PART 2

NOTES ON THE PREPARATION OF GEOTECHNICAL MODELS FOR UNDERGROUND WORKS

1. INTRODUCTION

There is no doubt that the formulation of an appropriate geological/geotechnical model for the site of an underground structure, be it a tunnel or deep basement, is the most important facet of the investigation/design process. This is a truth which has been presented in numerous papers and texts and there is no point in wasting more paper in repeating what is already well documented. However, it has been my experience that much energy in formulating geotechnical models for underground works is misdirected. This is for two reasons:

(1) Engineers are typically trained to be deterministic, i.e. we have in our psyche the desire to determine accurately the locations of specific geological features, such as stratigraphic horizons, joints, shears etc. Project after project has shown that, other than for macro-features, this deterministic approach is frequently quite futile.

(2) Our ability to deal with the full complexity of geological reality in our computation methods for the design of underground works is very limited. There is no point in gathering complex geological structural data or measuring sophisticated engineering parameters if we cannot use the information. The literature is littered with cases of great time and expenditure being directed to measuring properties such as the normal and shear stiffness of joints, the non-linear stress strain properties of soils and weak rocks, numerous cuttability and abrasivity indices etc. etc. where there is no realistic use for the resultant information. The words of Poulos and Brown (1986) should always be remembered, namely "it is far more satisfactory to use a simple model with parameters whose significance is readily understood, than to use a more complex model, the significance of whose parameters is obscure".

Numerous projects, a few of which have been discussed in Part 1, have led me to some conclusions regarding formulation of appropriate geological models. These may be grouped under the following headings

- planning and conducting the site investigation
- formulation and presentation of the model
- verification during construction

and are discussed in the remainder of this Part.

2. PLANNING AND CONDUCTING THE SITE INVESTIGATION

A frequent failure of investigations (in this sense failure means lack of value for money) arises from an attitude expressed by "let's log, test and measure everything we can think of because we do not really know what we need for design and construction".

Time must be spent up-front deciding what use is going to be made of the investigation and testing data before undertaking drilling, logging, in-situ testing and laboratory testing. The first question which should be asked is "what geological features and engineering parameters are going to have a significant impact on the design and construction of the project?". List these features and design the programme accordingly. The following is a good starting list:

(i) overall geological structure;
(ii) petrological descriptions with particular reference to quartz content and swelling/reactive minerals;
(iii) continuity and orientation of defect systems (joints, bedding, schistosity);
(iv) groundwater regime and rock mass permeability;
(v) natural stress field;
(vi) location in space of important macro structures such as faults and dykes;
(vii) the relative stiffness of the rock mass units and the anisotropy in strength and stiffness of the units;
(viii) the unconfined compressive strength (UCS) properties;
(ix) durability characteristics;
(x) abrasion characteristics.

Items which are usually not worth pursuing to any extent include:

- detailed geological description, and hence categorisation, of rocks which have similar engineering properties;
- absolute values of rock mass modulus (unless displacements around the tunnel or excavation are critical);
- accurate definition in space of defects which form parts of sets, i.e. joints, schistosity, bedding planes and minor faults;
- laboratory tests to measure triaxial strength parameters of rock, normal and shear stiffnesses of joints, shear strengths of defects which occur in sets, abrasivity and drillability indices.

It is not the purpose of this paper to address the mechanics of gathering the data set out in the ten point list above. These are largely routine matters covered by good texts on engineering geology. However, I think there are a few observations worth making, be they in some cases controversial, in relation to the listed matters. These are set out below.

2.1 Geological Structure

Successful interpretation of the geological structure for an underground project depends on having access to a very good engineering geologist - and there are not many of them around. This person must be able to "read" the geology in terms of the engineer's requirements and must be able to understand the depositional and tectonic processes which controlled the development of the piece of ground in which the structure is to be located. There are many good gatherers of geological data - there are few who can distil the essential features and point out to engineers the important geological nuances they would not see.

Notwithstanding the importance of good geological expertise, it remains the writer's firm conviction that the engineer responsible for design of an underground structure should work through the geology in the old fashioned way of hand drawing plans and sections. This is discussed in further detail in Section 3 of this part (Part 2).

2.2 Petrological Descriptions

It is my experience that much useful information can be gained from petrographic examination of thin sections, or in some cases XRD studies. However, the tunnelling engineer needs to know what he is after from the petrologist, otherwise he will be overwhelmed with detailed mineralogical descriptions which he will have difficulty in believing are written in English. Basic information should always include:

- approximate proportion of quartz (usually silt size or coarser);
- particle size range of quartz and other very hard minerals;
- presence of reactive minerals such as smectite which could cause slaking and/or swelling.

In cases of complex geology (usually involving metamorphism or volcanic origin materials) thin section studies can be very helpful in sorting out the stratigraphy. In such cases it is worth while for the engineer/engineering geologist to sit down with a petrologist and explain the problem before embarking on numerous thin section studies.

For example, in preparing the geological model for the site of the proposed Burns Peak tunnel in Port Moresby, the writer had difficulty in relating the core and surface exposures to the regional geology. However, it did not take a consulting petrologist very long to determine significant differences in materials which, in hand specimens, appeared the same. The resulting geological model (see Figure 2.1) could only be formulated with confidence as a result of this petrological input.

2.3 Joint Mapping - The Importance of Continuity

Books, and hundreds of papers, have been written on the subject of mapping and processing of joint (defect) data (Priest, 1993a). Good review articles are presented by Priest and by Einstein in Volume 3 of Comprehensive Rock Engineering (1993).

Most attention in publications and in projects is directed to mapping and analysing the <u>orientations</u> of defects. Priest (1993b) supports Piteau's contention that the most important consideration is "discontinuity orientation with respect to the engineering structure". It is the writer's experience that this is not true and that defect continuity (or persistence) is as important as, or more important than orientation (this assumes of course that defects are sufficiently frequent to be of concern).

