PUTTING THE GEO INTO GEOTECHNICAL - THE ROLE OF THE ENGINEERING GEOLOGIST DURING CONSTRUCTION OF THE EASTERN DISTRIBUTOR TUNNEL, SYDNEY, AUSTRALIA

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SUMMARY

The 1.5km long Eastern Distributor Tunnel, to the east of Sydney's CBD, is Australia's first piggyback tunnel. The tunnel is excavated through a highly urbanised area with limited rock outcrop, hence the geotechnical model formulated for design purposes was heavily reliant on borehole information. During construction, detailed engineering geological mapping served to 'proof check' the assumptions and interpretation made in the design model.

This paper briefly summarises the design geotechnical model, the role of the Engineering Geologist during construction and the Geologist's position in the overall context of the construction team. The post construction model, formed on the basis of in-tunnel mapping, is then presented.

It is concluded that the mapping undertaken during construction largely confirmed the preconstruction model. Where encountered tunnelling conditions were different to those anticipated in the design model, the results of engineering geological mapping allowed appropriate support redesign in 'real time'.

The paper therefore provides a working example of the concept of 'putting the "geo" into "geotechnical'.

1 INTRODUCTION

The 1.5km long Eastern Distributor Tunnel, located to the immediate East of the CBD of Sydney (Figure 1), will carry three lanes of Northbound traffic over three lanes of Southbound traffic in Australia's first piggyback tunnel.

The geotechnical model for design purposes was based on field investigations comprising a sum total of 103 cored boreholes, supplemented by geotechnical mapping in old quarries and along the Cahill Expressway, to the north of the tunnel. Hence the preconstruction or design geotechnical model was heavily reliant on interpretation between boreholes. Engineering geological mapping undertaken during the construction phase of the project was used to validate the preconstruction model.

This paper outlines the design geotechnical model, tunnel construction sequence and the role of the Engineering Geologist during construction. Elements of the post construction geotechnical model are then compared to the design model. It is concluded that mapping and database upkeep during excavation allowed for effective and quick support redesign where needed, served to largely confirm the design geotechnical model and has provided a permanent record of excavation conditions for the future.

2 PRECONSTRUCTION (DESIGN) MODEL

2.1 LITHOLOGY

The tunnel excavation was expected to occur entirely within subhorizontally bedded Hawkesbury Sandstone which forms the basement rock for the majority of the CBD. However, the upper part of the main south portal at South Dowling St and the ramp tunnels to Anzac Parade and Moore Park Rd were anticipated to be excavated through weathered Mittagong Formation. The boundary between the Hawkesbury Sandstone and the Mittagong Formation was known to be gradational and it was considered that the base of the Mittagong Formation formed a shallow basin structure near Taylor Square (Figure 1).

Three main sedimentary facies are apparent within the Hawkesbury Sandstone as outlined below.

1 Massive Facies: typically internally homogenous in particle size and either massive or displaying a poorly to well developed undulose layering. This facies usually displays a discordant, erosional lower layer and a planar

concordant upper surface, Herbert (1). Shale breccia commonly occurs within troughs above this erosional surface.

2 Sheet Facies: Sandstone in this facies consists of cosets of trough or tabular cross strata which are bounded by subhorizontal bedding surfaces. Cross bedded sets range from a few centimetres to more than 5m in thickness and commonly dip to the north to northeast. Syndepositional convolution and recumbent folding of foresets is common in this facies, Conaghan (2) and is probably due to mass movement events shortly after deposition involving the unconsolidated sediments

3 Mudstone Facies: This facies is laterally discontinuous and usually between 0.3 - 3m thick. It is composed of grey, fissile mudstone which in places is slightly carbonaceous and is often laminated with fine grained sandstone. The facies is often referred to by the terms "Shale lens" or "Laminite".

2.2 BEDDING

The results of the site investigation indicated that bedding was typically subhorizontal and typically planer to undulose, with high horizontal continuities. Sand to Clay infill was encountered along approximately one third of the measured bedding defects.

2.3 CROSS BEDDING

Cross bedding was inferred to typically dip towards the northeast. In fresh or slightly weathered sandstone at the depths of the tunnel, cross bedding was not anticipated to form planes of weakness. However, in moderately to highly weathered sandstone the cross beds were considered likely to form surfaces of incipient parting or low shear strength.

2.4 JOINTING

Based on oriented hole core and the limited surface mapping data, the dominant joints were inferred to be subvertical and strike north-northeast, although occasional vertical joints orthogonal to the main north-northeast system were interpreted. The joints were anticipated to have substantial horizontal and vertical continuity, and be planar, rough and mostly clean.

