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Design of Roof Support of the Sydney Opera House Underground Parking Station

P. J. N. Pells, R. J. Best and H. G. Poulos

Abstract—*The paper presents the method for design of primary roof support for the large todoidal cavern constructed to house the Sydney Opera House parking station. The cavern was constructed with 7 m of rock cover beneath Sydney's Royal Botanic Gardens. Design of rock reinforcement using a combination of fully grouted rock dowels and Macalloy bars was based on control of horizontal shear movement along bedding features in the roof. A combination of linear-arch-type analyses snd non-linear jointed finite-element analyses were used in the design study, together with an analysis of the shear resistance offered by fully grouted bolts under shear deformation.*

Résumé—*L'article présente la méthode de dimensionnement du soutènement primaire du toit dans la grande caverne todoidale qui abritera le parking du Sydney Operat House. La caverne est prévue avec une couverture rocheuse de 7 m au-dessous du Jardin Botanique Royal de Sydney. Le soutènement rocheux utilisant une combinaison de cheville injectées et de barres Macalloy est basé sur la maîtrise du cisaillement horizontal le long des lignes d'appui du toit. L'étude a utilisé un ensemble d'analyses de type arche linéaire et d'analyses par éléments finis joints non linéaires, ainsi qu'une analyse e la résistance au cisaillement des boulons injectés sous l'effet d'une déformation par cisaillement.*

Introduction

The Sydney Opera House parking station had to be constructed beneath the Royal Botanic Gardens under extremely tight environmental constraints. No disturbance of the gardens, with their ancient Morton Bay fig trees, was allowed. Furthermore, the parking station had to be in close proximity to the Opera House, but was constrained by the twin tubes of the Sydney Harbour Tunnel, buildings of Government House, and the Opera House forecourt.

All construction work had to be undertaken from Macquarie Street, through tunnels that would form the final entry and exit access to the parking station. These tunnels had to pass over the top of the Sydney Harbour Tunnel—a requirement which dictated that the roof of the parking station had to be within about 8 m of the ground surface. A location plan is shown in Figure 1.

The car park consists of a twin spiral road wrapped around a central intact core containing linking roads and service tunnels (see Fig. 2). The car park was constructed entirely within Sydney Sandstone, which is characterized by elastic modulus in the range 500 MPa–6000 MPa; dominant north-south-trending vertical joints; and a horizontal stress field of about 0.5 MPa in shallow competent rock, increasing with depth by from one to two times the change in overburden stress.

Instrumentation and monitoring of the car park during and after construction have been presented by Pells *et al.* (1993) and the general design approach has been presented by Pells *et al.* (1991). This paper provides details of the innovative method used for design of roof support.

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Design and Analysis

The car park construction required excavation of a toroidal void 34 m high with a roof span of a nominal 17.6 m and a central core 35 m diameter. Local cutouts in the walls meant that in places the span approached 20 m.

The nominal 17.6 m roof span was significantly larger than had been constructed previously in Sydney Sandstone. This fact, combined with the thinness of the sandstone roof and the presence of vertical joints and horizontal bedding features, dictated a careful approach to design. The features relevant to the design of support for the roof are summarized in Table 1.

Design was carried out in the following stages:

1. Analyses were carried out to demonstrate that the roof would stand unsupported if no effective horizontal planes of weakness existed. In other words, if the Sandstone could be reinforced so that it would act as a pseudo-elastic beam at least 5 m thick, then no passive support would be required.

2. Elastic finite-element analysis was carried out and the horizontal shear stress along the two pronounced bedding features was evaluated.

3. Jointed finite-element analysis was carried out to assess the magnitude of shear movements that would be developed across the two bedding features in the absence of reinforcement.

4. Analysis was carried out on the contribution to shear resistance offered by proposed roof anchors and dowels, for a range of shear displacement.

5. The roof-bolting arrangement was selected such that horizontal shear resistance offered by the roof bolting arrangement under the shear deformation pattern calculated from the jointed finite-element model exceeded by a margin of safety the shear stress requirement obtained from the elastic finite-element analysis.

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Table 1. Summary of design parameters.

