what had to be installed, even though the rock conditions encountered were as expected. In two cases failures occurred. Since that time there has been a further case study in Sydney where the Q system was being used to assess support as a tunnel was being advanced. A collapse occurred killing one of the tunnellers.

We have encountered similar issues with insufficient primary support determined in Q system based designs in tunnels in metamorphic rocks in Brisbane and Melbourne. In parallel with our experiences, Peck and Lee have shown that there is almost no correlation between Q-predicted support capacities and actually installed support in Australian mines. It is possible that none of the miners knows his business, but I doubt it.

Secondly, there is the use of GSI (a cut-down version of RMR) to determine rock mass shear strength parameters for the Hoek-Brown failure criterion, which in turn may be used for calculating support requirements. Here I am going to be treading on ground even more holy than Lang's bucket, but so be it.

Mostyn and Douglas (2000) provide a detailed critique of the Hoek-Brown failure criterion for *intact* rock. They conclude that:

'… there are inadequacies in the Hoek-Brown empirical failure criterion as currently proposed for intact rock and, by inference, as extended to rock mass strength. The parameter m_i can be misleading, as m_i does not appear to be related to rock type. The Hoek-Brown criterion can be generalised by allowing the exponent to vary. This change results in a better model of the experimental data.'

Mostyn and Douglas then proceed to discuss the Hoek-Brown failure criterion for rock masses, as given by the equation:

$$\sigma_1' = \sigma_3' + \sigma_c\left(m_b\frac{\sigma_3'}{\sigma_c} + s\right)^a \qquad [22]$$

where m_b and s are calculated from a GSI value. They note the following:

'The only 'rock mass' tested and used in the original development of the Hoek-Brown criterion was 152 mm core samples of Panguna andesite from Bougainville in Papua New Guinea (Hoek and Brown, 1980). Hoek and Brown (1988) later noted that it was likely this material was in fact 'disturbed'. The validation of the updates of the Hoek-Brown criterion have been based on experience gained whilst using this criterion. To the authors' knowledge the data supporting this experience has not been published.'

I considered it to be extraordinary that a failure criterion, widely used around the world, is based on such a paucity of data. Mostyn and Douglas discuss various improvements that should be made to the Hoek-Brown mass criterion for slope analysis but they too have only one case study plus a lower bound based on the shear strength properties of rockfill. They fully acknowledge the limited experimental data base. To my knowledge nobody has published a comparable study of this criterion for underground excavations.

I think matters are made even worse by the provision, through the computer program RocLab, of 'calculating' the rock mass 'modulus of deformation'. This is one area where, in the Sydney rock, we have plenty of good field experimental data. Table I shows this field data in comparison with the Hoek-Brown (RocLab) computed values. For these rock masses the RocLab values are nonsense.

I think that available evidence places the Hoek-Brown criterion for rock masses as no more than a hypothesis. It may be a good hypothesis, but until it is properly supported by, or modified as a result of, proper field experimental data it is not wise to use it as the basis of major design decisions. It has nowhere near the experimental foundation as the Mohr-Coulomb criterion has in the field of soil mechanics.

A case study

On 23 July 2006 a collapse occurred in the north-east face of the M1-K1 transfer cavern at the Chuquicamata open pit in Chile. Figure 25 shows the location of this cavern in relation to the pit, and Figure 26 shows the geometry of the cavern. Figure 27 shows some of the debris and destruction that shut down the cavern and all conveyor transport of ore from the in-pit crushers.

Space and legal constraints prevent a full discussion of this failure. However, the following can be recorded.

- ➤ Investigation boreholes drilled for the cavern were logged primarily in terms of Q-system and RMR values.

