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TUGGERANONG SEWER TUNNEL, ACT

ENGINEERING GEOLOGY COMPLETION REPORT, 1977

by

D.C. PURCELL

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SUMMARY

1. Construction of the tunnel and appurtenent works began on 2/3/72 and was completed on 7/12/75. The tunnel was operational on 8/12/75.

2. The rock was considered too hard for a tunnel boring machine, and the contractor used a standard drill-shoot-muck procedure, erecting support where necessary. Two headings and shaft excavations were worked simultaneously.

3. Geological conditions were essentially as predicted by geology and geophysics. Poor geological and hydrological conditions at the rhyodacite-dacite contact were predicted, but poor tunnelling in the rhyolite section was not accurately predicted.

4. 1437 steel sets (RSJ) were erected, for a total length of 1630 m. Most of the poor ground was encountered in the northern half of the tunnel (rhyodacite and rhyolite) and was due to a greater concentration of faulting and less cover generally than 30 m above the crown elsewhere.

5. Overbreak during construction was minimal, with an average figure somewhere between 11 and 20 cm outside of the c-line. Overbreak sections with significant overbreak was controlled by faults, sheared zones, seams, and incompetent rhyolite.

6. Concrete placement was a problem in the confined space of the tunnel. Repairs to the lining and grouting were thought to be reasonably successful; total groundwater flows into the completed tunnel were limited to about 32 m^3/h (monitored by V-notch weirs).

7. The only significant problems encountered in excavation of the 5 shafts were excessive groundwater flows into shaft 2 from a thick alluvial aquifer; and in shaft 5 - including the shaft-tunnel transition section - where poor rock conditions associated with a faulted rhyolite dyke necessitated complete steel support.

8. Effects of ground vibrations from tunnel blasting were slight, and only one complaint was registered. Tests showed the vibration level to be well within the specifications of 1.5 inches per second (3.8 cm/s)

9. The most significant groundwater flows into the tunnel were restricted to the north heading, especially the section of tunnel between the Cotter Road and Hindmarsh Drive and also in the region of the dacite-rhyodacite contact (stn 178+00 to 179+00). Total flows into the tunnel until breakthrough (23/4/74) were 955000 m^3 (the estimated amount before construction was 910000 m^3).

10. Seismic refraction profiles were particularly useful in predicting tunnelling conditions. Experience gained in this tunnel has enabled excellent correlations to be made between seismic velocities and tunnelling conditions in subsequent tunnels (Ryan, Pine Ridge, Ginninderra).

11. Bieniawski's Geomechanics classification and actual conditions encountered in the tunnel have been compared. Before any assessment of his classification and its usefulness in predicting stand-up times can be made, additional data are required.

1. INTRODUCTION

The project comprised everything necessary for the construction of a 2 m diameter trunk sewer tunnel, 29,552 feet long (9007.45 m), complete with manhole shafts etc, from the Tuggeranong Creek (inlet portal) to near the Western Creek Sewerage Treatment Plant (outlet portal; Figure 1). This tunnel will convey all raw sewerage from the Tuggeranong area into the Molonglo Interceptor Sewer System, which conveys it for treatment to the Lower Molonglo Water Quality Control Centre. After treatment at the Control Centre the effluent will be discharged into the Murrumbidgee River below Sturt Island.

The contract was awarded to Pearson-Bridge Group Engineering for $6.3 m. Excavation of the works commenced in June 1972 and was completed in April 1974. The contract was completed on 7th December 1975 and the tunnel was operational one day later. The Department of Construction (DC) designed and supervised construction of the works for the National Capital Development Commission.

A pre-construction geological report was prepared by the Bureau of Mineral Resources, Geology and Geophysics for DC, and selected parts were subsequently incorporated into an 'Information for Tenderers' document. On request from DC, BMR provided a Project Geologist whose duty was to provide DC with a complete geological service during construction.

The tunnel was driven by the conventional drill-blast-much method: excavation proceeded from the north and south headings. Five shafts were excavated from the surface to tunnel level by drilling and blasting. All shafts were offset 4 m from the tunnel centre-line.

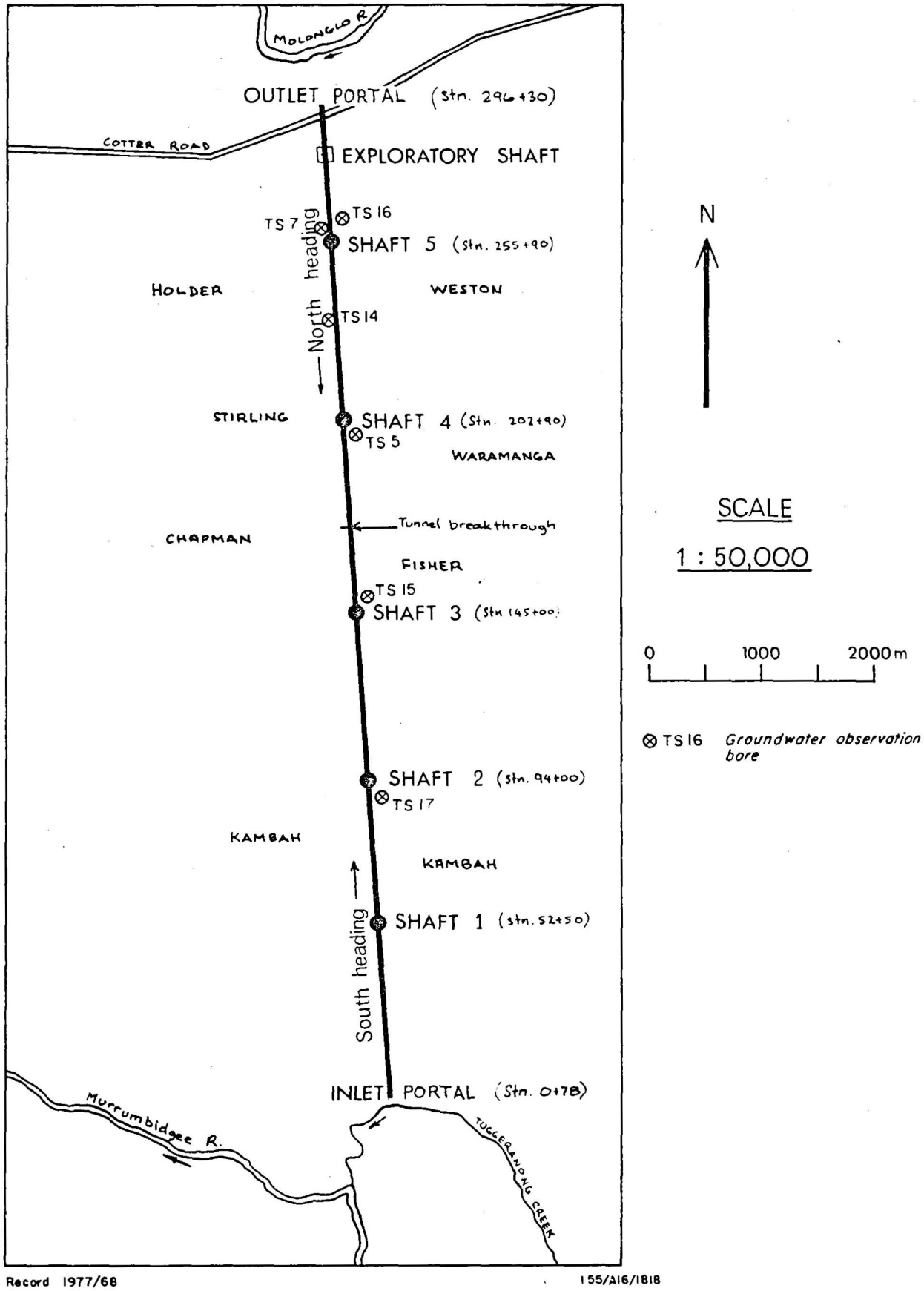
Most measurements mentioned in the text are metric values except tunnel stations (which are in hundreds of feet) and specifications for some engineering structures and materials.

2. GENERAL GEOLOGY

The geology exposed in the excavations did not differ greatly from that outlined in the geological design report (Purcell, 1974). The lithologies were generally predicted with accuracy, and the prediction of main rock defects was adequate for tunnelling.

TUNNEL LAYOUT

FIGURE 1



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Three main rock types were intersected by the tunnel (Fig. 2):

(i) Blue-grey dacite crops out from the inlet portal (south) to station (stn) 177 + 25. Apart from occasional defect zones the dacite was very hard and strong, with generally excellent tunnelling conditions. The dacite appears massive, but palaeomagnetic studies by M. Idnurm (BMR, pers. comm., 1973) show that it is bedded and dips generally southwards at about 4° from horizontal; these studies also show gentle folding of the dacite near its contact with the underlying rhyodacite.

(ii) Rhyodacite crops out north from tunnel stn 177 + 25. It is generally less competent than the dacite; sheared and fractured zones, quartz-epidote veins, and alteration (mainly hematite staining are common); the filling of defects with calcite is more common than in the less permeable dacite. The contact zone with the dacite is faulted, veined, and weathered.