Parameter	Source	Adopted for Design
Roof span	Car park layout design	17.6 m
Sandstone cover thickness	Car park layout design	5 m (actual thickness 7 m)
Height of chamber	Car park layout design	34 m
Vertical joint spacing	Mapping of exposures	2 m
Spacing of horizontal bedding features	Mapping of exposures and logging of drill core	Two major bedding horizons at 1.5 m and 3.0 m above crown
Joint strength properties	Previous testing for socketed piles	Friction angle 25°
Bedding strength properties	As above	Friction angle 25°
Sandstone elastic modulus	Pressuremeter testing and monitoring of displacements around deep basements	1500 MPa
Initial horizontal stress	Measurement using borehole slotter	200 kPa

Analysis of Roof Incorporating Vertical Jointing

A series of analyses were carried out to demonstrate that the roof would be self-supporting despite the presence of numerous vertical joints, provided that it could be made to act as a continuous beam at least 5 m thick.

This analysis of the roof unit was carried out using an extension of the linear arch approach first published by Evans (1941). The method allows treatment of a unit of known thickness containing vertical features incapable of supporting tension. Tension develops at the base of the roof unit at the roof centreline and at the top of the roof at the abutments. The roof is modelled as an arch formed by the roof zone under horizontal compression. The method provides a valuable tool for calculation of likely roof sag as a function of span and roof thickness, and provides an estimate of peak horizontal compressive stress.

As originally developed, the method provides no means of including the effect of pre-existing horizontal stress in the roof, and implicitly assumes a zero initial horizontal stress field. Horizontal stress within the roof unit has an important effect on roof stability, as it reduces the depth of zones in horizontal tension.

An extension to Evans' theory was developed using theoretical contributions from Professor John Booker of Sydney University, in which the stress distribution in the roof arch is computed using a series of one-dimensional beam elements, with the thickness and vertical position of each element controlled by the calculated region of horizontal compression.

Another factor introduced into the extension of the linear arch method was treatment of the rigidity of the