The great emphasis placed on defect orientation is substantially due to the fact that we can measure strike and dip with ease and the resulting data are readily amenable to numerical and graphical manipulation. It is difficult to measure defect continuity and even more difficult to present and manipulate the data (Einstein 1993). Hence, there has been a natural, but unfortunate, tendency to concentrate on orientation data.

It is worth noting that in the Rock Mass Rating (RMR) classification system (Bieniawski 1993) defect properties are given the following proportional levels of importance for underground structures:

• spacing	43%
• continuity, roughness and infill	41%
• orientation	16%

In my opinion this is a reasonable weighting, although continuity possibly deserves greater emphasis.

Information on continuity must be obtained from surface mapping and from an understanding of the genesis of the rock mass of interest. Modern line mapping techniques include recording trace length and the nature of discontinuity termination (Priest 1993b). An example of the output from such an exercise is given in Figure 2.2. However, line mapping is limited when it comes to assessment of continuity and therefore areal mapping should be adopted where possible. A good example of areal mapping is that undertaken by MacGregor (1980) which is summarised in Section 4.2 of Part 3 of this article.

2.4 <u>Groundwater Regime and Rock Mass Permeability</u>

I have been fortunate in not encountering major groundwater flows in underground works. The horror stories have happened to colleagues.

These problems are often associated with localised features which would only be found by chance in a site investigation. Massive inflow, leading to project shutdown, occurred in the 80 km long Orange Fish Tunnel in South Africa from a few open joints which were connected to a major groundwater compartment. A recent local example in the Blue Mountains is described by van Putten & McQueen (1993) where they record:

> *"A major inflow occurred in the Hazelbrook Carrier 3 tunnel at Ch7860 metres in the Woodford area beneath a valley where the minimum cover was 15 metres. The water was mildly acidic and was flowing through a single*

bedding parting over a 30 metre section of the tunnel at an estimated 20 to 30 litres/second, and showed little sign of decreasing. The TBM excavation was interrupted for five days as grouting was attempted from the surface. Cement grouting was also attempted from within the tunnel after the TBM passed through the area, however, neither method was successful. The inflow was stemmed using a latex emulsion grout known as SCEM 66. Final sealing of the area was achieved by in situ *concrete lining with water stops between construction joints and grouting behind the lining."*

Because such features cannot be located with confidence prior to tunnelling it is essential that probe drilling be performed ahead of the face wherever there is a risk of a large body of water being intersected by open defects.

In my opinion, simple packer testing (Burgess 1983) is an essential part of a site investigation and comprehensive data in this regard should be available in a proper site investigation report. The results can be quickly evaluated in terms of the classification system given in Table 2.1.

Table 2.1
Classification of Lugeon Test Results

Lugeon Value litre/m/min at 100 kPa	Permeability	Grouting (cement)
<1	low	ineffective
1 to 5	moderate	largely ineffective
5 to 20	high	effective
20 to 50	very high	effective
>50	extremely high	moderately effective

2.5 Natural Stress Field

Rational design of underground structures demands a knowledge of the natural stress field. Good methods exist for measuring stresses and fortunately there is now a reasonable database of stress measurements in many parts of the world. Volume 3 of Comprehensive Rock Engineering has a series of excellent papers on this topic. Some facets of the importance of the natural stress field have been discussed in Part 1.

From the writer's experience and review of case studies it appears that over-coring techniques (CSIR triaxial or CSIRO HI cell) give the best results. However, aside from porous rocks near the surface, hydrofracturing gives good data. Borehole slotter testing seems to work well in fine grained strong rocks but my limited experience has shown this technique to underestimate the stresses in porous materials like sandstone.

2.6 Location in Space of Important Structural Features

Major individual faults, dykes and zones of alteration can have a substantial impact on the construction of a tunnel or cavern. The possible presence of such features is usually known from regional mapping and it is worth going to considerable expense to determine the locations in space of features deemed critical to the project. Geophysical techniques can help but boreholes are usually necessary for accurate delineation of structure and properties.

Figure 2.3 shows the overlapping inclined boreholes which were drilled in the northern interface area of the Sydney Harbour Tunnel. This interface between the immersed tube units and the land tunnels included the main ventilation building which is a 5 storey underground building, free standing in a hole excavated on the harbour's edge. Interpolation of dyke outcrops west and east of the tunnel route suggested a 1m to 2m wide weathered dyke could have been located in the interface area - hence, the inclined boreholes. This feature could have had a major impact on design and construction. The overlapping boreholes indicated the dyke did not pass through the interface area. It was not encountered during construction and must be within the harbour to the south.

2.7 Relative Stiffness of Rock Mass Units

The design of underground openings is usually based on either empirical methods (e.g. rock mass classification) or methods built around elastic stress analyses. In neither case is it necessary to measure the absolute values of modulus of the different structural regions in a rock mass. Such measurements are difficult and expensive (e.g. large scale flat jack, plate loading or pressure chamber tests) and are only warranted where displacements are of critical importance (usually to other nearby facilities).

What is important is to develop a good understanding of the anisotropy (or inhomogeneity) of both stiffness and strength - in relative, not absolute terms. Such anisotropy/inhomogeneity has major impact on stress distributions around tunnels and caverns, which in turn has important impact on the design of the opening shape (see Part 1 of this article).

From my experience there are three ways of assessing relative stiffness:

(i) Use the RMR classification system (I do not believe the absolute values which are so obtained, but relative values can be reasonable).

(ii) Use pressuremeter testing.

iii) Use convergence or extensometer monitoring in any exploration adits or shafts (see Figure 2.4 taken from Pells, McMahon & Redman, 1981).

2.8 Unconfined Compressive Strength

The unconfined compressive strength (UCS) test is by far the most common laboratory test undertaken for rock mechanics studies, that is assuming that one accepts the point load index test as a field test. Applications include:

(i) estimation of the onset of compression or shear failure around underground openings;
(ii) estimation of the strength of rock pillars in underground mining;
(iii) estimating triaxial strength via empirical failure criteria;
(iv) estimating rock modulus for calculation of displacements and settlements;
(v) assessing excavation characteristics whether by ripping, by drag picks on roadheaders, by disc cutters on TBMs or by percussion drills;
(vi) assessing blasting requirements.