2.5 FAULTING

Faulting in the Hawkesbury Sandstone is uncommon. However, a fault zone, termed the Woolloomooloo Fault Zone (WFZ) was encountered while drilling for the piers of the Eastern Suburbs Railway viaduct in the late 1960's. It was expected that the WFZ would intersect the main tunnel between 160m and 240m in from the northern portal, and at the William Street Ramp Tunnel portal. Minor sub-parallel faults or shears were expected to intersect the main tunnel up to approximately 450m in from the northern portal.

The zone was inferred to be steeply dipping to the southeast; vertical displacement across the zone was interpreted to be about 5m, with the eastern (hanging wall) side up-thrown, ie an overall reverse motion.

Stress measurements were undertaken in a borehole just to the north of Stanley Street, using hydrofracture techniques. The results indicated that the horizontal stress was about five times the overburden pressure which was considered to also indicate reverse motion across the WFZ.

Borehole data suggested that additional minor faults or shears could possibly occur near the intersection of Flinders Street and South Dowling Street, ie approximately 200m in from the southern portal of the main tunnel.

Low angle thrust faults, which merge into bedding plane shears, are known to occur within the Hawkesbury Sandstone but were difficult to detect in borehole core. A thrust fault was observed in an exposure to the north of the tunnel. It was considered probable that thrust faults would be encountered along the tunnel route.

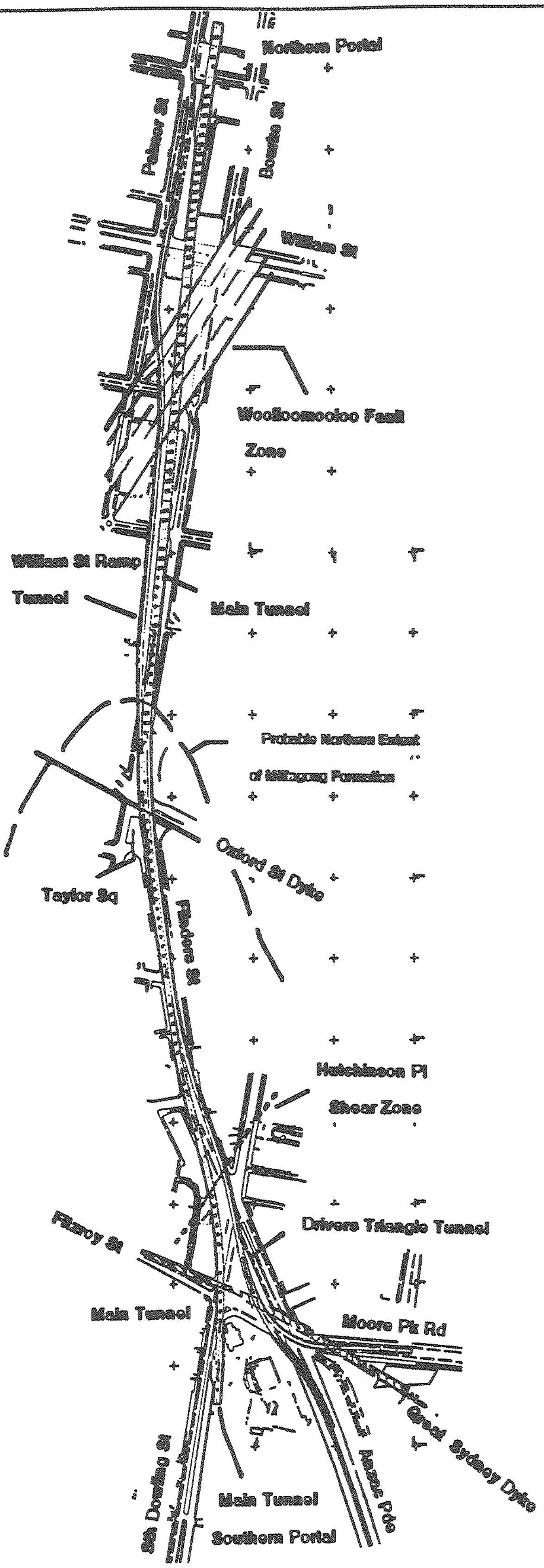

Figure 1 Tunnel Location and Encountered Geology

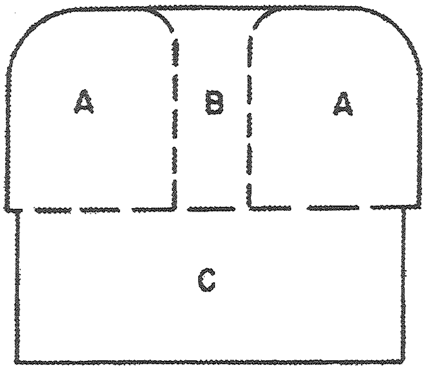

Figure 2 Tunnel excavation sequence. (See text for explanation.)