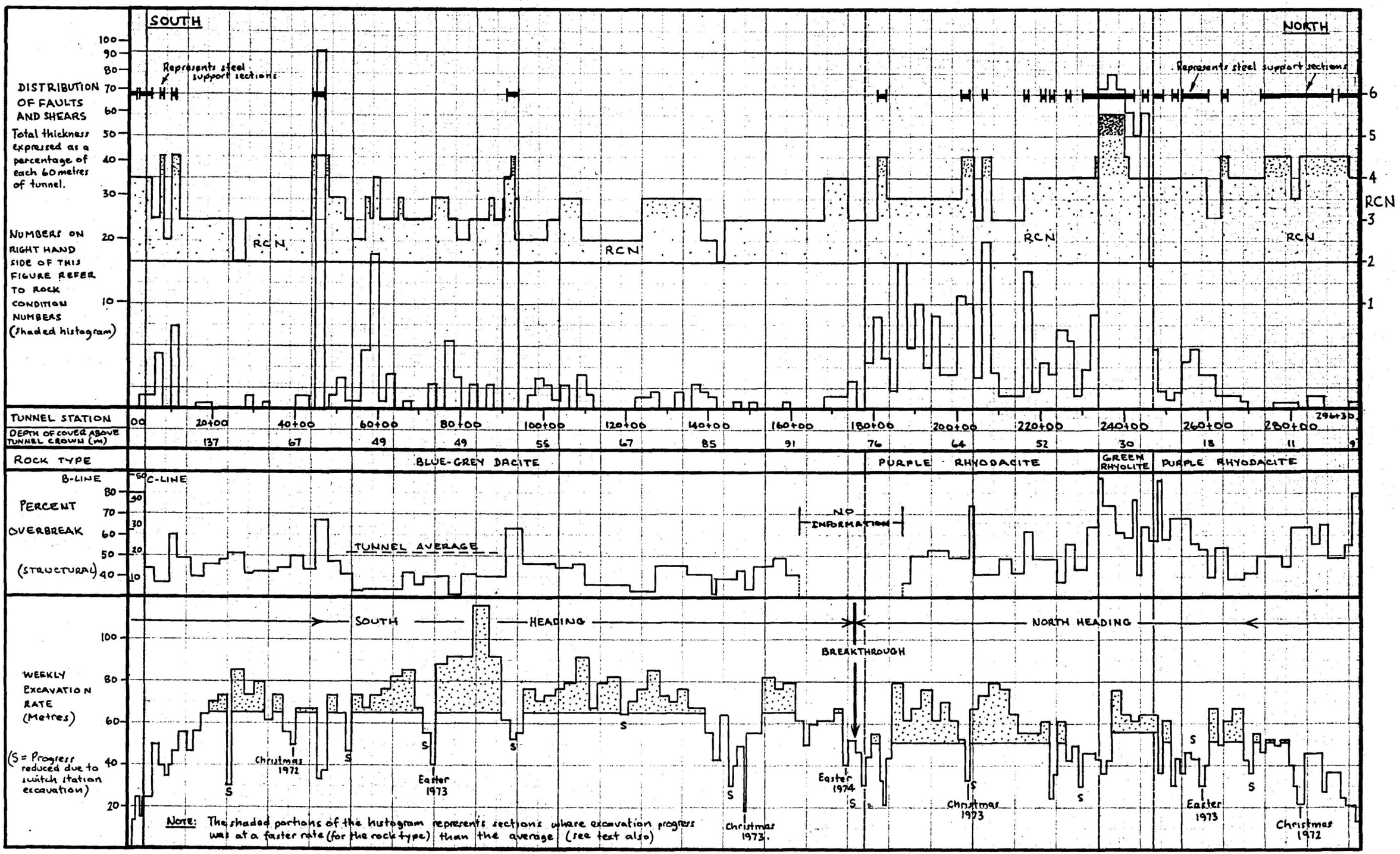
(iii) Rhyolite is younger than the rhyodacite, which it intrudes. It is coarse to fine-grained, green, and often with abundant pyrite. Apart from the main section of rhyolite between stn 234 + 70 to 247 + 00, a few narrower dykes occur close to the main body (see detailed tunnel logs).

All rock types are veined to some degree. Veins are common in the rhyodacite and dacite, and are mainly confined to fractured or faulted zones in the rhyolite. Quartz-epidote veins are the most common; they are especially prominent near faults and sheared and fractured zones. Calcite veins and deposits on defect surfaces are common in all rock types. Where there was seepage or groundwater flow, a film of calcite was often deposited on open joints or fracture surfaces within a day of tunnel intersection; in some instances the fractures were completely healed within a few weeks.

Alteration, as distinct from chemical weathering was mainly confined to the rhyodacite and rhyolite, or to rock adjacent to fractured and sheared zones regardless of the rock type. Most altered zones were stained orange-red by hydrothermally deposited hematite.

SUMMARY OF TUNNEL CONDITIONS

FIGURE 2



3. ENGINEERING GEOLOGY

This part of the report deals with rock type, structure, weathering, and groundwater, and the degree to which they determined tunnelling conditions. Assertions made in the pre-construction report on expected tunnelling conditions are critically examined and compared with actual tunnel conditions.

Table 1 and Figure 1 summarise geological conditions along the tunnel line. Summary tunnel logs appear as Plates 1 to 6.

'Rock condition' mentioned in this report is based on Terzaghi's Rock Condition Number (RCN), in which numbers ascending in sequence correspond to an increase in tunnelling difficulty, hazard, anc cost. However, Terzaghi's Numbers have been partly modified to suit local conditions; a full description of each of the eight categories appears in the definitions in the Appendix.

3.1 ROCK TYPES AND TUNNELLING CONDITIONS

3.1.1. Dacite

The dacite produced by far the best tunnelling conditions of the three rock types (refer to Table 1).

Weathering. A high proportion of the dacite excavated was fresh. Table 1 combines fresh and slightly weathered rock into one category; this was done as tunnelling conditions through slightly weathered rock were not significantly worse than conditions in fresh rock, all other factors being equal.

The 98 percent fresh/slightly weathered figure given for dacite is significantly higher than for the rhyodacite (70%) and rhyolite (28%) probably for the following two reasons: (a) the dacite is less faulted, sheared, or fractured, and the rock mass is generally very tight, and (b) the dacite is more resistant to weathering agents owing to its mineralogy. Depth below the surface is not considered as a third reason, because the depth of cover above the rhyodacite between stns 180 + 00 and 230 + 00 is

TABLE 1. STATISTICAL ANALYSIS OF GEOLOGICAL CONDITIONS

Rock type/ * Measurement	Dacite	Rhyodacite	Rhyolite	Totals
Section length	5403	3231	375	9009
No. of steel sets	195	1047	195	1437
Length steel supported (% of rock type)	206 (3.8%)	1190 (37%)	234 (62%)	1630 (18%)
Length of tunnel section with RCN 1, 2, 3 (%)	4548 (84%)	1319 (41%)	36 (10%)	5903 (65%)
Length of tunnel sections with RCN 4 (%)	786 (14.5%)	1476 (46%)	167 (45%)	2429 (27%)
Length of tunnel sections with RCN 5 or worse (%)	68 (1.5%)	436 (13%)	171 (45%)	675 (8%)
Length of tunnel section where rock is fresh and/ or slightly weathered(%)	5299 (98%)	2266 (70%)	107 (28%)	7672 (85%)
Length of tunnel section where rock is moderate-ly weathered (%)	78 (1.5%)	492 (15%)	15 (4%)	585 (7%)
Length of tunnel section where rock is highly and/or extremely wea-thered (%)	26 (0.5%)	472 (15%)	253 (68%)	751 (8%)

* Section lengths are in metres

similar to that of the dacite in the preceding section (Fig. 2).

Structure. All rock intersected by the tunnel was closely to moderately jointed. Continuous joints in the dacite were often widely spaced, more so than in the rhyodacite or the rhyolite. (A continuous joint is defined as one which is at least equal in length to the diameter of the tunnel and can be traced from springline to springline). Joints were generally tight and often completely healed, and when blasted the rock often broke across joint surfaces. Joint surfaces were found to be generally devoid of clay except in or adjacent to major defect zones. The major joint sets in the dacite, rhyodacite, and to a lesser extent the rhyolite were near vertical with a northwest or northeast strike. Detailed steroplots (joints and faults) have been filed in the project box.

The dacite was sheared and fractured to a relatively small degree compared with the rhyolite and rhyodacite. The average RCN was 3, which shows the dacite to be more competent than the rhyolite and rhyodacite (Fig. 1).

Rock condition and support. Only 3.8 percent or 206 m required steel-set support. Except for the first 122 m of tunnel (from the inlet portal) all supported sections of tunnel were in faulted, fractured, or sheared rock. Randomly located rock bolts were used in three sections of tunnel: stns 1+60 to 11 + 70, 49 + 50 to 53 + 50, and near 271+ 00; these were mostly inserted to stabilise blocky and seamy ground related to fractured zones. At least 50 percent of these bolts were considered by the geologist, on hindsight, not to be necessary in such a small-diameter underground opening.

3.1.2. Rhyodacite-dacite contact.

The contact zone (stn 178 + 00) is faulted and fractured. An intrusive rhyolite dyke along the contact is offset by a fault. Support was not required in the contact zone area.

3.1.3. Rhyodacite

Tunnelling conditions in the rhyodacite varied considerably; some

sections were excavated in fairly hard and strong competent rock while other sections were badly affected by weathering or fracturing. The rhyodacite was never found to be as hard, strong, or abrasive as the dacite.

Weathering. About 70 percent of the rhyodacite was fresh or slightly weathered, 15 percent moderately weathered, and 15 percent highly and extremely weathered.