Direct measurement of UCS values is difficult and is subject to errors which come from two sources, namely:

(i) bias in sample selection, and
(ii) errors resulting from inappropriate sample preparation, test apparatus or test procedure.

The substantial variability which is usually found in rocks in engineering projects means that critical appraisal must be made of errors which may occur in testing but which may have an effect substantially less than the inherent variability. This does not mean that a casual attitude to laboratory testing should be condoned, but it does mean that there is little point in spending time and money in chasing a 1% error in the laboratory tests when there is a 40% variability in the results due to natural variability, sample selection bias etc.

While research studies of rock properties can be undertaken on near identical specimens taken from single blocks of near homogeneous rock, the core from rock engineering projects is not uniform. Frequently, several different rock types are encountered and usually weathering and strength changes occur along boreholes. Superimposed on this natural rock material variability are the presence of defects (fissures, healed joints, veins) and bedding or schistosity. These defects mean that zones of poorer quality rock mass yield short lengths of core. Furthermore, fractures induced by drilling and core boxing are more likely to occur in zones of weaker, jointed and weathered rock. But uniaxial testing requires core lengths of at least 120mm for N-sized core and 180mm for H-size. Therefore, when samples are selected for testing it is inevitable that they are biased to the better quality rock and it is very difficult to generate a statistically true test population. To deal with this problem it is suggested that the following procedure be followed:

(i) Think carefully about the purpose or purposes of the tests. If they are to provide the basis for assessing, say, road header cutter performance then there may be merit in the data being biased to the high strength zones of the rock mass. It is these that may control the performance of the machine and, more importantly, may lead to cost overruns or contractual claims if not identified. If, however, the test data are to be used for assessing whether compression/shear failure will occur around a tunnel then a bias to the high strength may be very misleading.

(ii) If the normal bias to high strength is not appropriate then it is important that a large number of Brazilian tensile tests or point load index tests be conducted on the short lengths of core representing the poorer rock. There are generally good correlations between tensile strength or point load strength and UCS and these can be used to remove the bias which arises from using only uniaxial compressive strength results.

(iii) If it is critical to obtain strength data on weak/weathered or fractured zones further drilling may require:

(a) increase in core size, say to PQ (85mm) or larger;
(b) use of plastic core barrel liners (splits) which are sliced up at the time when test specimens are cut; and
(c) use of special sampling core barrels.

Pells (1993) lists the consequences of common laboratory "errors" on measured strength and stiffness. The important point to note is that in most cases the effect is to lower the measured strength and stiffness. This is important to remember because a lower strength assessment may be conservative when the results are being used to assess structural capacity of a rock mass (e.g. tunnel stability, allowable loadings on foundations), but is not conservative when the results are being used for assessing rock excavatibility characteristics (e.g. drag pick cuttability, blasting).

2.9 Slake Durability

The durability of certain rocks when exposed to the atmosphere may have important consequences with regard to tunnel construction and the design of tunnel support. Rocks which degrade or swell on exposure to the atmosphere will generate swelling pressures on tunnel linings or will be subject to degradation in the tunnel invert. These can have important economic consequences and therefore it is important to identify such swelling/slaking materials during a site investigation.

The slake durability test which was proposed by Franklin & Chandra in 1972 has long been touted as an appropriate test for assessing this important parameter of a particular rock. However, it is my experience that the slake durability test is of limited value because it is complex and time consuming.

A simple slaking test, involving placing a fragment of rock in water and noting the rate of dispersion and breakdown, is sufficient to categorise rocks which are a problem or which are benign. A brief description of the test method is given below.

1. Place a 100mm thick disc of core, or a single fragment, in a glass/perspex 600 or 1000ml container which can be sealed.

2. Fill the container to a height of 50mm with distilled water in order to provide 20mm sample cover. The use of distilled water serves as a standard. Additional testing using groundwater, surface water or a solution of known salinity can also be utilised.

3. Visual assessments of the specimen condition must be made at elapsed times of 1 minute, 1 hour, 5 hours and 24 hours. The specimen condition is reported as a letter/number code in accordance with the following table. Testing containers should not be disturbed during the 24 hour period. No mechanical abrasion is involved.

Slaking		**Swelling**		**Dispersion**	
Code	**Description**	**Code**	**Description**	**Code**	**Description**
A	No deterioration	**1**	No Swelling	**1**	Water clear
B	Slight deterioration, minor flecks and fragments	**2**	Slight swelling, volume increase 0 to 20%	**2**	Water slightly muddy
C	Medium deterioration, shape largely the same, surface deteriorated	**3**	Medium to high swelling, volume increase 20 to 100%	**3**	Water completely muddy
D	High deterioration, shape largely or completely destroyed, many fragments		Very high swelling, volume increase >100%		
E	Total disintegration	**4**			

Example
A mudstone exhibiting slight deterioration, high swelling and with clear water is defined as **B31** potential.

2.10 Cuttability Characteristics

It has become typical practice within Australia over the last decade or so to undertake numerous drillability and abrasivity tests on rock samples for tunnelling projects. These tests either fall into the category of specific energy and cutter wear tests undertaken at the University of New South Wales, or a suite of tests undertaken at Melbourne University, including Goodrich drillability number, Goodrich wear number, Cerchar abrasivity, Shore hardness, Scleroscope hardness.

