AN APPROACH TO THE SUPPORT OF VERTICAL CUTS IN WIANAMATTA SHALES

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Abstract

The support of vertical cuts in closely jointed, weathered shale subject to moderate loads has been provided through a system of rock bolts and shotcrete at a number of sites around Sydney. This paper presents the design approach adopted and an overview of the final support and some difficulties experienced during construction of the cut and cover tunnel and station box excavations at the Olympic site rail loop.

1. INTRODUCTION

One of the many successful aspects of the Olympic precinct is the rail access to the heart of the area. Part of the success is due to the minimisation of the infrastructures visual impact, and the freeing up of surface movement, through utilisation of driven and cut and cover tunnels.

To enable construction within the narrow strip of land available, the typical cut and cover tunnel profile comprised pre-cast concrete Bebo arches founding at the immediate rear of vertical cuts of up to 8m height in materials ranging from Class II (Pells et al, 1978) shale to residual clays. A similar story is realised for the station construction whose wide span and architecture sees loading at the edge of the structure's excavation.

Support of the loaded vertical cuts is provided using soil nails and rockbolts with a face membrane of steel fibre reinforced shotcrete.

2. SITE GEOLOGY

The Olympic Park site is underlain by Ashfield Shale which is part of the Wianamatta Group. Borehole evidence suggests that the subsurface materials at the site comprise the two lowermost members of the shale which are:

- Rouse Hill Siltstone, composed of dark grey to black sideritic silty claystone, which grades upwards into;
- Kellyville Laminite Member, which comprises a grey fine sandstone - siltstone laminite.

The siltstones and laminates of the two shale members have typical unconfined compressive strengths of 15 MPa to 25 MPa when fresh or slightly weathered becoming progressively weaker (to very stiff to hard soil strength) in the weathered horizon. In unmodified sections, the weathering profile is relatively deep:

- residual clay soils typically comprise a thickness of up to 3m;
- highly weathered shale extends to a maximum depth of 6 to 7m.

3. DESIGN PHILOSOPHY

3.1. Rockbolting

The design philosophy for excavation support in the shale bedrock is based on the assumption of short continuity joints dipping out of excavations faces. The design philosophy assumes a high probability that, over one or more 'patches' along each wall alignment, a joint set is sub-parallel to, and dipping out of the wall.

For the Homebush Rail Link site, rockbolts within shale are designed to resist an applied load generated by a wedge of rock, defined by;

- a rear slide surface striking parallel to the cutting ,
- dipping out of the face at any angle between 35° and 55°,
- with a maximum down-dip continuity of 3m,
- a horizontal top surface created by bedding,
- a continuity along strike of greater than 5m, and
- side release along near vertical, short continuity joints.

The design block is illustrated in Figure 1. Its dimensions are based on an assessment of the statistical mapping and borehole joint inclination data presented in the form of histograms and the stereoplot presented in Figures 2 to 5.

The design block is presumed to be ubiquitous, that is it may occur anywhere in the cutting faces. The assumption that the design block is ubiquitous has to be made when there is no way of knowing, in advance, the true locations of joints in space. Only the probable orientations and continuities are known. Clearly the key feature of this concept is that it does not invoke some form of active earth pressure wedge extending from the toe of the wall to the crest.