Structure. Joints in the rhyodacite are generally not as tight or as unweathered as in the dacite. Clean horizontal sheeting joints open up to 5 to 8 cm, between stns 250 + 00 and 256 + 00 admitted large initial water inflows. Sheared and fractured zones are much more prevalent in the rhyodacite than in the dacite, particularly the section of tunnel between the rhyolite intrusion (station 234 + 70) and the dacite contact at station 178 + 00 (refer to Figure 2 and summary logs Plates 1 to 6). The rhyodacite is typically highly veined; the veins often weathered, with clay seams criss-crossing the rock mass.

Rock condition and support. Table 1 shows that 37 percent or 1190 m of tunnel is the rhyodacite required steel-set support. Long sections of steel support were necessary, particularly those sections of tunnel with less than 25 m of cover. Most of the highly and extremely weathered rock required steel-set support; moderately weathered rock required support where the rock was fractured or sheared or the joints were open and continuous.

3.1.4 Rhyolite

Tunnelling conditions in the rhyolite were mostly poor. The rhyolite is generally soft and weak, although in some sections - between stns 243 + 00 and about 247 + 00 - it was quite competent and did not require support (see summary log - Plate 5). In other sections, clay-filled joints and fractures in relatively fresh and hard rock resulted in a weak rock mass requiring steel support. The overall condition of the rhyolite is best illustrated by the high percentage (45%) of RCN 5 or 6, whereas the tunnel average for RCN 5 or 6 is 8 percent.

Weathering, alteration, and structure. Table 1 shows that 68 percent of the rhyolite is highly and extremely weathered, and only 28 percent is fresh or slightly weathered. The 68 percent includes rhyolite that is altered

as well as rock that is very weathered. The many fractured and sheared zones and veins have generally been weathered to clay. Two rhyolite dykes, probably associated with the main intrusion were intersected near stns 250 + 00 and 256 + 00; the first was moderately strong and was not supported, but the other - at the Shaft 5/tunnel junction - required support.

Rock condition and support. The rock mass is mostly weak and very unstable, and 62 percent of the rhyolite was heavily supported. Numerous criss-crossing clay-filled sheared and fractured zones (mainly 1 to 1.5 m wide) and clay-filled joints are the main features of the rhyolite. Stand-up time was close to zero in some sections, and large amounts of overbreak occurred, mainly above springline (up to 4.5 m). The weight of loose blocks of rhyolite resting on support caused some deformation of steel sets and timber lagging, particularly near stn 240 + 40.

3.2. OVERBREAK

3.2.1. Overbreak and tunnel profile

Overbreak refers to volume of rock excavated outside either B or C lines (Fig. 3).

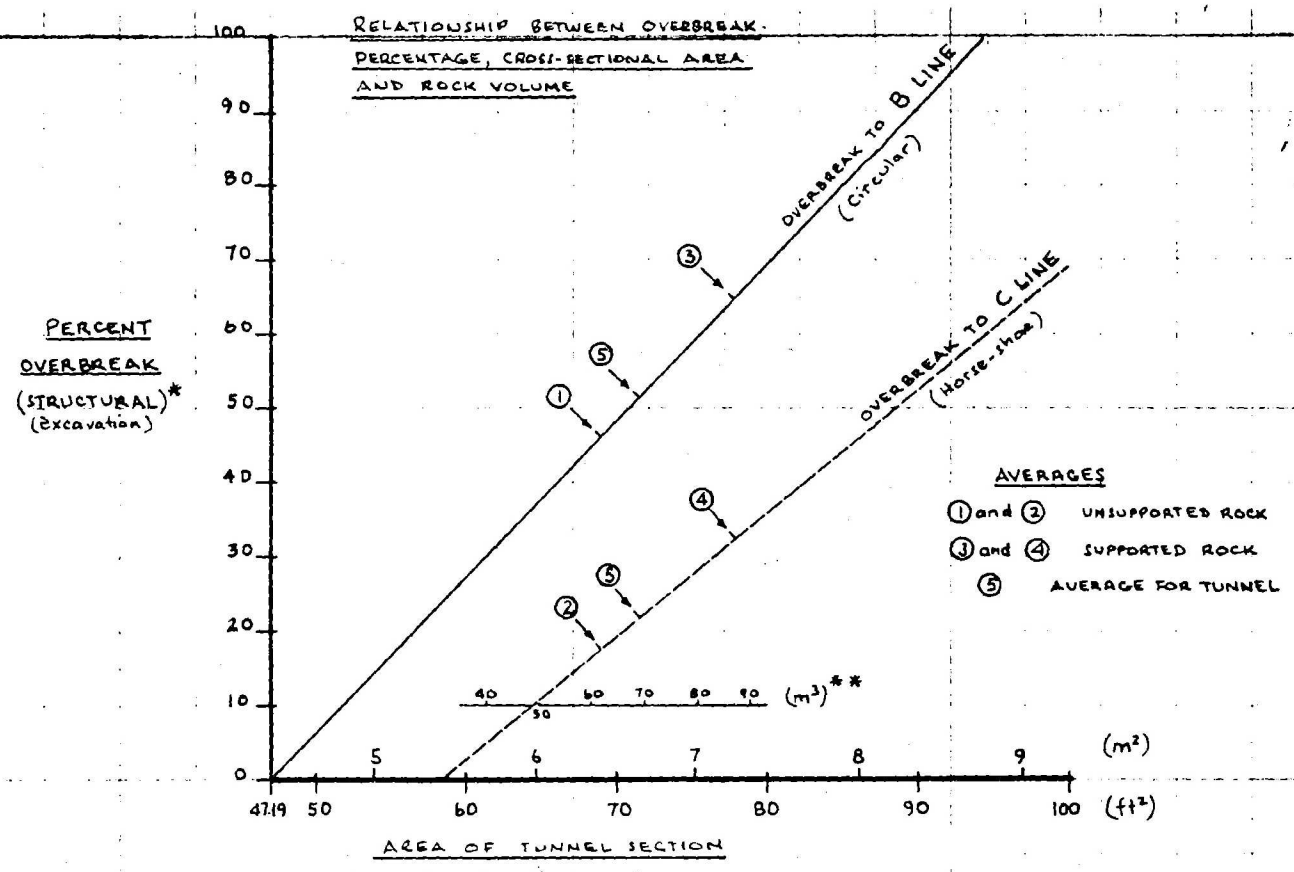
Defect spacing, orientation, and surface condition almost solely determined the tunnel profile and consequently the overbreak. Other factors controlling the amount of overbreak were overblasting (in places) and the trait of the 'Montebart' drilling jumbo, which had a tendency to angle its holes slightly outwards and the tunnel assumed a saw-tooth shape in plan view. The saw-tooth shape is most noticeable in sections of RCN 2 and 3; in RCN 4, 5, and 6 the saw-tooth tunnel shape was not in evidence owing to the overriding effect of poorer quality rock. Jointing was generally the main geological factor in the determination of tunnel profile.

Sections of tunnel excavated through RCN 2 or 3 finished with a tunnel profile reasonably close to design. Sections of tunnel in RCN 4, 5 or 6 finished with a ragged profile, as blocks of rock between intersecting seams, prominent joints, etc., fell during or immediately after blasting, although continual fretting or loosening of rock occurred in places, especially in the rhyolite.