Table I

Field mass modulus values for hawkesbury sandstone

Class	GSI	Measured field values MPa	RocLab prediction MPa
I	≈ 75	1500 – 2500	21000
II	≈ 65	1000 – 1500	12000
III	≈ 55	500 – 1000	6500

Note:
Measured field values from:
Poulos, Best and Pells (1993) *Australian Geomechanics Journal*
Clarke and Pells (2004) 9th Aust-NZ Geomechanics Conference
Pells, Rowe and Turner (1980) Structural Foundations on Rank
Pells (1990) 7th Australian Tunnelling Conference

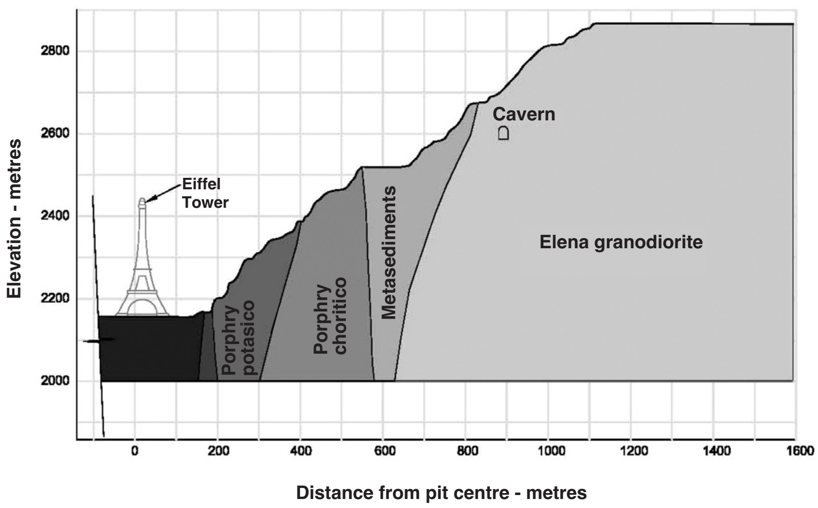

Figure 25—Location of chuquicamata cavern

- Mapping during excavation of the cavern was done primarily on the basis of Q-values, but with major faults being recorded.
- The installed support was designed partly on the basis of rock mass classification systems and partly on numerical analyses using parameters derived from the Hoek-Brown rock mass criterion.
- The conclusions of an independent audit of the failure were as follows:

  The collapse was a final manifestation of a widespread and general failure of the cavern support system. Failure was not a localized phenomenon particular to the north-eastern face. The inability of the cavern support to adequately reinforce and stabilize the surrounding rock was primarily a failure of the design.

Conclusion

I end this paper with a quote from a letter in *Time* magazine of 25 February 2008.

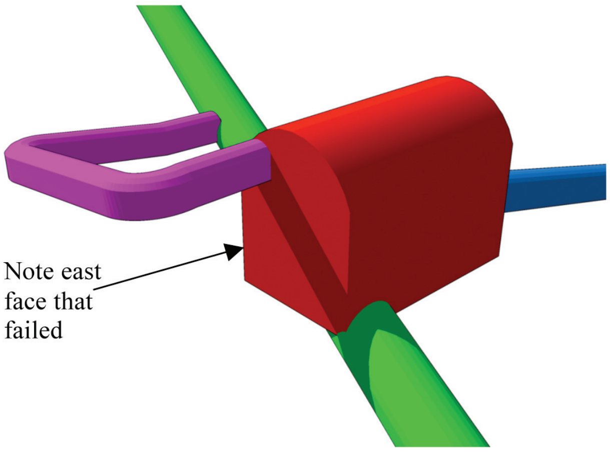

Figure 26—M1-K1 cavern

Figure 27—Debris and destruction from the collapse

'Such scepticism is commonly portrayed as a flaw, when in fact it's the single most valuable skill we can bring to bear on our work. Contrary to popular belief, good scientists don't seek to prove a hypothesis true. We make every possible effort to prove it wrong by subjecting it to the most withering attacks we can dream up. (It's actually great fun). This refusal to accept a new idea until it has run a gauntlet of testing is the very reason scientific 'truth' is so reliable.'
Paul G. FitzGerald, PhD, University of California

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