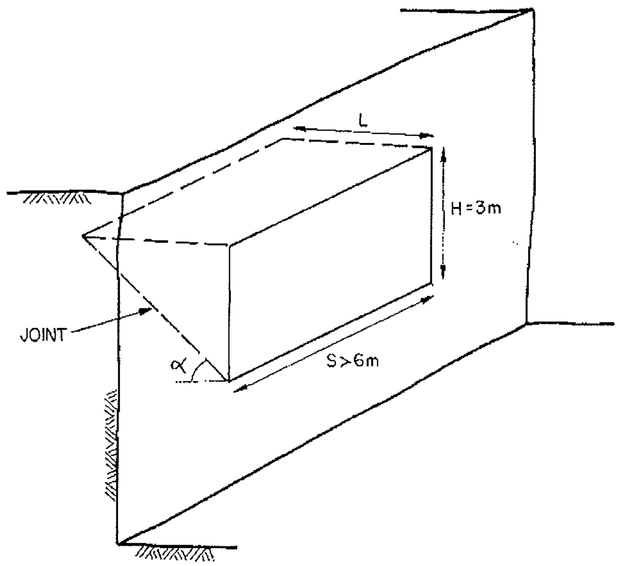


Figure 1 Design Block

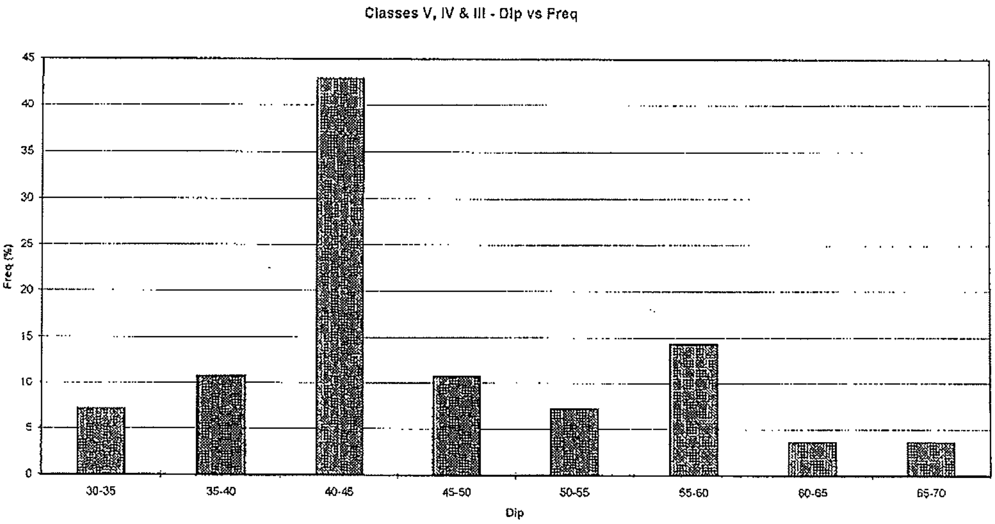
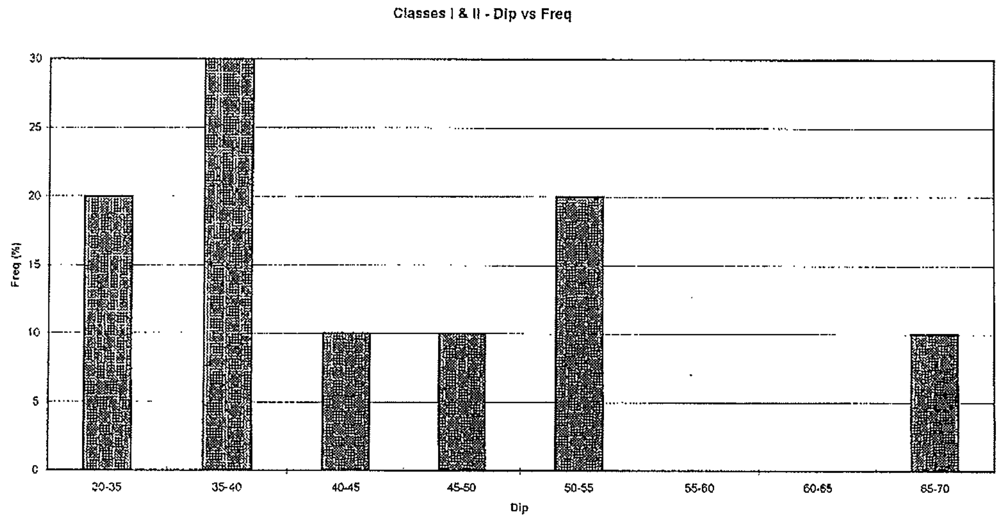


Figure 2 Histogram of joint dip frequency in Class V to III Shale.

Figure 3 Histogram of joint dip frequency in Class II & I Shale.