Short sections of tunnel have profiles affected by faults, sheared

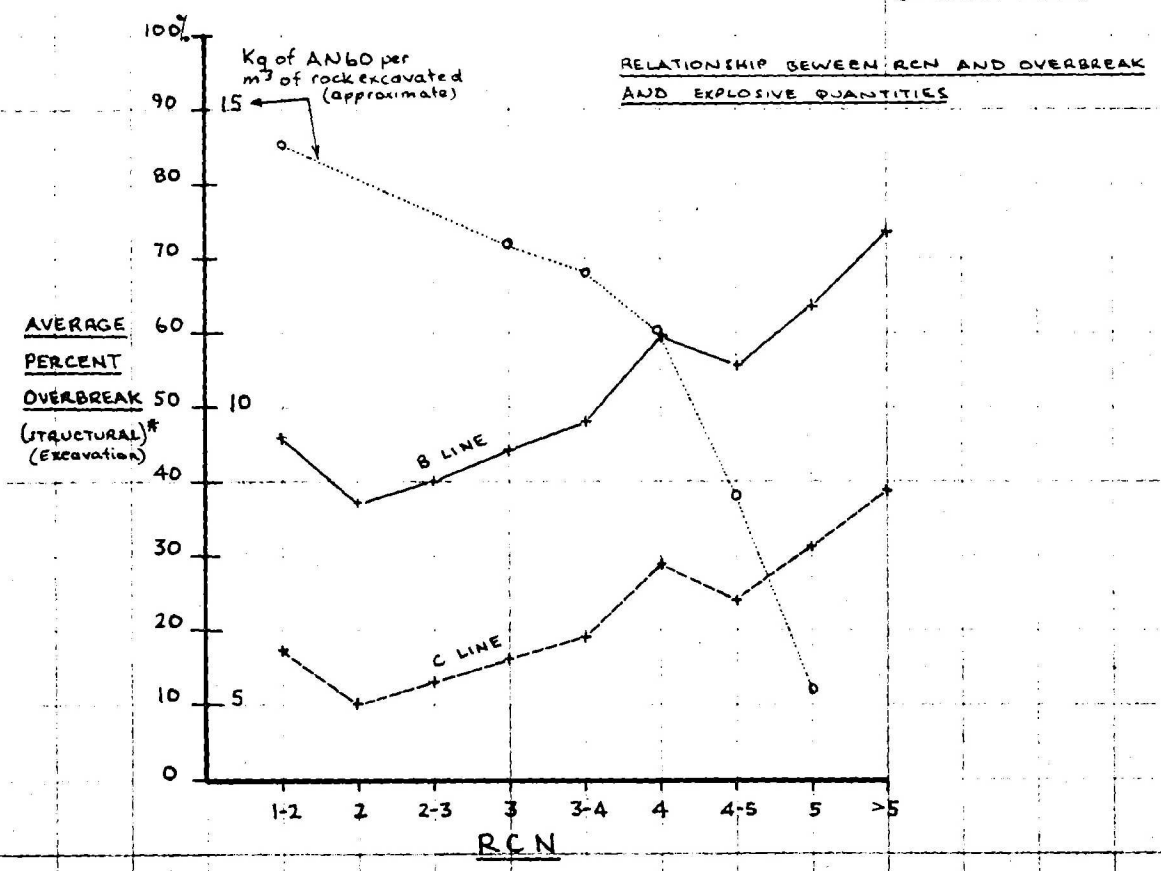
OVERBREAK CORRELATIONS

FIGURE 3



* As distinct from "concrete" placement derived overbreak figures

** These figures represent m^3 of overbreak per 100 foot (30.5 m) linear tunnel length, and correspond to figures given on X co-ordinate of this graph. From this graph the total volume of overbreak from stns. 0+78 to 296+30 can be calculated thus: Average overbreak (51.3%) corresponds to 69 m^3 per 30.5 m of tunnel length – ∴ 69 m^3 x tunnel length ÷ 30.5 m = 20,391 m^3 (outside of B-Line)



or fractured zones; the ultimate shape depended on the defect attitudes. Near horizontal seams, clay-coated joints, or shears near crown level caused some overbreak, resulting in a flat crown. Similarly near-vertical seams, joints, or shears running near parallel to tunnel direction resulted in near vertical walls.

Prominent open horizontal sheeting joints continuous for more than 30 m between stns 250 + 00 and 256 + 00, and prominent near-vertical joints almost parallel to tunnel direction, produced a square tunnel profile. In most of the remainder of the tunnel, horizontal joints were not as prominent.

3.2.2. Overbreak calculations

Tunnel overbreak has been calculated by (i) using survey data (referred to here as excavation or structural overbreak), and (ii) using grouting and concrete placement quantities to compare the emplaced concrete volume with the design concrete volume (referred to as concrete placement overbreak). Results using survey data can be obtained by referring to either the total excavation cross-sectional area (Case 1, Fig. 4) or the designed concrete cross-sectional area (Case 2, Fig. 4). Results using actual concrete and grout quantities placed are derived from a comparison of the emplaced concrete volume with the designed concrete volume (Case 3, Fig. 4).

Overbreak calculated using survey data. During construction the contractor surveyed a tunnel section every 10 to 20 feet using a 'star probe' centred on the centreline. The survey records are complete except along an unsurveyed section between stns 161 + 80 and 167 + 00 (only 180 feet of this section was steel supported). Percentage overbreak beyond the B line was calculated for each section surveyed (2170 in all), and the results were plotted onto the detailed tunnel logs as an average figure for each 100-foot section of the tunnel. Results given in Figures 3 and 4 are mostly self explanatory; however some data need clarification:

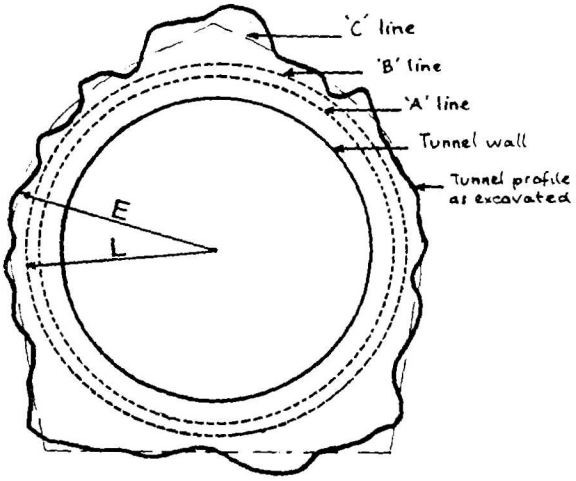
(i) Figure 4, Case 1 shows the steel-supported dacite to have an overbreak percentage seemingly too high. This may be due to the fact that almost 50 percent of the steel-supported dacite occurs in the first 120 m of tunnel excavated (stn 0 + 78 to 4 + 80); that is, before excavation techniques had been perfected and crew expertise fully developed.

OVERBREAK FORMULAE AND RESULTS

FIGURE 4

(NOTE: Tunnel sections not to scale)

CASE 1 EXCAVATION OVERBREAK FROM TUNNEL SURVEY



$$\text{PERCENT OVERBREAK} = \frac{E-L}{L} \times 100$$

Where: E is the excavated cross-sectional area
L is the cross-sectional area inside the reference line i.e. 'C' or 'B' line.

RESULTS

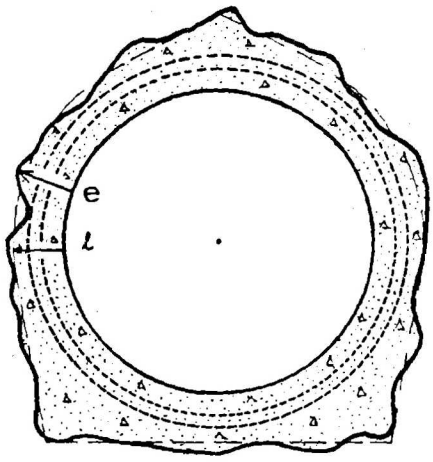
TUNNEL REFERENCE LINE OR ROCK TYPE	AVERAGE PERCENT OVERBREAK		
	UNSUPPORTED TUNNEL	STEEL SUPPORTED TUNNEL	* TUNNEL OR ROCK TYPE AVERAGE
'B' LINE	46.1	64.7	51.3
'C' LINE	17.3	32.3	21.7
DACITE	17 (46)**	40 (74)	17 (48)
RHYODACITE	20 (49)	29 (61)	24 (55)
RHYOLITE	21 (50)	39 (74)	32 (65)

* Includes tunnel switch stations

** 'B' Line figures in brackets

Approximate values of L are 47.2 ft² ('B' Line) and 58.6 ft² ('C' Line)

CASE 2 CONCRETE OVERBREAK FROM TUNNEL SURVEY



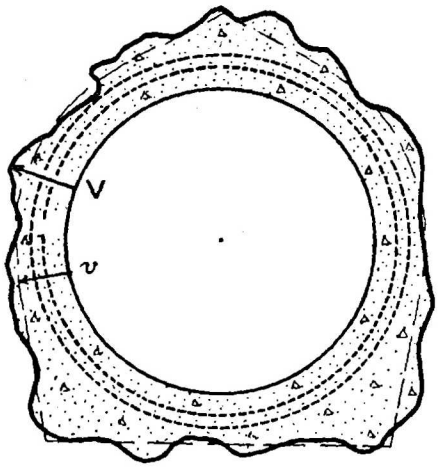
$$\text{PERCENT OVERBREAK} = \frac{e-l}{l} \times 100$$

Where: e is the cross-sectional area outside the tunnel wall
l is the cross sectional area between the tunnel wall and 'c' line

RESULTS

PERCENT 'C' LINE OVERBREAK	
UNSUPPORTED TUNNEL	39
STEEL SUPPORTED TUNNEL	69
AVERAGE	44

CASE 3 CONCRETE OVERBREAK FROM CONCRETE PLACEMENT DATA



$$\text{PERCENT OVERBREAK} = \frac{V-v}{v} \times 100$$

Where: V is the volume of concrete placed
v is the volume between the tunnel wall and 'c' line.

RESULTS

PERCENT 'C' LINE OVERBREAK		
	INCLUDES GROUT	MINUS GROUT*
UNSUPPORTED TUNNEL	22.3	20.4
STEEL SUPPORTED TUNNEL	53.3	42.6

* TUNNEL AVERAGE = 24.5%

(ii) The lower graph in Figure 3 shows RCN 1-2 as having a higher percentage overbreak than expected. It also shows explosive quantity versus RCN and indicates a slightly greater amount of explosive used in RCN 1-2 sections of tunnel than expected from the trend shown. These sections of the tunnel may have been overblasted, resulting in 10 percent more overbreak than expected. Twenty-three sections were surveyed in RCN 1 and 2 and all except 2 sections measured gave overbreak figures with 10 percent (average 5%) of the mean value of 46 percent (B line).

In unsupported rock an average of 48 kg per 2.1 m pulled round was consumed (equivalent to 3.2 kg/m^3). This appears heavy compared with open-cut surface work (0.45 kg/m^3), but is acceptable considering the burn cut used, the need for good fragmentation for ease of mucking in such a small tunnel, and the need for face advance at the expense of more efficient powder consumption. Consumption in supported rock varied with RCN but a typical 1.2 m round averaged about 14 kg, averaging 1.8 kg/m^3 (Fokkema & Noack, 1976).