2.6 IGNEOUS DYKES

Two near-vertical igneous dykes were interpreted to intersect the tunnel route (Figure 1):

- a relatively thin (0.3 to 0.6m wide) dyke underneath Oxford Street (the Oxford St Dyke), and
- the Great Sydney Dyke which runs along the alignment of Fitzroy Street. The dyke varies in thickness from 5 to 7m and, at the depth where it is penetrated by the tunnel, was inferred to comprise stiff fissured clay (extremely weathered dolerite).

3 TUNNEL EXCAVATION

3.1 EXCAVATION SEQUENCE

Excavation was mostly by roadheader. Excavation of the main tunnel typically involved the following sequence (Figure 2):

1 Excavation of 5 to 6m wide parallel headings at Northbound level
2 Pillar stripping
3 Southbound excavation.

At the peak of excavation activity, six roadheaders were operational with a maximum advance rate of 80m per week for a 6m wide heading attained.

3.2 COMPANY ROLE AND PERSONNEL OBJECTIVES DURING CONSTRUCTION

Pells Sullivan Meynink Pty Ltd (PSM) had the responsibility for the design of roof and side wall support of the Eastern Distributor Tunnel. The basic philosophy for tunnel support involved the concept of a linear arch being formed in the rock above tunnel roof which was supported by rockbolts and shotcrete.

PSM's role on site was involved with the on-site "Surveillance" team, whose job it was to ensure that the support for the tunnel was installed as designed. Where ground conditions differed substantially from the design geotechnical model, it was the responsibility of Surveillance to provide support redesign. PSM's task was therefore to check that geological reality reasonably matched the design model and to undertake redesign when and where significant departures from the design model occurred.

Significant departures from the design geotechnical model would have included such scenarios as:

- relatively continuous moderately dipping defects,
- in conjunction with monitoring, evidence of low horizontal stress (eg. open joint planes),
- ground water inflows greater than expected,

- faults and shears other than predicted, and
- weathering conditions other than predicted.

The objectives of the Engineering Geologist's role were therefore to:

- modify and update the geotechnical models used for design purposes,
- forecast tunnel conditions, more particularly in twin headings, and through known structures,
- reduce the risk of 'geological surprises',
- allow quick response and localised redesign of tunnel support where 'geological surprises' were encountered, and
- provide a record of tunnel conditions for the future.

These objectives were achieved by:

- engineering geological tunnel mapping of all Northbound headings as well as Southbound walls,
- observation and interpretation of drilling for rockbolts and pipe canopy tubes to assess geotechnical conditions above the tunnel roof,
- routine sample collection for point load strength testing , and
- creation and management of a database containing the properties of mapped defects (orientation, length, persistence, roughness etc).

The major task was in-tunnel mapping. Detailed 1:100 scale maps were produced for all headings and included both sidewalls and the roof. When access allowed, the active face was logged. An example of a completed mapping sheet is shown in Figure 3.

Mapping of a particular heading typically lagged some 20 to 30m behind the tunnel face because the roadheader and haul trucks severely affected compass readings. Periodic logging of the face itself was necessary, however, as the face allowed a cross-tunnel section of the local geology to be viewed as well as assessment of the latest excavation conditions.

3.3 DATA FLOW

It was attempted where feasible to provide support redesign to the main contractor in what was termed 'real time'; or in other words with a minimum of delay. The theoretical data flow from in-tunnel mapping and monitoring through support redesign to support installation is shown in Figure 4.