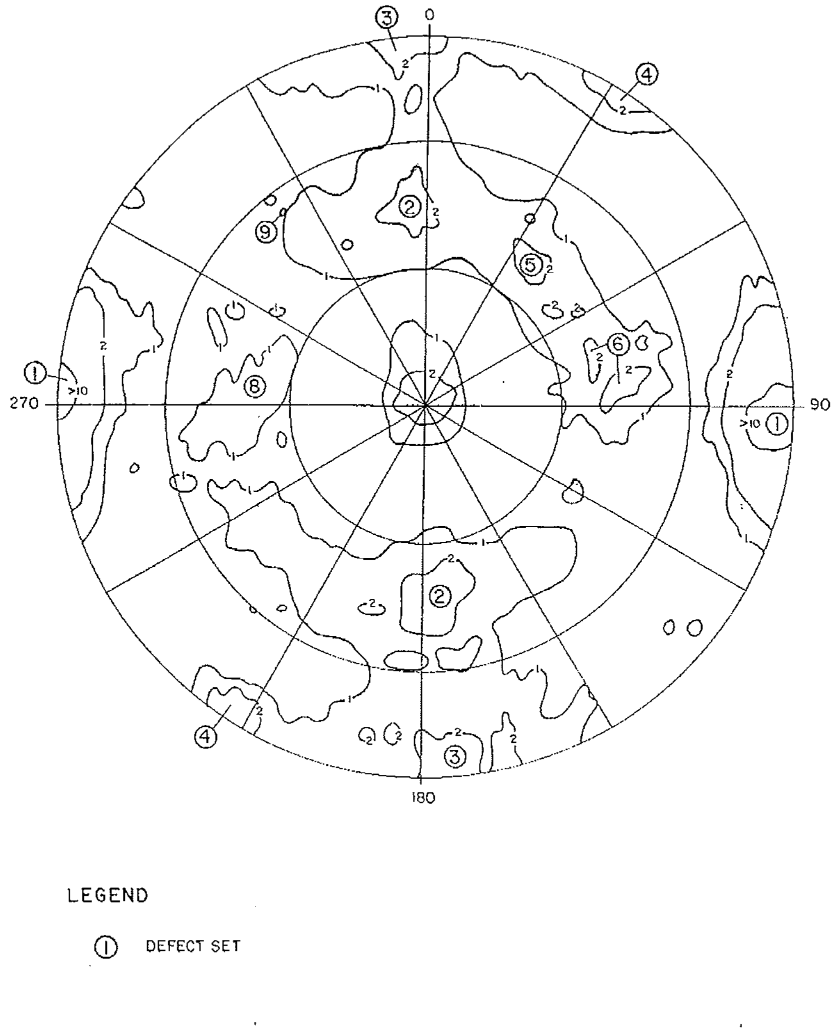


Figure 4 Stereoplot of defects recorded at the Homebush Bay area.

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Line mapping traverse
Data Histograms

case no.: All Joints

project:	Homebush Bay Rail Link	datum:	
location:	Pits adjacent to Parkview Drive	date:	27-Aug-96

Data records 60 Pit: Domain: Lithology: Type Defect: jn, jn, etc.
Traverse: Length Continuity Spacing Water Roughness Filling Thickness ILA Wave length
1, 2, etc.

Note: Unless specified all data types are included. Note also "etc" indicates complex query.