Without wishing to be unduly negative, the facts of the matter are that these various tests have had limited value in predicting the productivity and cutter wear of tunnelling machines. In many cases one simply does not know how to use a particular test result to predict machine performance, but because everybody does the tests one tends to join the queue. As stated by Baxter (1993) *It would appear that predictability of performance from cuttability tests is yet to be achieved.*

However, a recent publication by Verhoef (1993) gives some cause for optimism inasmuch as careful analysis of road header and dredging performance in the Hawkesbury Sandstone has indicated that a reasonable correlation between laboratory tests and actual performance is obtained using the *F*-value defined by Schimazek. This *F*-value is given by the equation:

$$F = \frac{Q \times \phi \times \sigma t}{100}$$

where Q = equivalent quartz volume percentage
ϕ = grain size
σt = Brazilian tensile strength

The fact that Verhoef has shown good correlation between the *F*-value and actual cutter wear performance accords with my experience that the quartz content and rock strength (whether defined by Brazilian tensile or UCS) are the critical factors with regard to road header performance.

3. FORMULATION AND PRESENTATION OF THE MODEL

It is important that the engineer/engineering geologist works through the available geological data using the following steps:

(i) Draw a plan showing all sources of data (boreholes, exposure mapping, geophysics traverses).
(ii) Select appropriate section orientations and draw summary borehole logs, geophysics traverses and summaries of exposure mapping on those sections.
(iii) Using tracing paper overlays, prepare alternative interpolations of the major structural features.
(iv) Divide the ground mass into zones of similar engineering and structural properties.
(v) Plan further investigations to clarify important uncertainties.
(vi) Give the whole lot to a top quality engineering geologist for review and modification.

However, in considering the above process, it is important that a clear distinction be made between long tunnels, typically with widely spaced borehole data, and local structures like hydro-power caverns or deep basements where data information may be dense.

All our deterministic training as engineers makes us want to prepare geological structural models for long tunnels which show the correct <u>locations</u> of structural features such as joints, faults, dykes, bedding planes, zones of alteration etc. However, the fact remains that unless boreholes are closely spaced (30m or less, depending on geological complexity), or there is good exposure mapping, predicting the locations of particular faults, dykes, open water bearing joints etc. is usually fraught with error. Typically, in a long tunnel we can successfully determine that certain types of features are likely to be encountered, but usually we get the locations wrong and this can lead to significant contractual claims.

The concept has thus been developed (Pells & Best, 1991) of a geotechnical model which does not attempt to show the location of structural features but rather is expressed in terms of *Typical* conditions, *Adverse* conditions and *Special* conditions in each geological unit. These are defined as follows:

Typical Conditions
These represent average rock mass behaviour which may be expected along more than 50% of the length of tunnel in a particular unit. Over portions of this length the behaviour may be somewhat better than *Typical* and over other portions somewhat worse, but the differences should be conservative from the costing viewpoint. The percentage of a unit with these *Typical* conditions must be assessed on the available knowledge (may be, say, 70% of the unit).

Adverse Conditions
These represent significantly poorer rock mass conditions within a particular unit. Such poorer conditions may, for example, be associated with closer joint spacing or a greater degree of weathering than in the *Typical* conditions. Again, the percentage of a unit with these conditions must be assessed.

Special Conditions
There may be short lengths within a unit where the conditions are quite different from the remainder of the unit, typically due to the presence of dykes or major faults.

In applying the above concept it is necessary for the geotechnical engineer/ engineering geologist to do two things, namely:

(i) assess in quantitative terms the properties of the *Typical, Adverse* and *Special* conditions in each geotechnical unit, and
(ii) assess the percentage of tunnel in each category.

As an example of this approach, Table 2.2 gives the tabulated data available at the time of tender for the Bulgo Sandstone unit which was encountered in the outfall tunnels at North Head and Malabar. From these data and inspections of exposures of the Bulgo, the parameters for *Typical* and *Adverse* conditions were assessed. These are expressed in Table 2.3 in terms of the classification parameters for the SCIR and NGI classification schemes.

Table 2.2
Rock Mass Data - Bulgo Sandstone

GEOLOGICAL DESCRIPTION

Massive to laminated, white to grey-green and brown, fine to very coarse grained sandstones with siltstone laminations and bands and occasional claystone layers. The sandstones may be current bedded with dips to 30°. The siltstone and claystone beds generally vary from 0.1 to 2m but there are claystone members to 12m.

MAJOR GEOLOGICAL FEATURES

Crush zones and minor faulting dipping from 25° to 70° may be anticipated at a vertical spacing of the order of 20m. Also occasional bedding plane crush zones.

DEGREE OF WEATHERING (AS1726-1981)

Boreholes NO1, NO3, MO3, NO4B

Fresh	Slightly W	Moderately W	Completely W	Seams & Crush Zones
100%				10.14m in 312m of core

ROCK SUBSTANCE STRENGTH (AS1726-1981)

<2 MPa	2 to 6 MPa	6 to 20 MPa	20 to 60 MPa	>60 MPa
	lowest value 18.8 MPa	Lowest value 18.8 MPa	Offshore: χ = 33 MPa s = 15.3 Onshore: χ = 39 MPa s = 20.4 Gordeaux Tunnel χ = 55 MPa	Highest value 75.6 MPa

ROCK MASS PERMEABILITY DATA

µL	Length of Borehole	
0.01	11m	Geometric mean = 0.6µL
0.05	205m	
2	20m	
17 to 57	15m	

HYDROSTATIC HEAD	REGIONAL STRESS FIELD
	Assume $\sigma_h NS = 2.5\sigma_v$ $\sigma_h \omega E = 1.5\sigma_v$

JOINT SETS

Source of Information: References 10, 11, 14

Set 1	Set 2	Set 3	Set 4
Bedding	Tension	Tension	

ORIENTATION OF JOINT SETS (True North)

	Set 1	Set 2	Set 3	Set 4
Dip	0 to 5°	90° ± 10°	90°± 10°	Minor trend at 140°
Strike	0°	100° ± 10°	10° ± 10°	

EFFECTIVE JOINT SPACING (m)

References 4, 10

Set 1	Set 2	Set 3	Set 4
0.5 to 5 in sandstones	2 to 15m Average ≈4m	2 to 15m Average ≈4m	