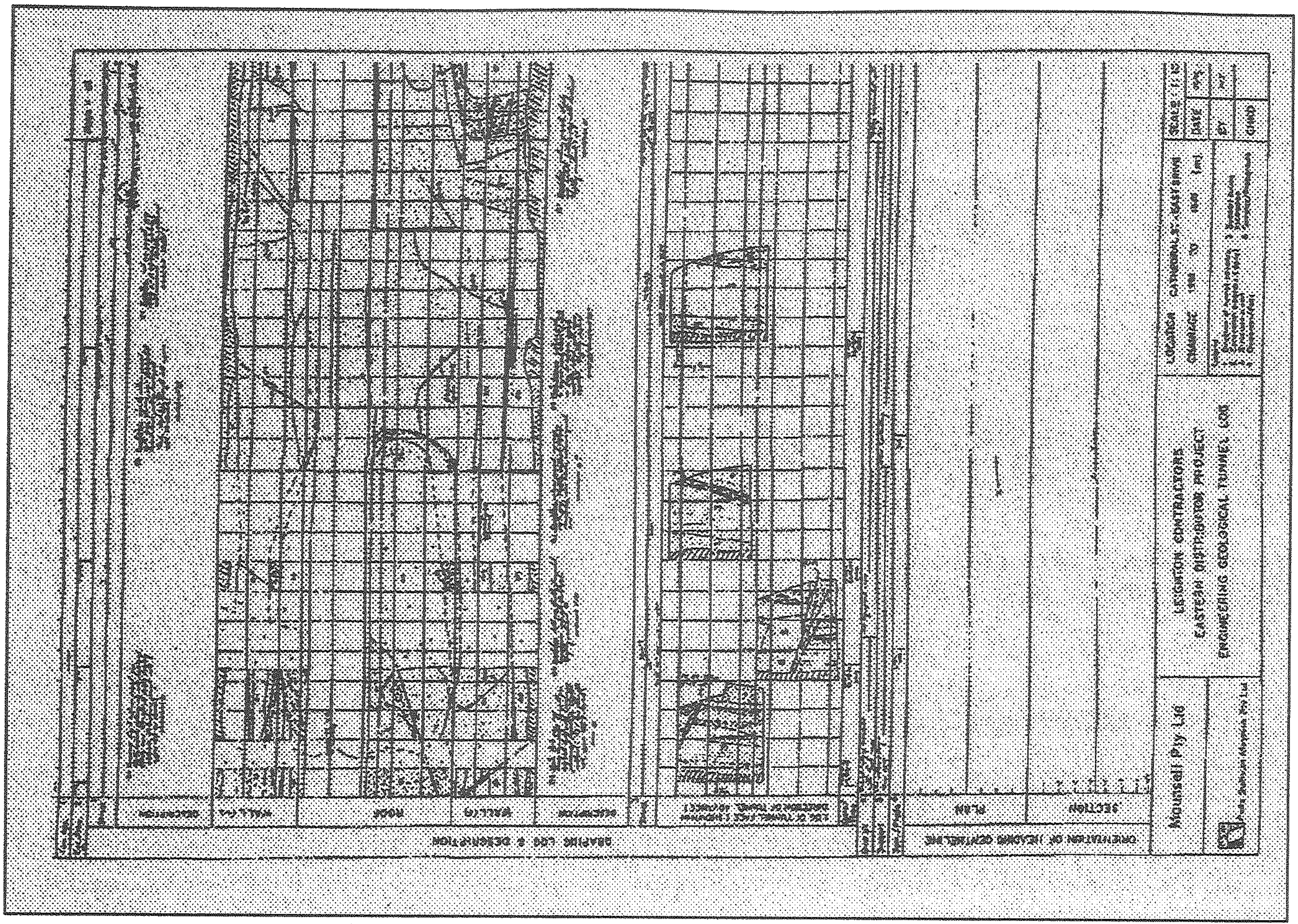

Figure 3 Example of a Completed Tunnel Mapping Sheet

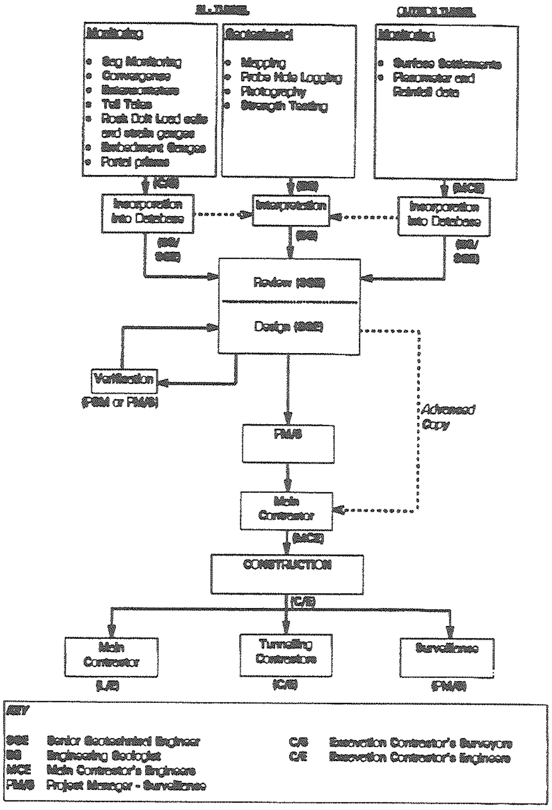

Figure 4 Theoretical Data Flow Sequence

Support review and redesign was carried out by a Senior Geotechnical Engineer, also from PSM, on the results of in-tunnel mapping and monitoring.