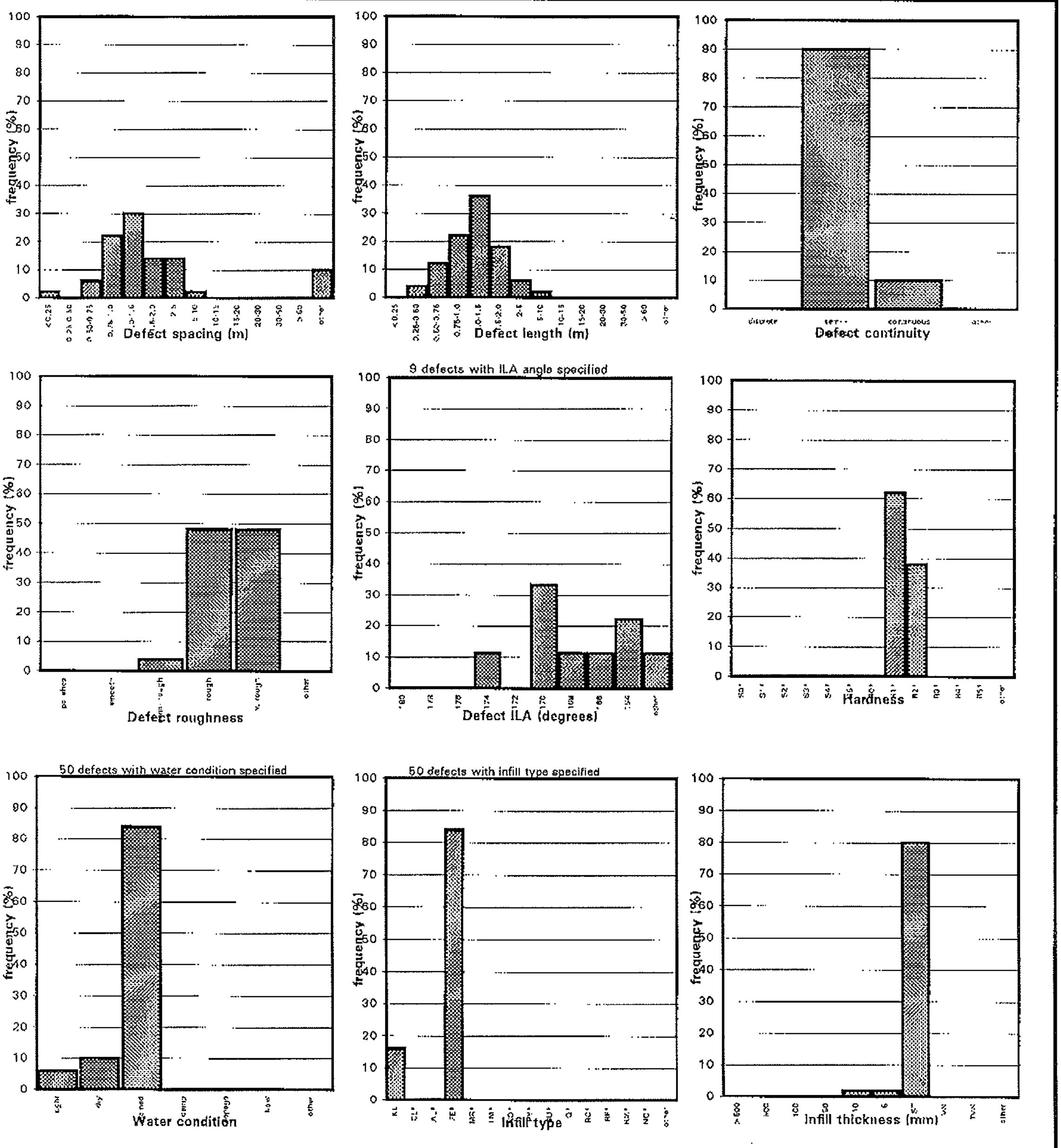

Figure 5 Histograms of joint characteristics.

3.2. Fibrecrete

Fibrecrete thickness design is based on the assumption of the formation of a plastic hinge under an ultimate loading condition. The fibrecrete is assumed to deflect between two linear supports. The plastic moment (M_p) is equal to the Modulus of Rupture (Mr) determined from flexural strength testing. The plastic moment is defined as;

$$M_P = \frac{3}{2}\frac{\sigma_t I}{y} \quad (1) \qquad and\ I = \frac{t^3}{12}; y = \frac{t}{2} \quad (2)$$

where t is the fibrecrete thickness, and

σ_t is the tensile concrete strength (ie flexural strength).

From limit state theory $pL^2 / 4 = M_p$, hence for fibrecrete with at least $50kg/m^3$ of steel fibres, the ultimate load capacity is given by $p = 4t^2 / L^2$. The allowable fibrecrete load is obtained by applying a Factor of Safety of 1.5 to the ultimate load capacity, and the allowable fibrecrete tensile strength is obtained by applying a Factor of Safety of 0.75 to the flexural strength.

3.3. Design Safety Factors

The design adopts the following factors based on the Austroads Bridge Design Code, given the nature of the structures involved.

- the tangents of friction angles will be multiplied by 0.8 in calculating the out-of-balance horizontal force generated by the sliding block,
- the mass of a design wedge will be multiplied by 1.1,
- vertical line loads on footings supporting cut and cover tunnel roof and on bored piers will be increased by a factor of 1.2 for any dead load component and 1.4 for any live load component,
- ultimate bond shears will be multiplied by 0.5 to determine the generalised allowable design resistance, and
- maximum steel stresses will be limited to 80% of yield stress.

The above safety factors are combined in a MathCad computer routine for which the active force required to be resisted by soil nails/rock bolts, P_{act}, is given by the following:

$$P_{act} = \frac{\tan\alpha - 0.8\tan\phi}{1 + 0.8\tan\alpha\tan\phi}\left(\frac{1.1\gamma H^2}{2\tan\alpha} + 1.2P_{dead} + 1.4P_{live} + \frac{1.4Hq}{\tan\alpha}\right) + H\sigma_w \quad (3)$$

Where:

	α	=	angle of defect/joint (ie sliding plane - See Figure 1)
	H	=	height of design wedge (See Figure 1)
	ϕ	=	angle of friction of shale
	γ	=	unit weight of shale
	θ	=	surface surcharge load
	P_{dead}	=	vertical dead load on footing (Cut & Cover Tunnel)
	P_{live}	=	vertical live load on footing (Cut & Cover Tunnel)
	σ_w	=	water pressure (5 kPa) allowing for a perched water table 0.5m deep occurring anywhere on bedding.