Figure 3 shows a higher percentage overbreak for RCN 4 than expected. There are three possible reasons for this:

(1) measurements made in and near the rhyolite section (stn 234 + 70 to 247 + 00) made up about 50 percent of the RCN 4 sections tallied;

(2) prominent horizontal and vertical joints, especially in the purple rhyodacite between stns 250 + 00 and 256 + 00, resulted in a square tunnel profile with excessive amounts of overbreak; and

(3) RCN 4 was found to present a borderline situation in deciding whether or not to erect steel support.

It was thought that some unsupported sections of RCN 4 yielded more overbreak than would have occurred if support had been erected immediately after blasting. However, calculations based on the contractor's tunnel section surveys show that supported sections of RCN 4 gave an average overbreak of 65.5 percent compared with 53.3 percent for unsupported sections of RCN 4. In other words, steep support was generally erected only where unstable rock

was falling in immediately after blasting; erection of the support immediately after blasting kept the overbreak within 12 percent of that in the more stable unsupported sections of RCN 4.

Concrete placement overbreak.

Overbreak figures (Fig. 4) derived from concrete placement are shown to be less than figures derived from excavation overbreak measurements.

Possible reasons for the differences are given by Fokkema & Noack (1976):

1. The surveyor may have tended to look for the deepest points using his star probe
2. Analysis of the concrete design mix did reveal initial overyielding (increased volume)
3. Overbatching of concrete ingredients at the plant.

The 'concrete placed' (including grout) give an average overbreak outside the C line of 6 cm for unsupported tunnel and 13.5 cm for supported tunnel; the excavation measurements from tunnel survey give figures of 11 cm and 20 cm. A figure somewhere in between is probably closer to the true overbeak.

3.3. TUNNEL SUPPORT

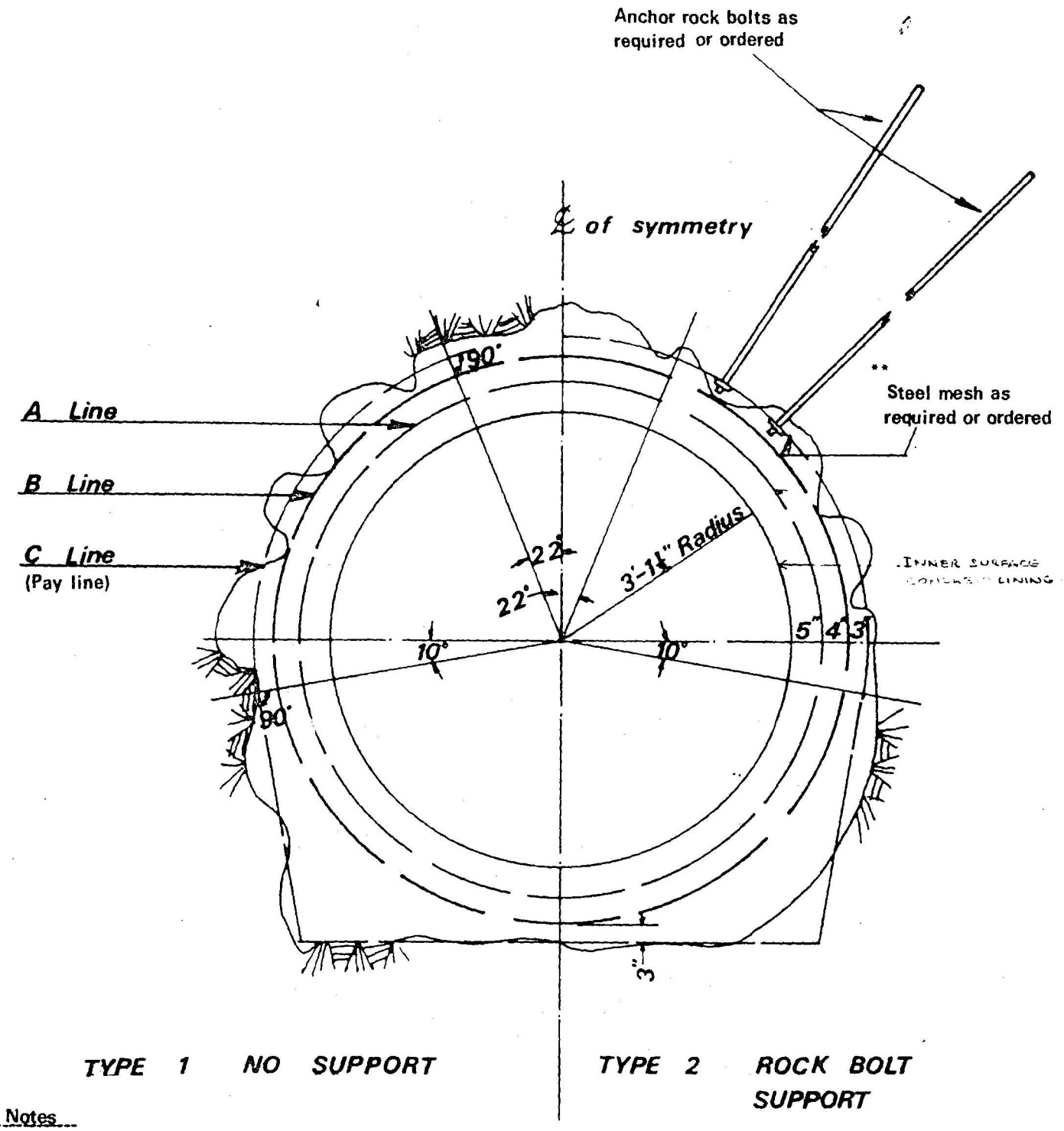
3.3.1 General.

A typical tunnel section showing the type of supports used, tunnel dimensions, A, B, and C lines, etc. is shown in Figures 5, 6, and 7. It should be noted that invert struts were not used. Most steel sets were spaced at 1.2 m centres, some at 0.9 m, and only a few at 0.6 m. Steel supports were 4" x 3" x 10 lb RSJs (two segments) braced with 5/8" mild steel tie rods. Inferior rock generally required steel sets and very few bolts were used. Those used were 'TITAN' 1" diameter x 6 ft slot and wedge.

In preparation for concrete lining, cleaning-up operations preceded the formwork erection by a hundred or so metres. All loosened material, either inside or outside of the C line was removed and timber lagging was

TUNNEL SUPPORT

Figure 5



Notes

A Line No material used for the ground support of the tunnel or unexcavated material shall remain within this line

B Line Except for circumferentially placed steel supports and longitudinal tie bars, no part of any ground support shall remain within this line. No excavated or unexcavated material, logging, spreaders, bracing or crown bars shall remain within the B Line

C Line The C Line is the pay line for excavation and concrete and does not mean restriction of any ground supports

** Not used in this tunnel

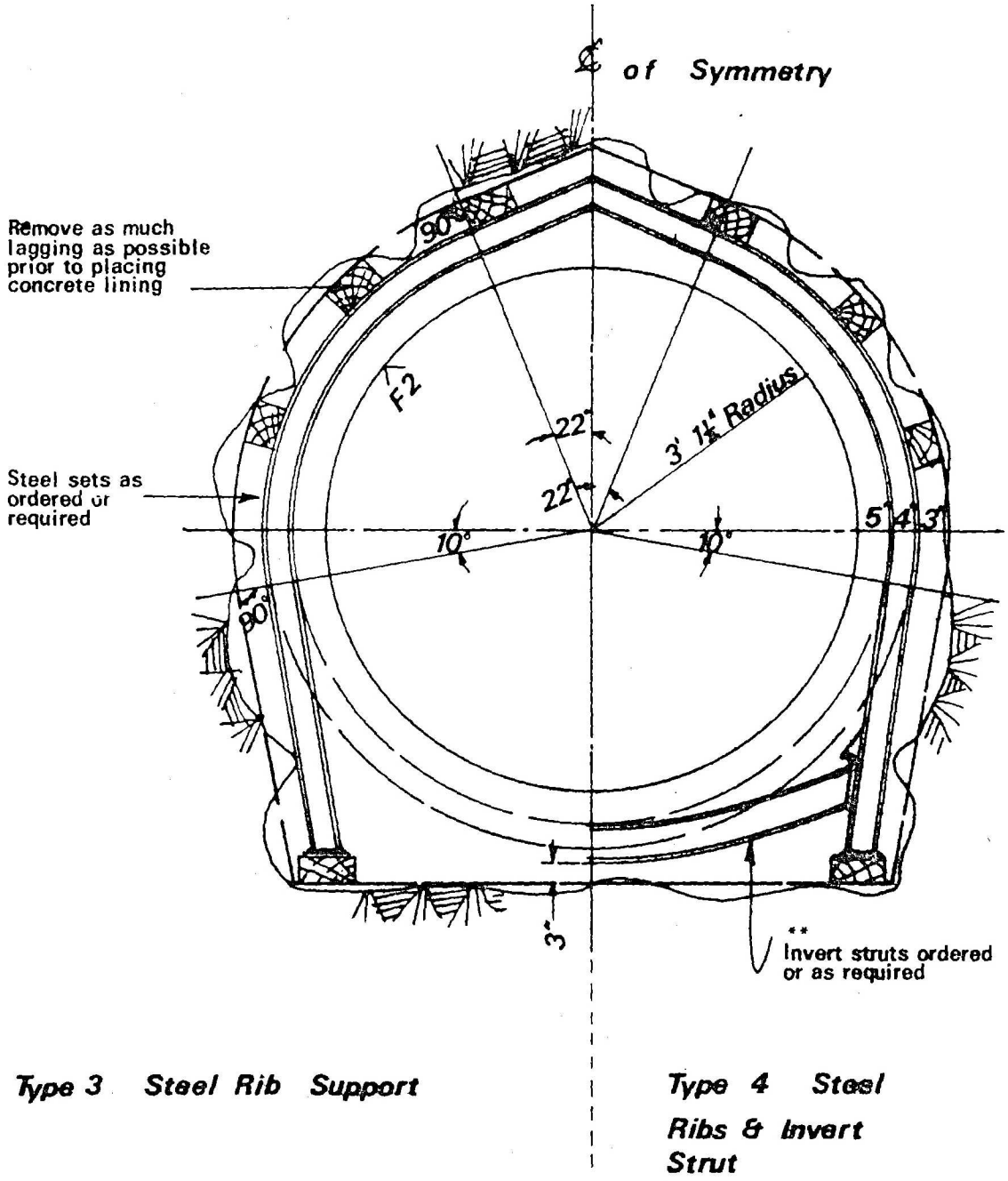
1 0 1 2 3 4

Scale in feet

(After DHC. Design Specification drawing – sheet 2)