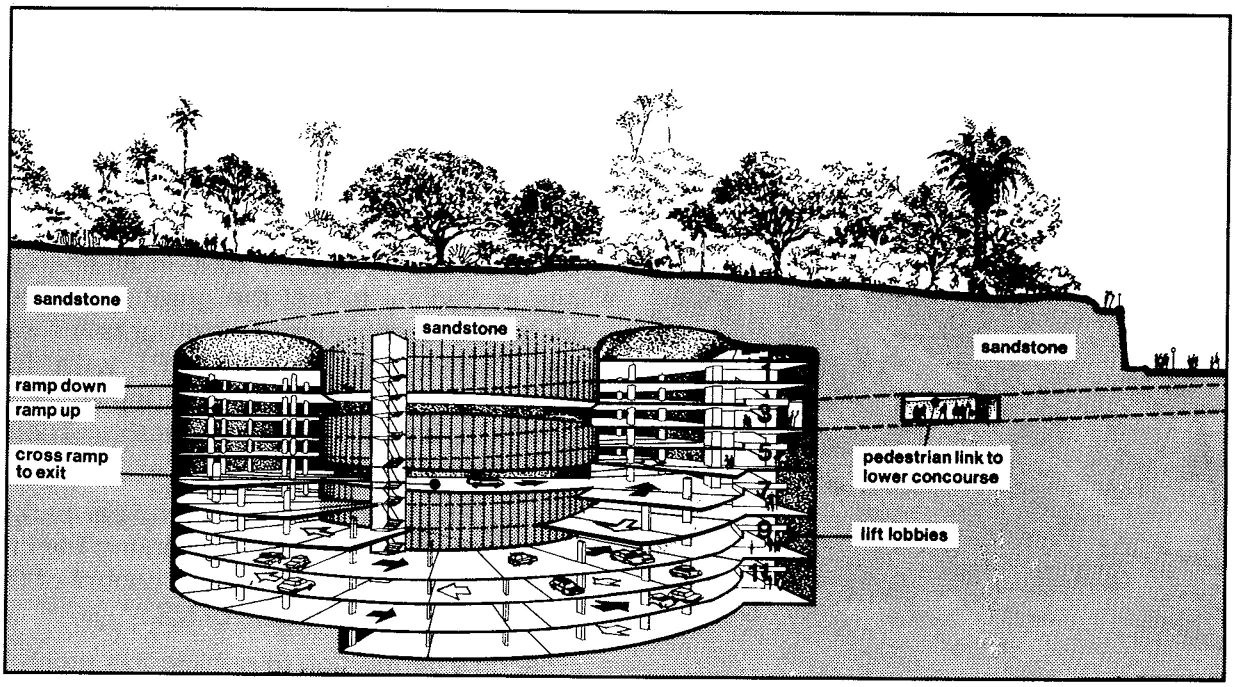

Figure 1. Car park structure.

abutments. Central deflection decreases with increasing stiffness of abutment rigidity. This effect was modelled by representing the abutments as elastic units with a rigid boundary at specified distance into the rock mass adjacent to the excavation.

Figure 2 presents the results of modified linear arch analysis of the design case of a 5-m-thick roof unit with horizontal prestress of 200 kPa, sandstone modulus of 1500 MPa, and an assumed 15-m-wide stress transfer zone at each abutment. For the self-weight-loading of 200 kPa, a mid-span deflection of 9.5 mm was calculated. Figure 2 shows the calculated distribution of horizontal tension in the lower half of the roof at mid-span, and in the upper half of the roof at the abutments.

Additional analyses were carried out to assess the sensitivity of mid-span deflection to initial horizontal stress and the rigidity of the abutments. The results of these analyses are presented graphically in Figure 3. For the conditions anticipated, mid-span deflections were expected to be in range 5 mm–12 mm. Figure 3 illustrates the importance of the ini-