4. DESIGN LOADS

Vertical loading on the design wedges in shale bedrock of between 50kN/m and 1000kN/m results from retained soils, surface surcharge loadings and from footings supporting cut and cover tunnel roof sections.

4.1. Vertical Load Transfer Through Shale Bedrock

Loading is decreased as the top of the wedge is located further below the rock surface or tunnel roof footing. Assessment of the reduction is based on and FE elastic analysis. The design includes the reduction in loading with increased depth given in Table 1.

TABLE 1

CLASS SHALE ROCK	APPLICABLE DEPTH RANGE BELOW LOADED ROCK SURFACE	% LOADING USED IN DESIGN
V-III	0 – 4m	100
	4 – 6m	60
	>6m	40
II-I	0 – 2m	100
	2 – 4m	50
	>4m	0

5. DESIGN PARAMETERS

5.1. Rock Bolt Steel Capacity

Rock bolt design layouts are based on the Ingersoll Rand CT-M22 bolt. The bolt has a published yield load capacity of 250kN (fully grouted) and incorporates a steel bolt within a deformed plastic sleeve.

5.2. Ultimate Grout/Rock & Grout/Bolt Bond Shear

Typical grout/rock bond shear values in shale presented in Table 2 were adopted for design.

TABLE 2

CLASS OF SHALE	PEAK BOND SHEAR STRESS (kPa)
I & II	1000
III	400
IV/V	200

However, field testing of the initial CT bolts being manufactured in Australia at the time found the bond shear developed between the plastic sheathing and the grout resulted in an upper bound load capacity. Consequently, the capacity of the bolts in the design was reduced.

Time limitations on the overall project completion and requirements for a long design life did not allow the reduced capacity to be improved for this project. Since this time it is understood the manufacture of the deformed sheathing has been improved to the equivalent of the original Scandinavian product to allow full load capacity to be developed and CT bolts have been used extensively on other projects such as the M5 tunnel.

5.3. Joint Friction Angles

Friction angles for sliding along the inclined joints forming the rear failure surfaces are:

Class I and II shale
Joints of roughness R3 and R4 with UCS of joint wall rock of 10 MPa or greater — $\phi = 36°$

Class III to V shale
Joints possibly clay coated, smooth but not slickensided. — $\phi = 29°$

These friction angle values are based on:

- Barton-type block sliding tests on joints in borehole core which gave values between 38° and 54° (mean 42°),
- Triaxial and direct shear tests on smooth bedding surfaces (Ghafoori, Carter, Airey, 1983), and
- experience with completed rockbolt walls in Ashfield Shale at North Ryde (Time Life Building), Strathfield (Inner West Hospital), Lidcombe (Inner West Hospital), Crows Nest (Mater Hospital) and Devlin Street underpass.

The design also incorporates internal drains and face drains behind the fibrecrete.

6. SUPPORT DESIGN

A typical excavation support design is presented as Figures 6 and 7. It should be noted the 12mm galvanised trench mesh placed horizontally over each row of bolts is to provide the load transfer mechanism from the fibrecrete membrane to the bolts as assumed in the design.