In reality, the roles performed by the Engineering Geologist and the Geotechnical Engineer were somewhat overlapping and became more so in the latter stages of the project. The Geotechnical Engineer sometimes performed mapping, whilst the Engineering Geologist routinely undertook support design, particularly sidewall bolting. This had the advantage of 'decompartmentalising' both roles, ie the data flow sequence did not come to a grinding halt because of personnel unavailability etc.

The collection of data from in-tunnel monitoring was the responsibility of the tunnelling excavation contractor. Surveillance had overall control of where monitoring points were installed, and the frequency of reading. It was the responsibility of Surveillance to maintain a monitoring database.

3.4 DATA COLLATION

Data collation involved:

- daily updating of standard plans and sections in paper form,
- incorporation of structural data into a standard database from which defect stereoplots and defect histograms were produced, and
- inclusion of the results of Point Load, or other, testing into a testing database.

The histogram sheet and stereoplots were typically generated for 100 – 200m segments of the tunnel and allowed a quick visual check to be made between the design model and the encountered conditions.

3.5 SOUTHBOUND WALL BOLTING

The Eastern Distributor Tunnel is designed as a piggyback tunnel with Northbound traffic supported on a 600mm thick steel reinforced concrete decking, supported at each end on a 400 to 500mm wide sill beam.

Imposed loads of up to 827kN/m were calculated at the widest span of the tunnel. The orientation of the major joint set at an acute angle to the tunnel alignment and the imposed loads acting on the sill beam meant that potential "one sided" wedge failure, Pells and Dai (3), under the sill beam could occur. This failure mechanism involves release along an existing joint plane with shearing and tensile failure through intact rock (Figure 5).

Depending on the orientation of a joint beneath the sill beam and the local tunnel width, one of a series of four different patterns was installed adjacent to the joint. Support patterns were marked up during southbound wall mapping. This served to minimise the delay between mapping and support installation and allowed a 'one-pass' operation for the Engineering Geologist.

4 ELEMENTS OF POST CONSTRUCTION MODEL

For most of the length of the tunnel, the mapping confirmed the preconstruction model. The following sections provide a summary of the characteristics encountered for various features. Figure 6 summarises the post-construction engineering geological model.

4.1 HAWKESBURY SANDSTONE

Sheet and Massive Facies Sandstone as well as the minor Mudstone Facies were encountered during mapping. In general the Hawkesbury Sandstone encountered was typically classified as Class II with Class III near the portals (after Pells et al, Reference 4). A general profile can be summarised as:

- Mixed Massive and Sheet Facies Sandstone with few shale horizons (encountered in the southern half of the tunnel) overlying;
- Sheet Facies Sandstone with relatively common shale or shale breccia horizons (encountered in the northern half of the tunnel).

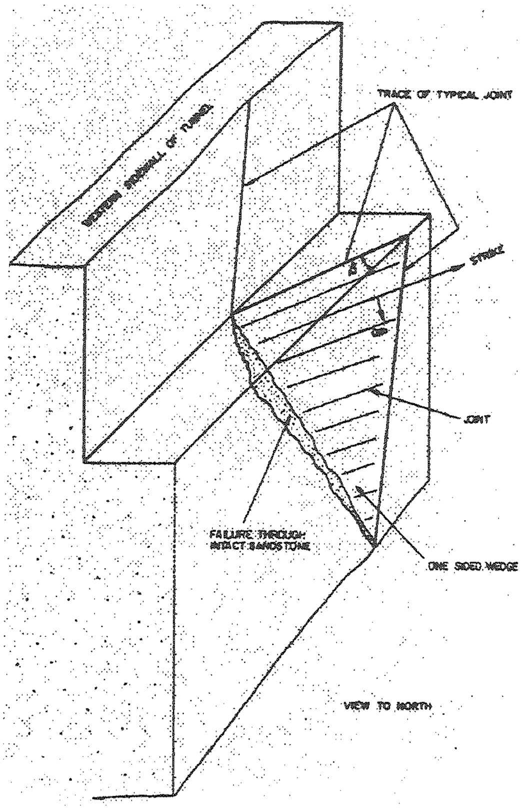

Figure 5 Geometry of One-sided Wedge Failure

The shale breccia was found to comprise zones of reworked and/or syndepositionally deformed shale within a fine to medium grained sandstone matrix. The horizons had a typical maximum thickness in the order of 1m and occurred at greater frequency than anticipated from borehole logging. Typically, the borehole logs indicated the horizons as one or two discontinuous shale lenses in a sandstone matrix, and were mostly noted as being minor features within the drill core. However, exposure within the tunnel indicated these apparently minor features typically formed part of a more extensive zone, with individual continuities of up to 100m.