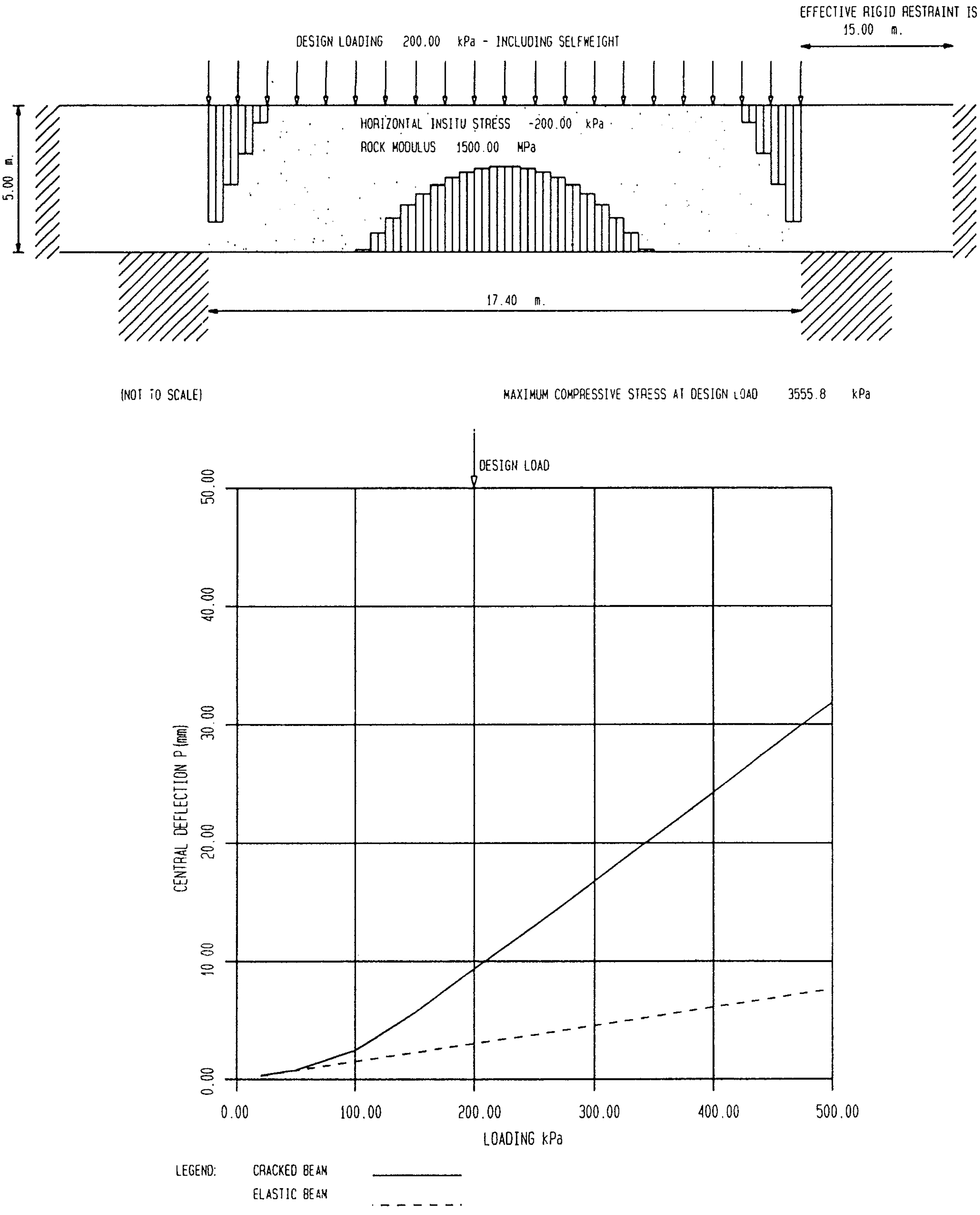

Figure 2. Linear arch analysis—design case.

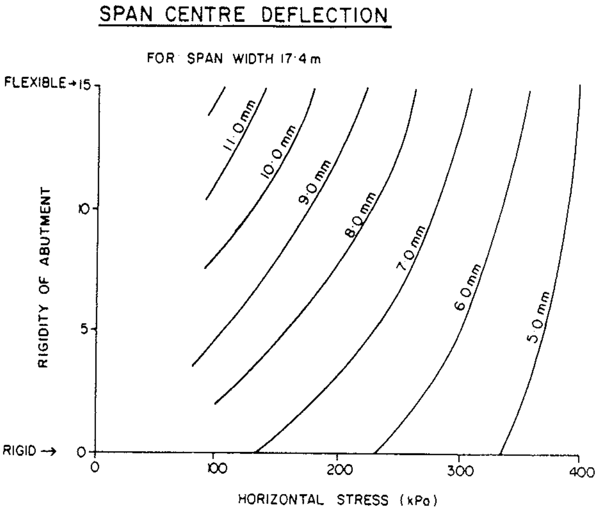

Figure 3. Variation of sag with horizontal stress and abutment rigidity.

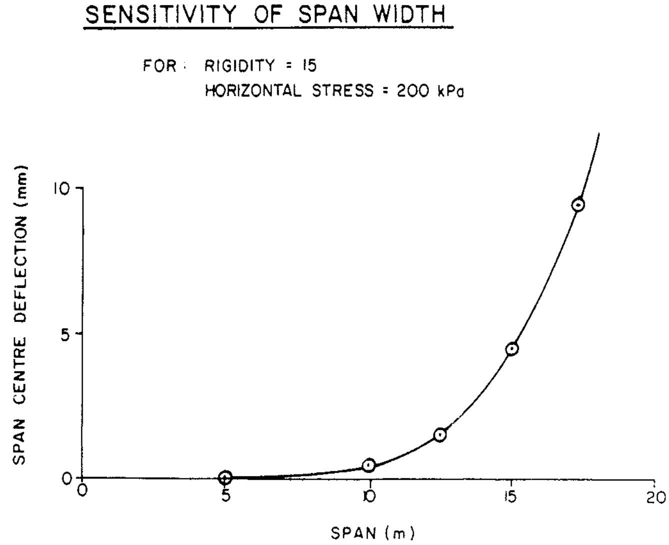

Figure 4. Sensitivity of sag to roof span.

tial horizontal stress field in the roof on roof deflection.

Excavation commenced at the top of the chamber due to access constraints. Measurements of the stress at roof level carried out by James Cook University using a borehole slotting device indicated horizontal stress of 200 kPa. The horizontal stress at the roof level was expected to increase during excavation of the chamber to full depth as a result of the transfer of stress originally carried by the excavated sandstone. Thus, the worst conditions for roof stability occurred at the commencement of construction, under conditions of least horizontal roof stress.