TUNNEL SUPPORT

Figure 6



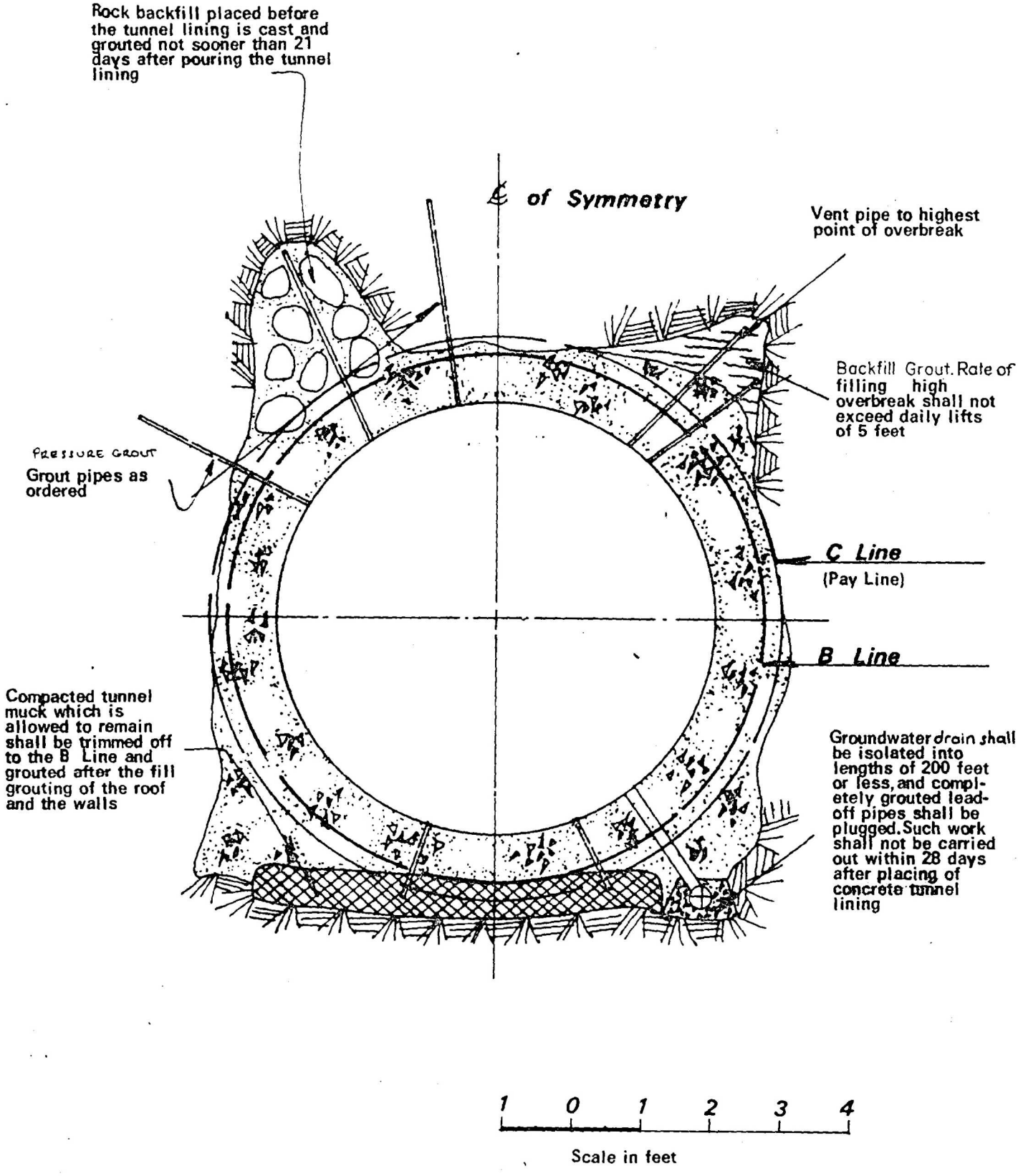
NOTES – Notes in Figure 5 also apply to this Figure

** Not required in this tunnel

(After DHC Design Specification drawing–sheet 2)

GROUTING DETAILS

Figure 7



Note: Notes in Figure 5 also apply to this Figure

(After DHC Design Specification drawing – sheet 2)

lagging was either removed or rewedged. In some sections the lagging and sets had no weight bearing on them.

Much of the support in the rhyolite section was supporting blocks of loose rock that had fallen onto the lagging, behind which a large amount of overbreak had accumulated. Only in isolated sections of the rhyolite did the rock mass exert pressure on the supports; for example, stn 240 + 40. In some sections of rhyolite, many sets moved out of alignment, up to about 20 cm, and protruded into the A line. The reason the sets were misaligned is not certain; it would have been either survey error, carelessness of the miners, collisions with the muck train, or the rock mass exerting side pressures. The latter explanation can probably not be entirely discounted as the rhyolite in these sections was generally blocky, very seamy, and loose.

3.3.2. Rock condition and support.

All of the tunnel with RCN 5 required steel-set support (8% of tunnel length); 40 percent of RCN 4 required steel-set support (10% of tunnel length). Support was not required in RCN 1, 2, or 3. These calculations are based on figures from Table 1, which shows that 18 percent of the tunnel required steel-set support.

3.3.3. Support of sheared and fractured zones

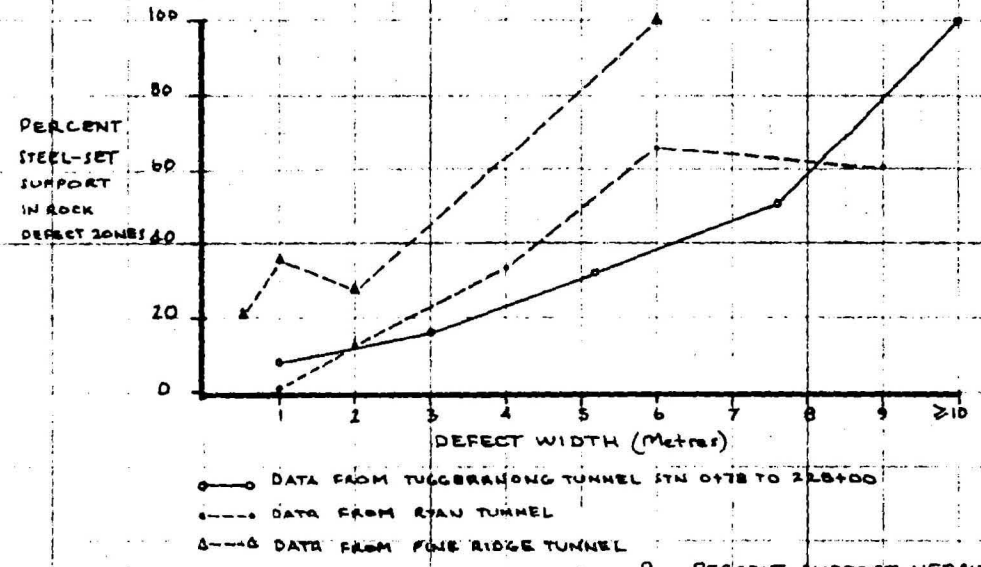
Most (about 85%) of the sheared and fractured zones less than 3 m wide that occurred in otherwise sound rock did not require steel support (Fig. 8); wherever these zones were closely spaced or a unfavourable attitudes to the tunnel, steel support was generally required. Data from Ryan and Pine Ridge tunnels (slightly larger in diameter) has also been plotted in Figure 8 for comparison.

With increasing width, a greater percentage of sheared zones in otherwise sound rock required support. All shears greater than 10 m wide were supported regardless of their attitude to tunnel alignment.