CONDITION OF JOINTS

Set 1	Set 2	Set 3	Set 4
Planar, clean, occasional crush zones, infrequent seams (spacing >20m)	Planar, clean, fresh wall rock. Continuity >>2m but some greater	Planar, clean, fresh wall rock. Continuity >>2m but some greater	

RQD

Overall, of 360m cored	78%	with RQD >90%
	18%	with RQD >75% <90%
	3.5%	with RQD >50% <75%
	0.5%	with RQD >25% <50%

Table 2.3
Classification Parameters of Typical and Adverse Conditions for the Bulgo Unit

Classification System	Item	Typical	Adverse
RMR	Strength	4	2
	RQD	18	13
	Joint Spacing	27	23
	Joint Condition	20	12
	Groundwater	4	4
	Joint Orientation	-7	-7
	Rock Mass Rating	66	47
	Rock Class	**III**	**IV**
NGI	RQD	90	50
	Jn	9	12
	Jr	1.5	1.5
	Ja	0.75	3.0
	Jw	0.5	0.5
	SRF North Head	1.2	7
	Malabar	1.2	7
	Q	**8.3**	**0.15**

Table 2.4 shows the assessment made of the percentage of *Typical, Adverse* and *Special* conditions in each unit for each of the three Sydney ocean outfall tunnels.

Table 2.4
Percentages of Conditions in Each Unit for the Sydney Ocean Outfall Tunnels

Geotechnical Unit	Percentage of Tunnel with Given Rock Mass Conditions		
	North Head	**Bondi**	**Malabar**
Hawkesbury			
Typical	88	95	84
Adverse	10	5	12
Special	2	0	4
Newport			
Typical	88	87	88
Adverse	8	8	8
Special	4	5	4
Bald Hill			
Typical	73	73	73
Adverse	25	25	25
Special	2	2	2
Bulgo			
Typical	76	N/A	76
Adverse	20	N/A	20
Special	4	N/A	4

For localised structures such as an underground chamber, or for tunnel lengths where there is closely spaced data, it is feasible and important to develop plans and sections which show as closely as possible the actual locations of key structural features. An example of one such model is given in Figure 2.5. This was for the Sydney Opera House underground parking station where there was a wealth of borehole data and extensive mapping from construction of the nearby Sydney Harbour tunnel and from surface exposures.

4. VERIFICATION DURING CONSTRUCTION

It is very important that during the design stage thought be given to how the design assumptions will be verified during construction. Tunnel design is fraught with uncertainty and therefore a risk assessment must be completed during design to evaluate the uncertainties and to decide how these uncertainties are to be dealt with during construction. Normally this involves detailed geological mapping and monitoring. This facet of construction verification is fundamental to the so-called "New Austrian Tunnelling Method" and has been dealt with extensively in the literature. A good example of construction verification is the Sydney Opera House Underground Parking Station which has already been discussed in Part 1 of this article and is dealt with in detail by Pells Mikula & Parker (1993).