Figure 4 shows the form of the relationship between span and mid-span deflection. This relationship was valuable in assessing results from monitoring of roof performance during construction. For spans greater than 15 m, deflection increases rapidly with span, indicating a need for special precautions where the effective roof span was increased, at the point at which the vehicle entry tunnel (12-m span) joined the main chamber. Steel sets were installed to support the vehicle entry tunnel to provide additional roof support, and similar measures were adopted at other locations where the effective span of the roof was increased.

The modified linear arch analysis demonstrated that the roof would be self-supporting, provided that it acted as a single unit without excessive shearing along horizontal bedding. The design approach taken was to specify roof bolting sufficient to carry the shear stress that would develop along the bedding features in the roof, in order to ensure that the roof sandstone acted as a single unit. Roof central deflection at completion of excavation of the chamber was anticipated to be approximately 10 mm.

Finite-Element Analyses

Two finite-element analyses were carried out to provide an assessment of horizontal shear requirements on bedding. A linear finite element analysis was carried out and the shear stress along the bedding features calculated in the absence of shear movement. The calculated stresses at various positions along the two bedding horizons are tabulated in Table 2.

A jointed finite element model was analysed to assess the potential shear movement across bedding in the absence of rock bolting. Modelling of rock defects consisting of bedding partings and vertical cracks was performed using elasto-plastic joint elements with limited tensile capacity. Modelling was carried out using COFSTRS, a jointed finite-element program developed by Coffey Partners International Pty. Ltd. (Coffey 1989). Excavation was proposed to be carried out by staggered advance of three headings, and this development process was simulated in the modelling.

Figure 5 shows the deformation pattern obtained. Deformations have been exaggerated by a factor of 100 to illustrate the model behaviour. The horizontal slabs formed by the bedding partings behave independently, with tensile opening at the base of each slab at centre span and shear displacement across the bedding partings. Figure 6 shows the distribution of shear movement across the bedding planes.

Analysis of Action of Reinforcement

The contribution of reinforcing bolts to shear resistance across joints was analysed by considering the following components:

- Lateral resistance developed by the bolt via dowel action.
- Increased joint frictional resistance by virtue of increased normal stress induced by the bolt.
- The component of the bolt axial force acting against shear movement.

Additional normal stress across a bolted joint or parting is equal to the normal component of axial tensile force in the bolt. Three components contributing to this force were considered:

1. Dilatancy of the joint during shearing.
2. Axial force developed due to lateral extension of the bolt.
3. Prestress in the anchor.

Figure 7 shows the increase in shear resistance contributed from a Y24 roof bolt and a Macalloy bar for reinforcing installed normal to the defect, having a 15° dilatancy angle and a friction angle of 25°. Application of prestress to the reinforcement causes significant installation difficulties while leading to only modest improvement to shear resistance (see Pells *et al.* 1991). For this reason, only the Macalloy bars were pretensioned.

Evaluation of the Reinforcement

Based on the results of the above analyses, it was possible to assess the effectiveness of roof reinforcing by comparing the shear resistance available, with horizontal shear calculated from elastic finite-element analysis for each section of the roof considered.

Table 3 shows the results of these calculations for the adopted roof reinforcement, illustrated in Figure 8. The factor of safety, calculated as the ratio of available shear capacity along partings to calculated shear stress, averages 2.3 for the lower roof parting and 2.2 for the upper roof parting.

Table 2. *Calculated joint shear.*

Zone	Horizontal Shear Stress Elastic FEM (kPa)	Normal Stress Elastic FEM (kPa)	Shear Deflection Jointed FEM (mm)	Shear Contribution per Reinforcing Bar	
				(Y24) kN	(Macalloy) kN
1 upper 1 lower	130 185	200 320	4.2 6.8	85 120	435 450*
2 upper 2 lower	120 150	120 155	5.4 7.7	95 130	450* 450*
3 upper	50	50	1.7	50	350 (50°) 200 (90°)
3 lower	50	70	2.4	50	370 (50°) 220 (90°)
4 upper	40	70	2.2	50	360 (50°) 205 (90°)
4 lower	45	105	1.8	50	350 (50°, 200 (90°)
5 upper 5 lower	110 140	90 45	3.0 3.5	70 75	390 410
6 upper 6 lower	125 180	230 350	2.5 3.0	60 70	375 390

* Shear limited to nominal yield capacity of reinforcing.