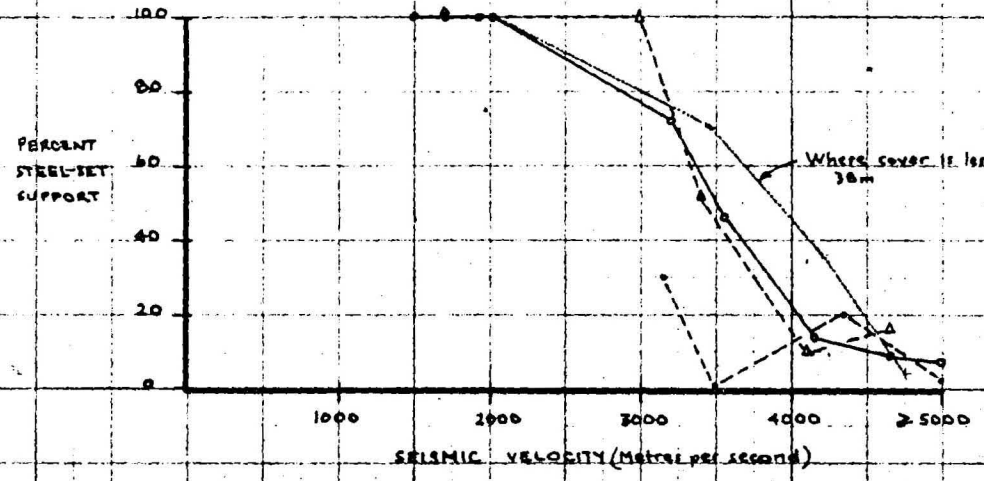
Most of the sheared and fractured zones (most of which are less than 1 m wide) north of Station 228 + 00 were supported. These shears and

PERCENT SUPPORT VERSUS SEISMIC VELOCITY & DEFECT WIDTH

8 PERCENT SUPPORT VERSUS DEFECT WIDTH



9 PERCENT SUPPORT VERSUS SEISMIC VELOCITY



fractures were generally separated by weathered, blocky, and seamy rock which generally would have required steel-set support even if the shears and fractures had not existed.

As stated previously, 18 m out of every 100 m of tunnel excavated required steel-set support. Of this 18 m, shearing or faulting has accounted for only 5 to 6 m; rock classed as highly weathered and/or blocky and seamy, which includes most of the rhyolite section, has accounted for the remaining 12 to 13 m.

In Figure 9 an attempt has been made to correlate seismic velocities and support. In situ measurements are required for a true assessment as velocities from near the surface were projected down to tunnel level.

3.4. TUNNEL CONCRETE AND GROUT

Only a brief resume is given here as a detailed account has been given by Fokkema & Noack (1976). Details of shaft concrete and grout are not repeated.

3.4.1. Concreting

Concreting was by a continuous full circle operation commencing at the inlet portal after all excavation was completed. Concrete batch plants were set up at each shaft in turn and lastly at Weston Creek. The sub-contractor was Ready Mix Concrete Pty Ltd.

Concrete was not being compacted satisfactorily owing to difficulties in manipulating vibrators through the formwork doors into the limited space between the concrete formwork and the excavated rock surface. Compaction was improved by increasing the slump characteristic of the concrete to 18 cm (i.e. wetter) and installing external air-operated vibrators, but the latter compacted only a skin near the form face. The main problems were encountered in the sidewalls between springline to near crown and showed up as ground-water inflows through apparently good concrete.

3.4.2. Backfill grouting (Fig. 7)

It was found that with such a small-diameter tunnel, greater backfill grout takes occurred on the shoulders and not in the crown as first thought. This was due to the difficulty in compaction in the shoulder areas during concrete lining operations.

Grout takes for the tunnel were as follows:

Unsupported tunnel
Crown, 6130 feet grouted, 342 cu ft injected (av. 0.06 cu ft/ft)
Invert, 130 feet grouted, 11.5 cu ft injected (av. 0.09 cu ft/ft)

Supported tunnel
Crown 5475.5 feet grouted, 15 350 cu ft injected (av. 2.84/cu ft/ft)
Invert, no attempt made to grout.

Switches
Nine switches totalling 766 feet were grouted and a total of 3050 cu ft of grout was injected (av. 4.0 cu ft/ft)

3.4.3. Pressure grouting

Pressure grouting was specified (where required) for consolidation of the rock mass behind the lining as a means of reducing leakage through the concrete lining. Pressure grouting was done after backfill grout and repairs were completed.

Owing to a lack of time available, grouting was limited to the worst sections of water inflows, namely the contact zone between stations 178 + 00 and 179 + 00, the area immediately upstream of shaft 4, and near shaft 5. Most significant groundwater inflows were through shrinkage cracks in the lining; smaller flows and seeps occurred through voids in poorly compacted concrete. Pressure grouting was generally considered successful. The geological logs of the tunnel were examined in planning the grout program and in assessing its effectiveness.

3.4.4. Water inflows

Before it was lined, the tunnel was discharging approximately 110 m^3/hour. After lining and grouting this was reduced to about 27 m^3/hour. A very gradual rise in the water-table is expected, but probably not to the pre-tunnel level; it may be accompanied by a small increase in groundwater entering the tunnel through defective concrete, even though many existing cracks and defective concrete areas may be effectively plugged up by precipitates such as calcium carbonate and limonite.

3.5. EXCAVATION RATES

Excavation progress at both headings is shown in Figure 2. Daily progress has been plotted on the detailed and summary tunnel logs. Mobilization of machines and settling in of crews resulted in relatively slow excavation progress for the first two months from both headings. Similar lead-up times, but much shorter, occurred after holiday periods. Average daily advance rate for unsupported rock was 11 m, and for supported rock was 6 m. Excavation of the south heading (commencing at the inlet portal) was about 20 percent faster than the north heading.

South heading

Advance rates in the south heading were almost always greater than 2 m per cycle; very little support was needed in this heading. Groundwater inflows were insignificant and did not slow excavation.

North heading

Owing to the poorer-quality rock in the north heading, advance rates were often around 1.2 to 1.5 m per cycle and significant time was spent erecting support. In poorer-quality rock, drilling rounds longer than 1.2 to 1.5 m often resulted in excessive overbreak that required additional support and extensive timber lagging. As a result, the oncoming shift was often delayed through cleaning-up operations and sometimes retimbering of sets. Groundwater inflows were large enough at times to slow mining operations, but inflows were merely a nuisance to mining and concrete lining operations. Constant silting-up of the muck-car rails often caused short delays through derailments.

Rock type and excavation rate. Table 2 shows the relation between excavation rate and rock type. The rate of progress in the rhyolite of 9.4 m per day seems high compared with that of the rhyodacite (8.5 m), considering that 45 percent of the rhyolite is classified in RCN5. This can be explained as there was no lead up time associated with the relatively short section of rhyolite, and excavation did not coincide with any holiday periods; also, experience gained in poor quality rhyodacite closer to the outlet portal would have helped to develop more efficient techniques.

Rock condition and excavation rate. Rock condition and support requirements have affected excavation rates (Fig. 10, Table 3). Rate of excavation of rock not requiring support was 11 m per day, or almost twice the rate in supported tunnel sections (6.3 m/day).

TABLE 2. EXCAVATION RATE VERSUS ROCK TYPE

MEASUREMENT	ROCK TYPE			
	Dacite[1]	Rhyodacite[2]	Rhyolite[3]	Totals
Length of section excavated (m)	5258	3124	396	8778
Number of working days taken to excavate	484	370	42	896
Average excavation rate (m per 24 hr working day)	10.9	8.5	9.4	9.8[4]

Notes: Only that section of tunnel between stations 6 + 00 and 294 + 00 has been considered in these calculations.

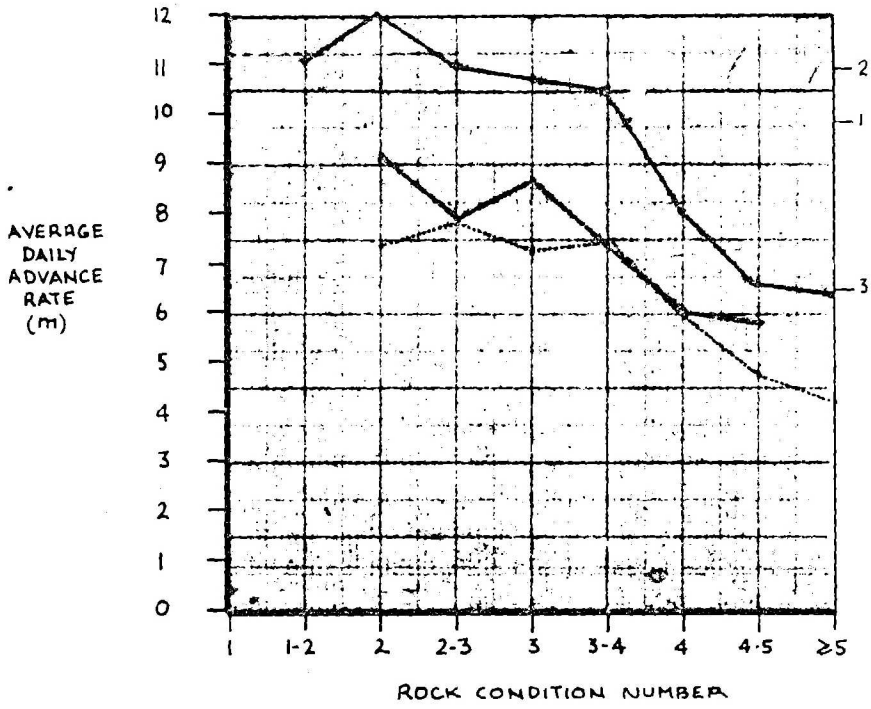
1. Stations 6 + 00 to 178 + 50.
2. Stations 178 + 50 to 234 + 00 and 247 + 00 to 294 + 00
3. Stations 234 + 00 to 247 + 00
4. Tunnel average.