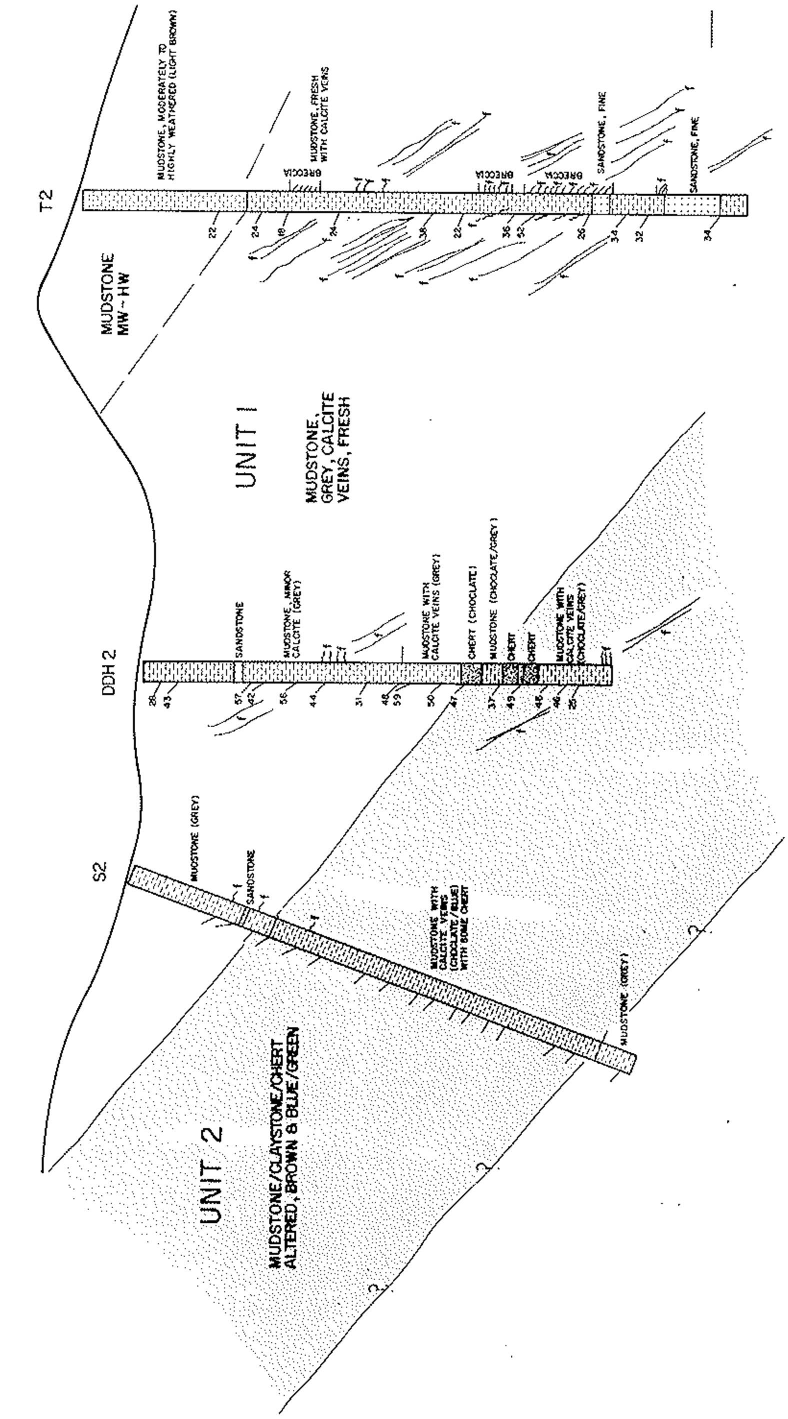


FIGURE 2.1 : SECTION THROUGH ENGINEERING GEOLOGICAL MODEL
BURNS PEAK TUNNEL - PORT MORESBY

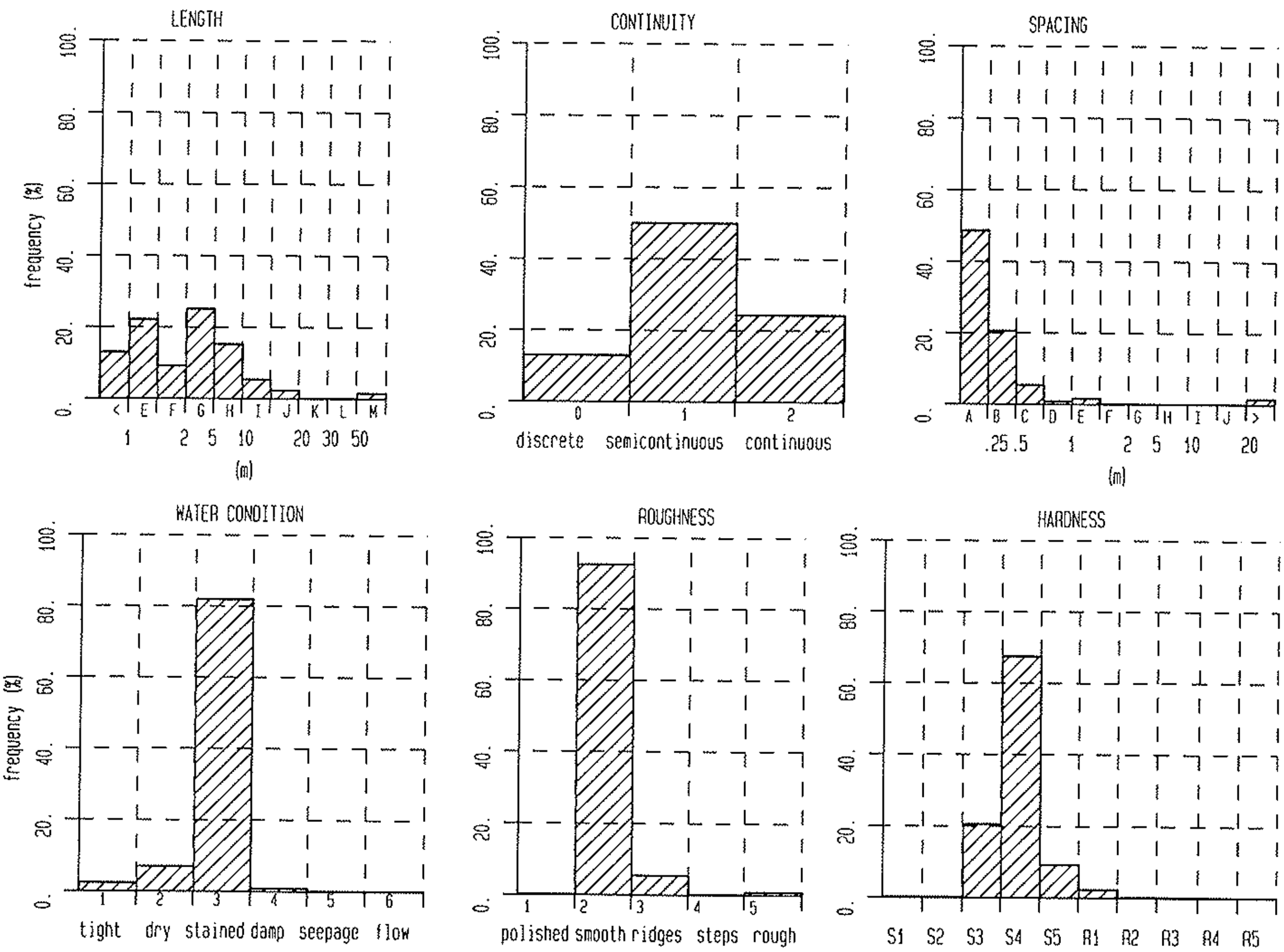


FIGURE 2: LINE MAPPING OUTPUT - CANNINGTON DECLINE NORTH QUEENSLAND (from Coffey Partners Int., 1993)