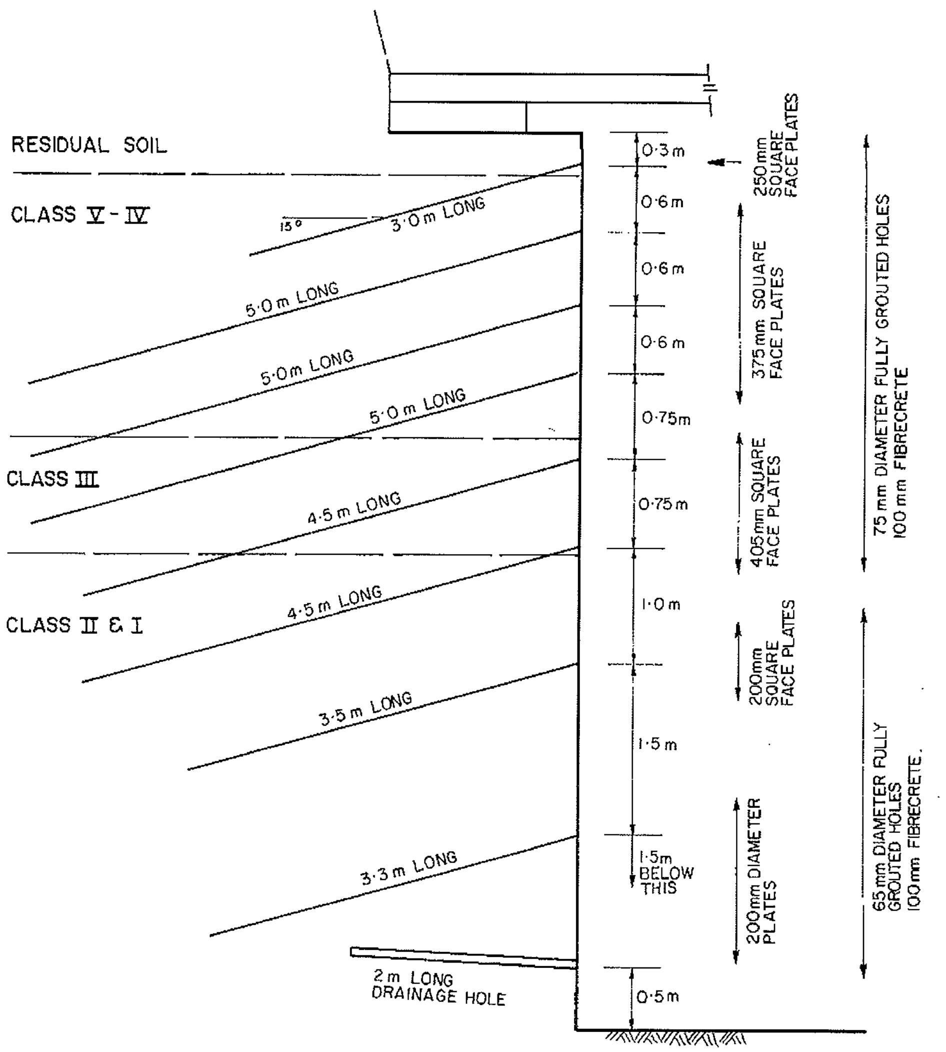

Figure 6 Typical Design Support.

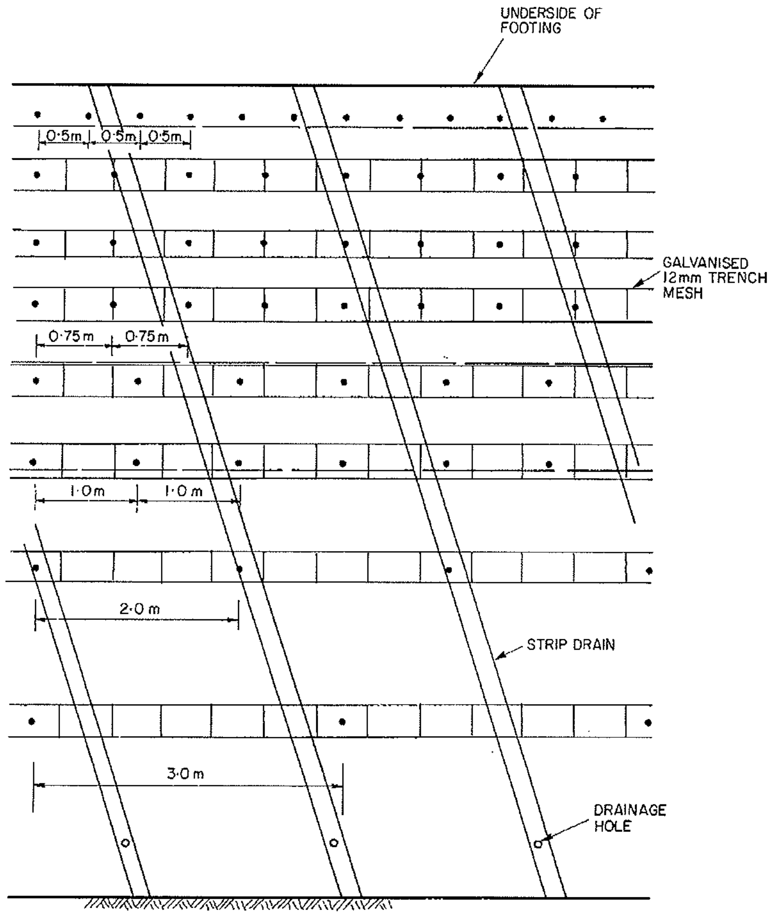

Figure 7 Elevation view - typical design support.