Monitoring

Monitoring of deflections in the roof of the chamber during construction was carried out to measure roof behaviour for comparison with design analysis. Pells *et al.* (1993) describes results from monitoring program, which confirm that roof deformations were consistent with expectations.

Measured roof sag deflections agreed well with design predictions (see Pells *et al.* 1993); peak roof vertical movement of 12 mm was observed. Some of the vertical movement measured can be attributed to vertical compression of the core pillar so that roof sag deflections were within the calculated design values.

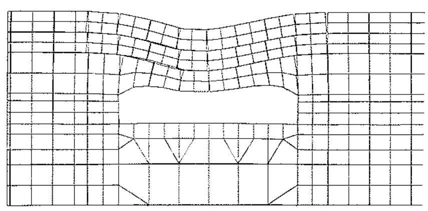

Figure 5. Roof deformation pattern—jointed analysis.

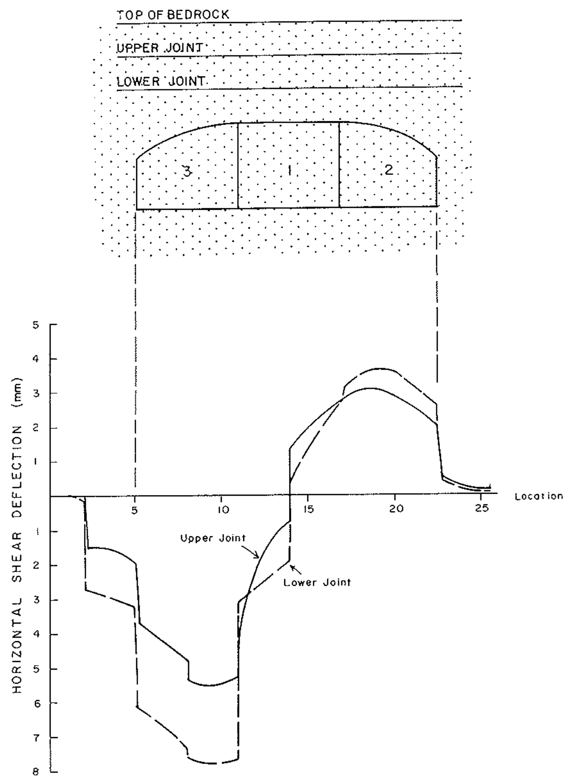

Figure 6. Horizontal joint shear in roof beam. (Note: +v θ shear deflection denotes relative movement to the right of the upper layer over the lower level.)

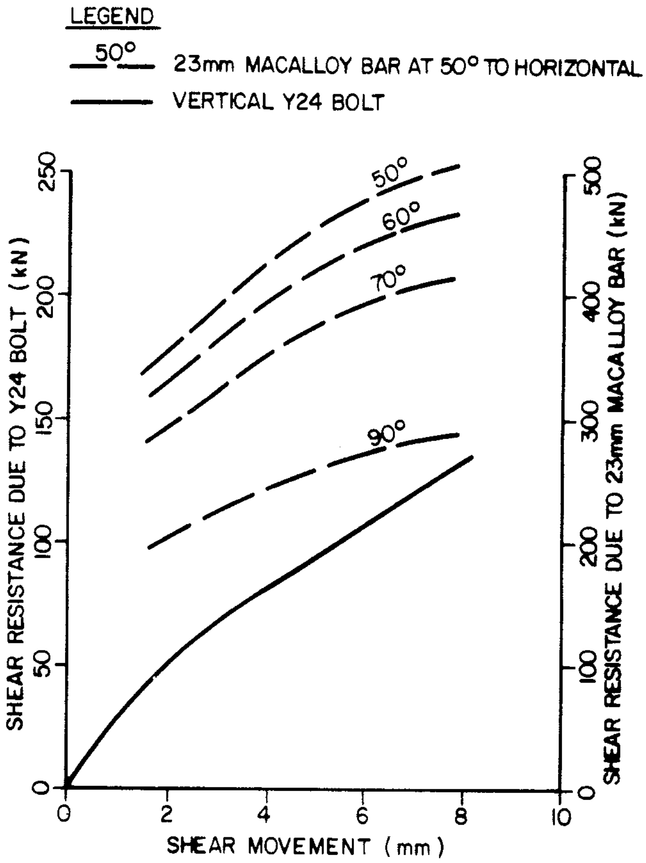

Figure 7. Shear resistance offered by reinforcement.