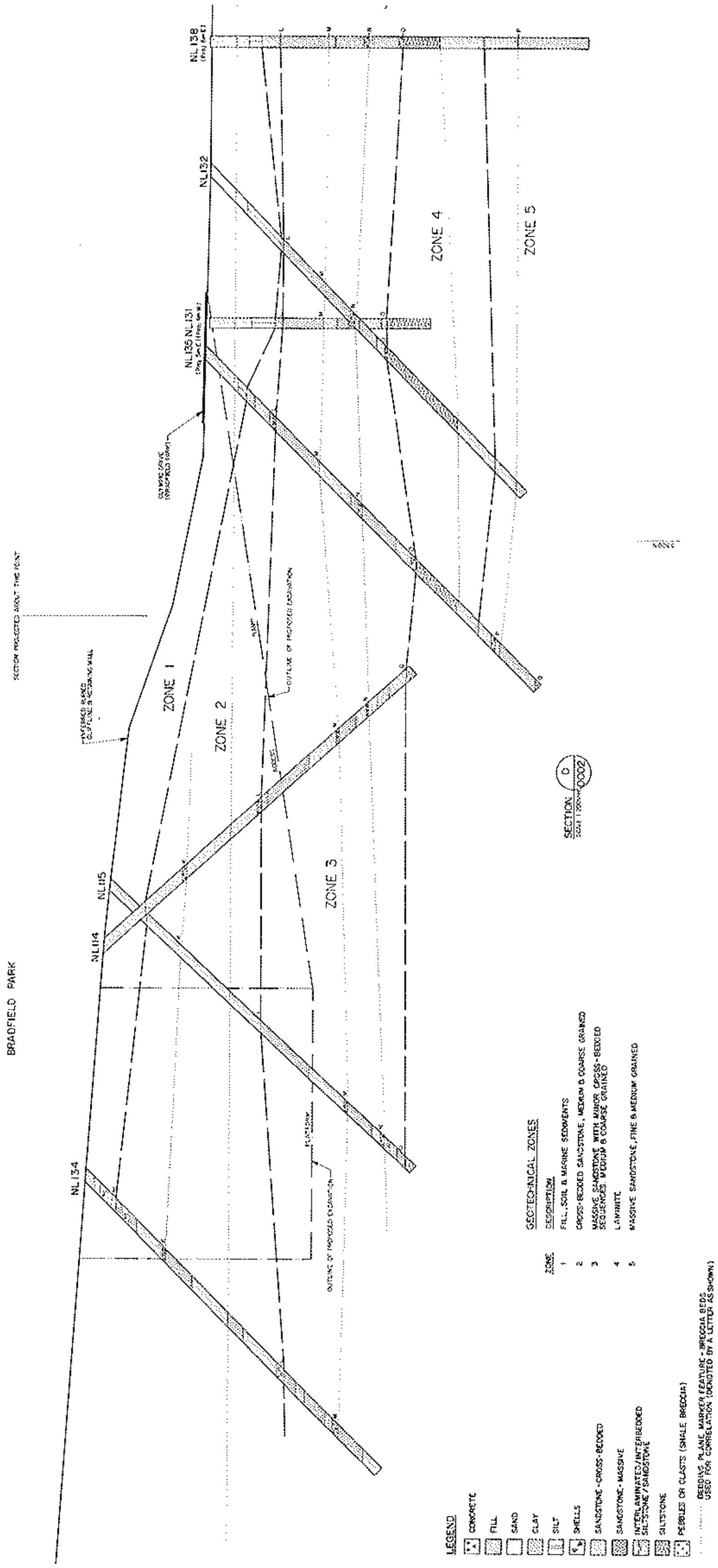
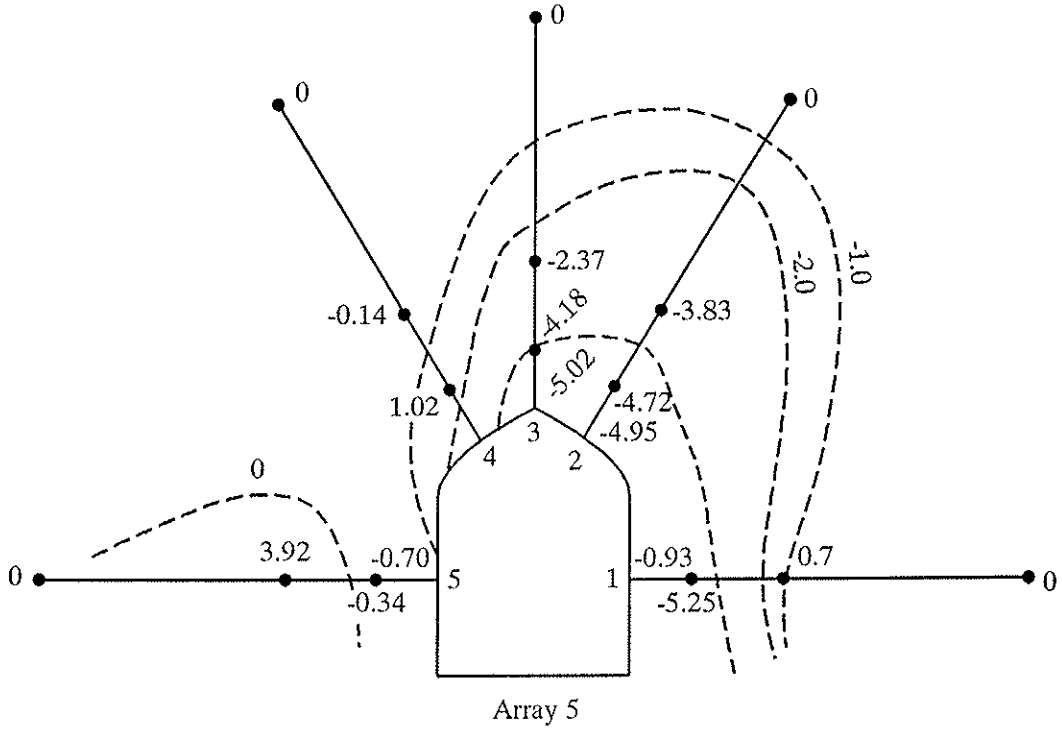