7. CONSTRUCTION

The design intention was to excavate and provide support in a series of intermediate cuts. However it was found during construction that the majority of cuts did not require intermediate support. In addition, accurate mapping of the exposed rock faces enabled the design to be relaxed in areas in areas where the 'assumed' design block was not present. Wider bolt spacings and a thinner fibrecrete membrane could be adopted.

Probably the most limiting detail of the design from the contractors point of view was the detail around the trench mesh/fibrecrete/bolt head shown in Figures 8 and 9. The detail of cranked trench mesh was found to be difficult and time consuming to construct, and through discussion with site personnel, was modified to the application of a thin layer of fibrecrete (say 50mm), placement of the trench mesh and bolts, followed by a final fibrecrete membrane layer. Plate 1 shows the initial layer of fibrecrete, bolts and mesh.

A further point of note in terms of constructability, the application of angled strip drain across the cut faces was difficult to achieve due to the height of the cuts and requirement for using cherry pickers on rough ground.

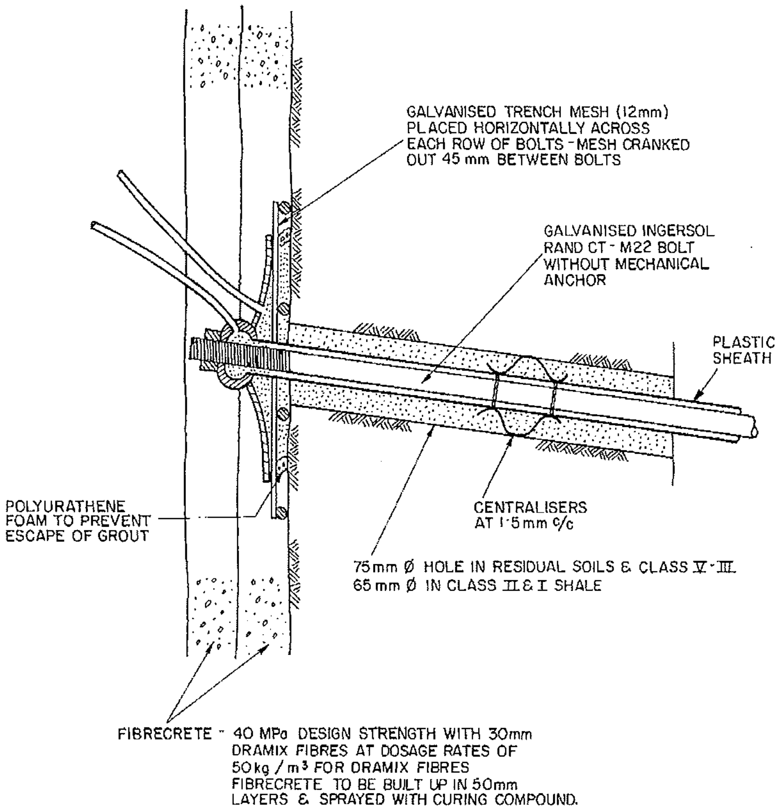

Figure 8 Bolt head, fibrecrete and trench mesh layout in section.

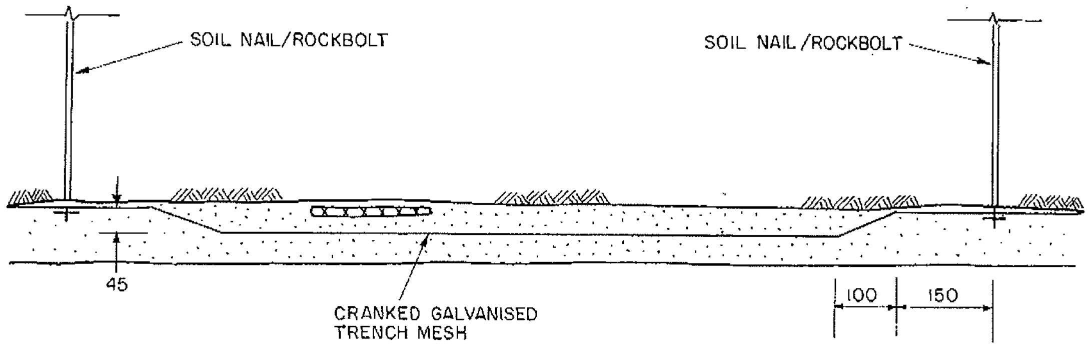

Figure 9 Bolt head, fibrecrete and trench mesh layout in plan.