4.2 BEDDING

Figure 7 presents the stereographic projections of the structure mapped during tunnelling and shows that bedding was mainly sub-horizontal. Analysis of bedding defects recorded during tunnel logging indicate the following defect characteristics:

- bedding spacing averaged about 2.0m;
- 85% dip less than 10°, 95% dip less than 20°;
- 60% had some infill varying from a stain to over 50mm;
    - 5% of which are Clay,
    - 50% of which are sandy Clay with or without some iron oxide,
    - 25% of which are clayey Sand,
    - 10% are Sand, and
    - the remaining 10% either iron oxide or carbonaceous material.

Clay dominated infill typically occurred near the surface whereas sand dominated infill occurred at the depth of the tunnel. This is interpreted to be a weathering related effect.

4.3 CROSS BEDDING

Figure 7 shows that cross bedding typically dips at 15° towards 045°. The mapping also indicated the following characteristics:

- cross bedding spacing was typically less than 0.2m;
- average length of cross bedding is approximately 4m;
- 66% dip between 10 and 20° typically to the northeast; and
- approximately 50% of the measured defects had minor infill of up to 3mm.

During logging, focus was placed on those defects that had some infill. Hence, the proportion of infilled cross beddings calculated above is much higher than reality.

In Class II sandstone or better, the cross bedding surfaces were not noticeably weaker than the rest of the rock material, and as such, did not tend to form planes of separation within the rock mass. However, in zones of Class III, or poorer, sandstone (ie within the WFZ and adjacent to the Great Sydney Dyke), cross bedding planes acted as release surfaces for roof failures. In these sections roof failures were typically defined by 'feather edging' on cross bedding with side release on joints.

4.4 MITTAGONG FORMATION

The Mittagong Formation formed the roof of the Drivers Triangle Ramp tunnel. The sequence comprised highly to extremely weathered interbedded sandstone and black mudstone (Class IV/V), and displayed a gradational relationship with the underlying Hawkesbury Sandstone which was assessed to be Class II/III.

4.5 JOINTS

The major joint set was confirmed to strike north-northeast and the minor set east-southeast (Figure 7). Both defect sets occurred as discrete features or as swarms of about 3 to 10 defects. Defect spacings within the swarm range from 20 to 500mm, whilst spacing between swarms was up to 20m. Typical joint characteristics are outlined below.

- 15% are continuous out of both the floor and roof of the tunnel, crossing several bedding planes, with a vertical continuity greater than 10m.
- 45% are semicontinuous, and transgress one or more bedding planes, with a vertical continuity of between 3 and 10m.

No clear correlation exists between sandstone type and joint occurrence or characteristics. However, joints were much less frequent in the central part of the tunnel compared to near the portals. This may be an effect of near surface stress relief.