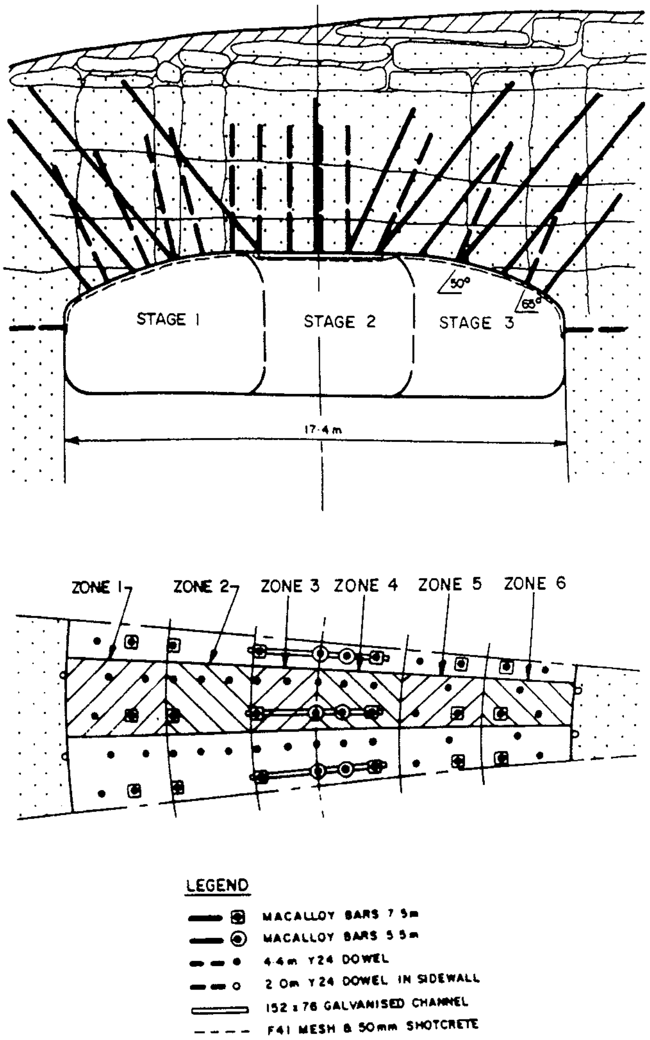

Figure 8. Roof reinforcement design.

Table 3. Evaluation of reinforcement.

Zone	Area (sq. m)	Number of bolts		Horizontal Shear Elastic FEM (kPa)	Shear Resistance Due to Bolting Plus Joint Shear (kPa)	Factor of Safety
		Y24	Macalloy			
1 upper 1 lower	7.6	3	1 @ 50°	130 185	185 255	1.42 1.38
2 upper 2 lower	7.6	3	1 @ 50	120 150	155 180	1.29 1.20
3 upper 3 lower	5.2	2.5	1 @ 50° 0.5 @ 90°	50 50	135 150	2.70 3.00
4 upper 4 lower	4.8	2.5	1 @ 50° 0.5 @ 90°	40 45	225 240	5.62 5.33
5 upper 5 lwoer	5.1	2	1 @ 50°	110 140	145 130	1.32 0.93
6 upper 6 lower	4.7	2	1 @ 50°	125 180	210 275	1.68 1.53

Conclusions

The approach and calculations for design of roof support for a shallow wide-span opening has been presented. The key feature of the design was control of shear along horizontal bedding features. The approach presented provides a means of obtaining using quantitative methods—a design which results in ductile rock joint behaviour. □

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