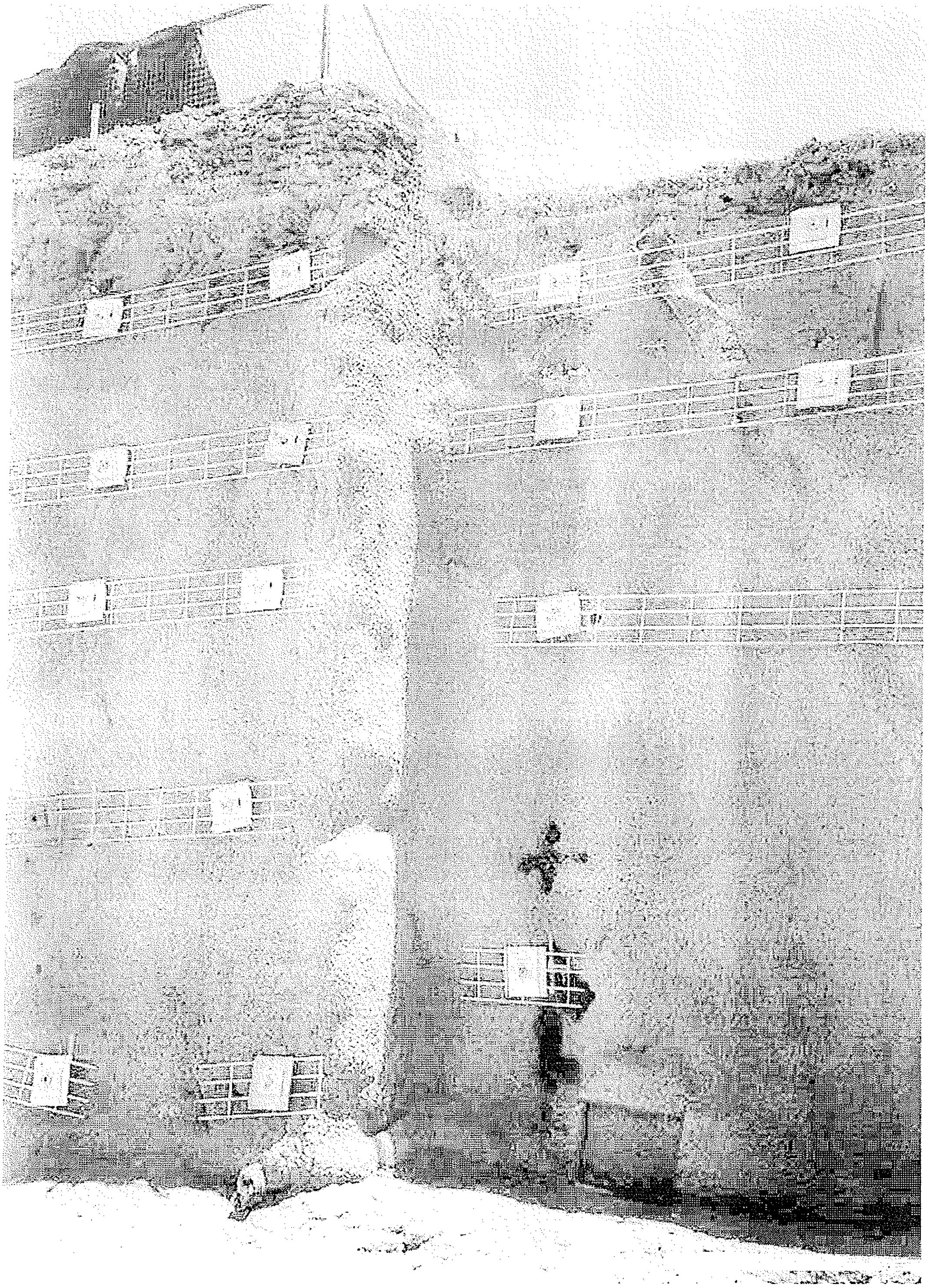

Plate 1 Initial layer of fibrecrete, bolts and mesh.

8. CONCLUSION

The design method presented here, which is based on the ubiquitous occurrence of limited size design blocks has provided designs which have performed well at six different sites around Sydney at heights of up to 8 metres.

REFERENCES

1. Pells et al, Foundation on Shale and Sandstone, Australian Geomechanics Journal, 1978.

2. Ghafoori, M., Carter, J.P. and Airey, D.W. Anisotropic Behaviour of Ashfield Shale in the Direct Shear Test. Geotechnical Engineering of Hard soils - soft Rocks, Anagnostopoulos et al. (eds). 1983 Balkema, Rotterdam.