FIGURE 3 : OVERLAPPING BOREHOLES FOR DYKE INVESTIGATION SYDNEY HARBOUR TUNNEL - NORTH SIDE



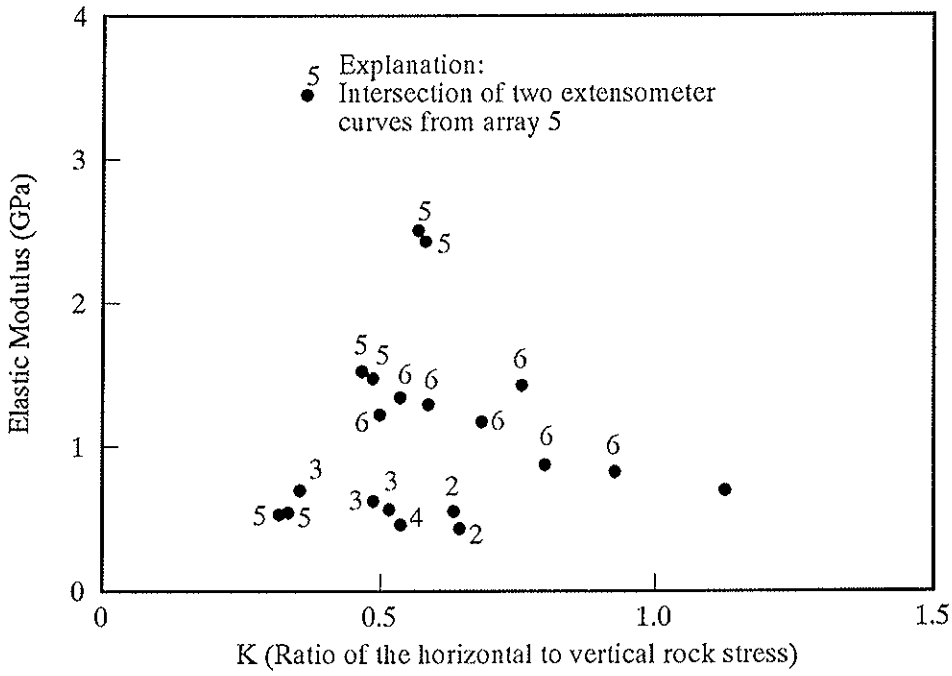


FIG. 4 INTERPRETATION OF INSITU MODULUS AND NATURAL STRESS FIELD FROM INSTRUMENTATION OF TEST ADIT AT THOMPSON DAM, VIC.

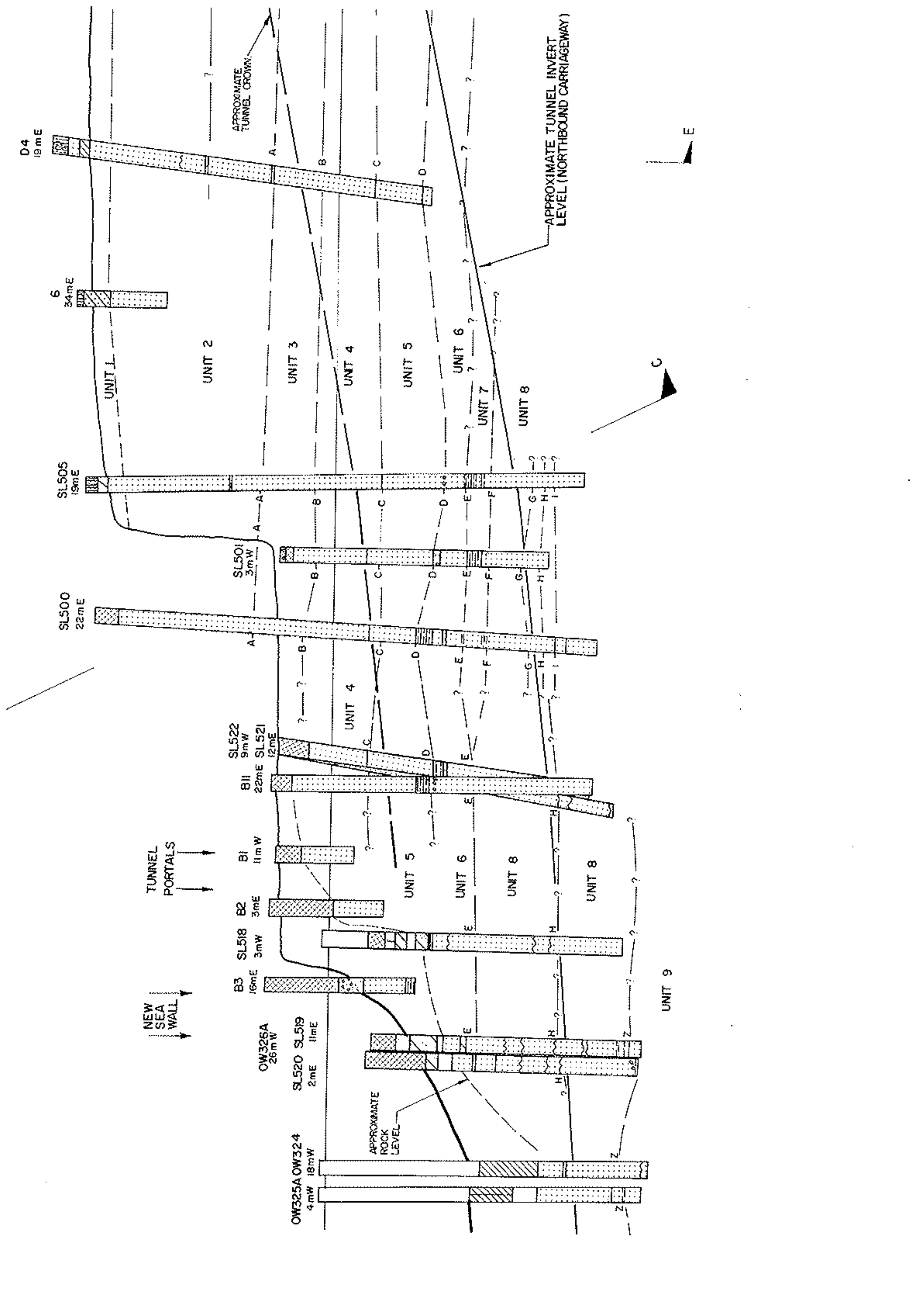


FIGURE 5 : GEOLOGICAL MODEL - SYDNEY HARBOUR TUNNEL SOUTH SIDE