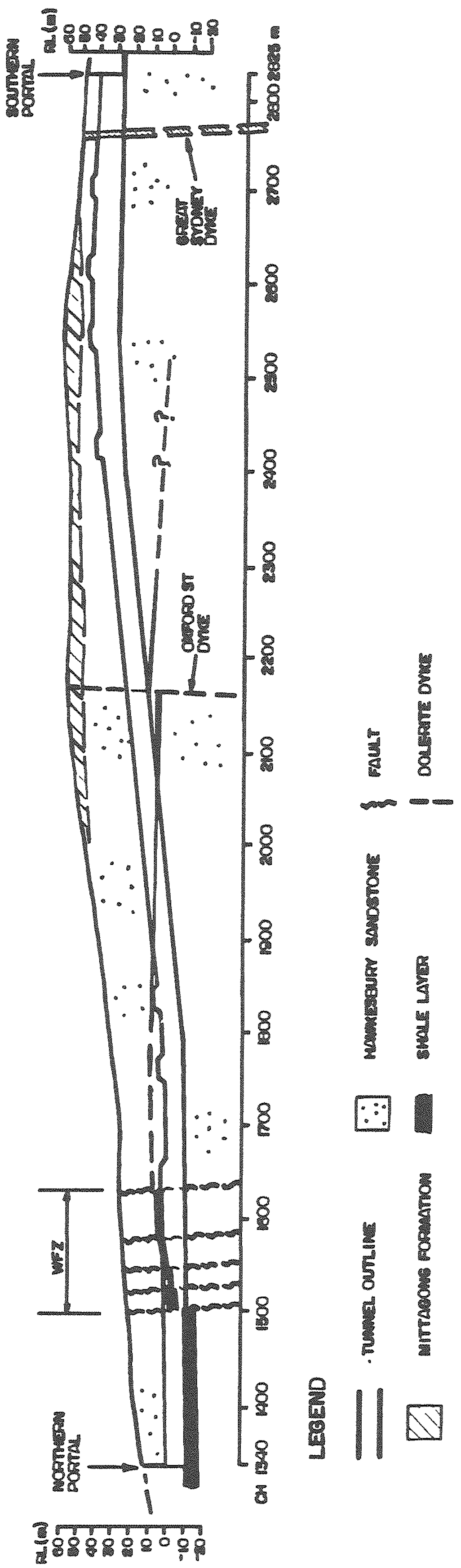

Figure 6 Post construction model

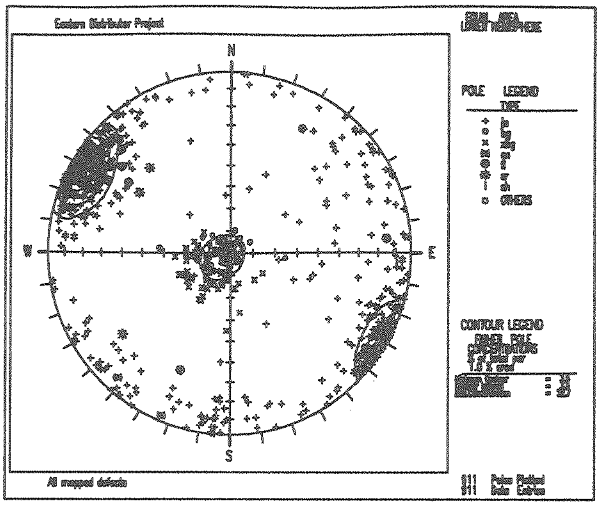

Figure 7 Stereoplot of defect data (relative to true north)

4.6 FAULTS

The WFZ and low angle thrust faults were encountered during tunnelling. Low angle thrust faults were recognised on both sides of the Great Sydney Dyke, and adjacent to, or as part of the WFZ.

The low angle thrusts ranged in length from about 1m up to approximately 20m. A sandy clay infill of up to 50mm was typical. The faults were often formed between adjacent bedding planes. This characteristic is interpreted to mean that:

- some amount of movement occurs along bedding planes, and
- the low angle thrusts serve to provide a movement surface, or 'connection path', between adjacent bedding planes.

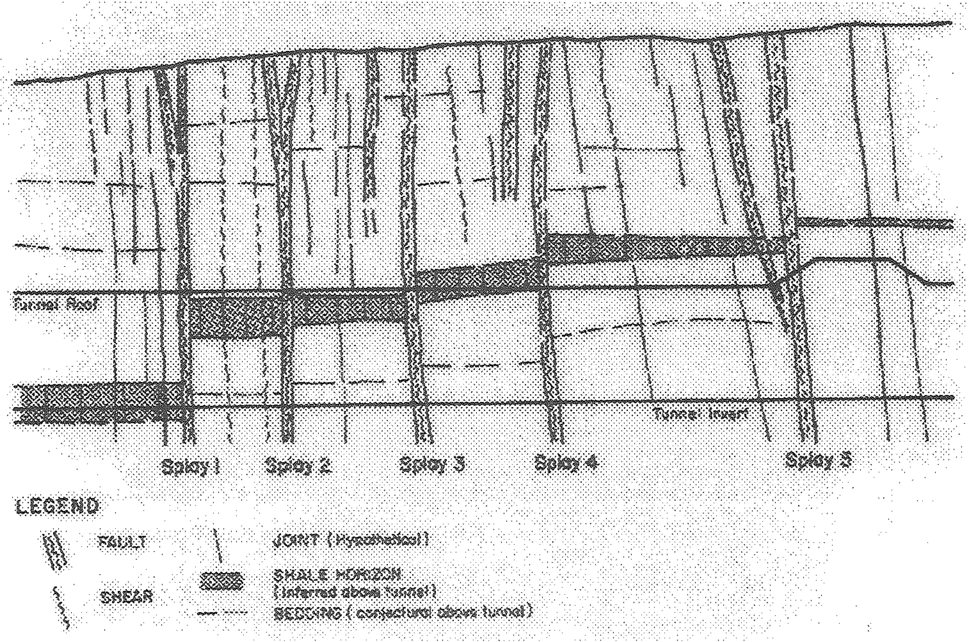

Figure 8 Interpreted Cross Section of the Woolloomooloo Fault Zone

The WFZ was intersected in the main northern tunnel between Ch1521-1625m with parallel faulting extending to 1700m, and at the portal of the William St Ramp tunnel. The WFZ is approximately 50m wide, dips at about 70° towards the southeast, and intersects the tunnel at about 25°. The WFZ comprises five major fault splays, (Figure 8) comprising 50 to 1000mm of crushed sandstone within a soft to firm clay matrix. Smaller faults and shears were apparent between the major splays. These minor faults had a maximum width of about 20mm. Rock quality between the fault splays improves towards the south, starting as poor between splays 1 and 2 and becoming very good between splays 4 and 5.

Overall movement of the zone is interpreted to be reverse (ie northwest side up relative to the southeast), although some of the fault splays are normal. Mapping indicated a displacement across the fault of approximately 8m. The component (if any) of strike slip motion was not able to be determined.

The gross characteristics and the overall sense of movement of the WFZ was accurately determined in the design geotechnical model, however, the number and position of the major fault traces was not realised.

The zone of minor faulting near the intersection of Flinders Street and South Dowling Street interpreted in the design model was intercepted during tunnelling, and was termed the "Hutchinson Place Shear Zone" (Figure 1). The zone was intercepted some 190m in from the main tunnel southern portal and comprised a number of closely spaced joints shears and faults. Vertical displacement of up to approximately 8mm was observed on some planes. The overall orientation of the zone is interpreted to be subparallel to both the WFZ and the major joint set, ie northeast - southwest.

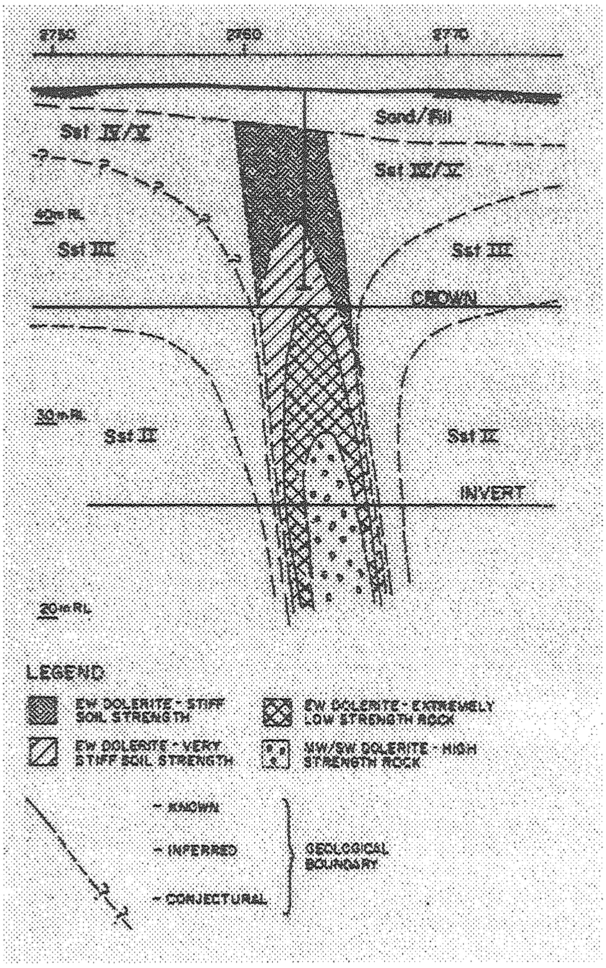

Figure 9 Insert caption here if at all possible at any stage

4.7 DYKES

Both the Great Sydney Dyke and the Oxford Street Dyke were intersected within 1m of their expected positions.

The post-excavation geotechnical model for the Great Sydney Dyke is shown in Figure 9. Key features of the model are:

- at tunnel level, the dyke comprises extremely weathered low strength rock with stiff clay near the sandstone contacts,
- the central core of the dyke is relatively less weathered than material near the contacts with the surrounding sandstone rock,
- the quality of the dolerite improves with depth to very high strength rock, and
- sandstone rock quality is degraded adjacent to the dyke. This is likely to reflect a combination of;
    - fracturing/faulting within sandstone prior to emplacement of the dyke,
    - baking of the sandstone immediate to the dyke during emplacement,
    - possible post emplacement faulting and weathering effects.

The design geotechnical model for the tunnel was able to quite accurately determine the location, thickness and material properties of the dyke at intersection depth.

5 CONCLUSIONS

Engineering Geological mapping and database upkeep during construction largely confirmed the design geotechnical model. In situations where the encountered excavation conditions varied from that predicted, data collected during mapping allowed for accurate redesign of tunnel support in 'real - time'.

The mapping sheets provide a complete record of the geotechnical conditions encountered during tunnelling. Given the potential traffic volumes through the tunnel over the 100 year design life of the tunnel, the effort put in to the engineering geological mapping and data collection during construction is considered appropriate.

This paper therefore provides a working example of the issue raised by Stapledon (5) of 'putting the "geo" into "geotechnical", by

- using graphic representation of data on excavation logs,
- presenting detailed logs of high standard, and
- regular and on-going comparison between the encountered tunnelling conditions and the design geotechnical model.

ACKNOWLEDGEMENT

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