TABLE 3. EXCAVATION RATE AND SUPPORT

SUPPORT	NUMBER OF WORKING DAYS SPENT	LENGTH OF SECTION INVOLVED (m)	AVERAGE PROGRESS PER DAY (m)
Sections of tunnel NOT supported	664	7310	11
Sections of tunnel supported	232	1468	6.3
Total tunnel[1]	896	8778	9.8[2]

ROCK CONDITION AND TUNNEL EXCAVATION RATE

FIGURE 10



NOTES:

.———. Data from Tuggeranong tunnel

.--------. Data from Ryan tunnel

..... Data from Pine Ridge tunnel

Tuggeranong tunnel:

1 Average tunnel advance rate

2 Average advance rate in unsupported rock

3 Average advance rate in supported rock

Sections of tunnel not included in these calculations are from:
- Stn. 0+78 to 6+00 (South portal)
- Stn. 294+00 to 296+30 (North portal)
- Sections of tunnel where switching stations excavated

Pine Ridge tunnel: Most of the data for Pine Ridge tunnel plots of RCN 4-5 and 5 were obtained from incompetent sediment beds within the dacite near the east portal. (Stn. 6+80 to 12+20)

RELIABILITY OF PLOTS CAN BE ASSESSED BY COMPARING LENGTHS OF SECTIONS TALLIED FOR EACH ROCK CONDITION NUMBER (OBTAINED FROM TUNNEL LOGS)

ROCK CONDITION NUMBER	TOTAL LENGTH (m)			NUMBER OF DAYS TAKEN TO EXCAVATE		
	TUGGERANONG	RYAN	PINE RIDGE	TUGGERANONG	RYAN	PINE RIDGE
1-2	119			10.5		
2	250	18	73	20.5	2	10
2-3	774	143	125	70	18	16
3	3706	509	570	335	58.5	79.5
3-4	1327	73	396	123.5	7	54.5
4	1079	119	330	130	19	56
4-5	678	49	98	101	8.5	20.5
≥5	378		316	57		51.5

1. Stations 6 + 00 to 294 + 00
2. Rate from portal to portal (0 + 78 to 296 + 30) was 9 m per day, i.e. 9008 m/978 days = 9 m.

3.6 GROUND VIBRATIONS AND TUNNEL BLASTING

3.6.1. Specification

The engineering specifications (CD71/15) stated that 'when blasting is carried out in the proximity of unoccupied buildings, structures or services, the peak particle velocity of the ground on the surface at such structures and services shall not exceed 1.5 inches/sec., and the amplitude shall not exceed 0.024 inches for frequencies less than 10 cycles.sec.' (Australian Standard CA23-1967 for built-up areas).

3.6.2. Measuring and equipment

BMR recorded and measured vibrations during tunnelling whenever necessary. 14 vibrations were measured between 14/8/72 and 29/1/74 using a Sprengnether VS-1200 seismograph. The Sprengnether measures the three components of ground particle velocity in inches/second (in/sec): horizontal, vertical, and radial velocities; amplitude frequency and acceleration, and a site constant (K) were also determined for each test.

Details of each vibration measurement have been filed with the BMR Project records, together with blasting patterns, charges, depths, locations, and velocity computations. The results have been plotted as Figures 11 to 14 and summarised in Table 4. Results of experimental vibration tests carried out by blasting diamond-drillholes before the tunnel was excavated have been summarised in Table 5 and plotted as Figure 12.

3.6.3. General results

Resultant particle velocity: The maximum particle velocity of each component was determined from the delay resulting in the maximum vibration level (in/sec). The resultant particle velocity (or peak velocity of ground motion) was then determined from the following formula:

Resultant particle velocity $= v = a^2 + b^2 + c^2$

where: a = horizontal ('right-left') component

b = vertical component

c = radial ('to and from') component

The resultant particle velocity v was then plotted against 'scaled distance' on log-log paper. The formula for scaled distance is $D/W^{\frac{1}{2}}$ $(Ft/lb^{\frac{1}{2}})$ where D - Distance tunnel face to instrument, and W is weight of explosive charge.

The determination of D is straightforward; the determination of W is more complex and is calculated as follows:

In most instances the maximum level of ground vibration occurred where there was more than 1 drill hole loaded with explosive per half second delay. Although there were often upwards of 10 holes per delay it was generally possible to observe most of the holes in the delay as separate vibration peaks (i.e., instantaneous detonation did not occur). The total weight of explosive per delay was therefore divided by the number of discernible vibration peaks to obtain the actual weight of explosive responsible for the maximum vibration. The corrected weight of explosive was substituted for W in the scaled distance formula $D/W^{\frac{1}{2}}$.

A plot of peak ground motion versus scaled distance from the tunnel face appears as Figure 11. Figure 12 is compiled from results of vibration measurements obtained from detonating charges down exploratory diamond-drill holes. Figure 11 shows results from Figure 12 for comparison.

The use of scaled distance is a rough means of vibration control where no measuring is available; it should be used with caution as the criterion for damage is v, not scaled distance.

Amplitude: Amplitudes for the maximum vibration recorded for each test were determined from the following formula:

FIGURE 11

PLOT OF PEAK GROUND MOTION VELOCITY VERSUS SCALED DISTANCE ($D/W^{1/2}$)

+ — Plots taken from tunnel blasting.

D — Distance, instrument to tunnel face in feet

W — Weight of explosive (AN 60) creating maximum v

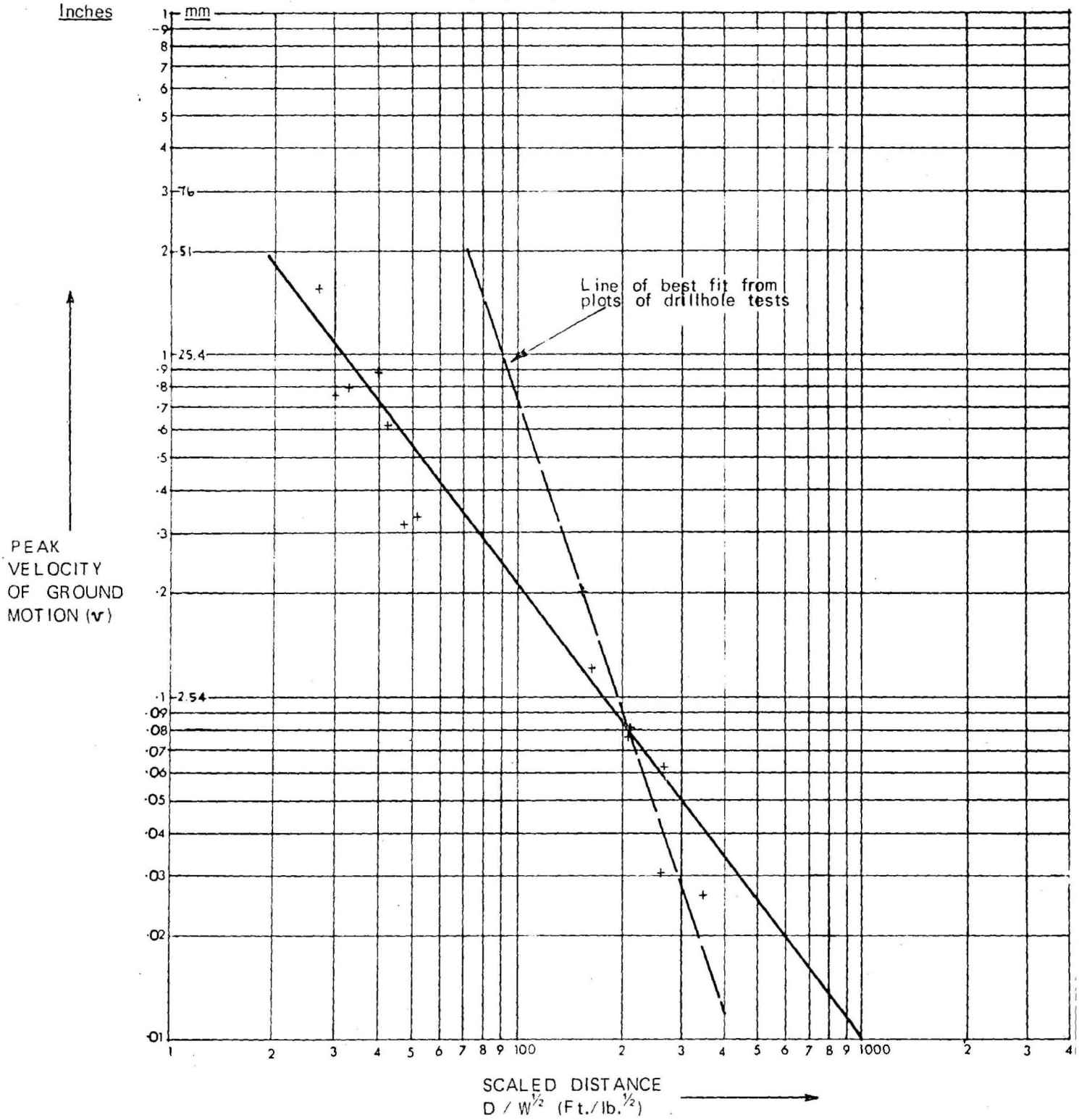


FIGURE 12

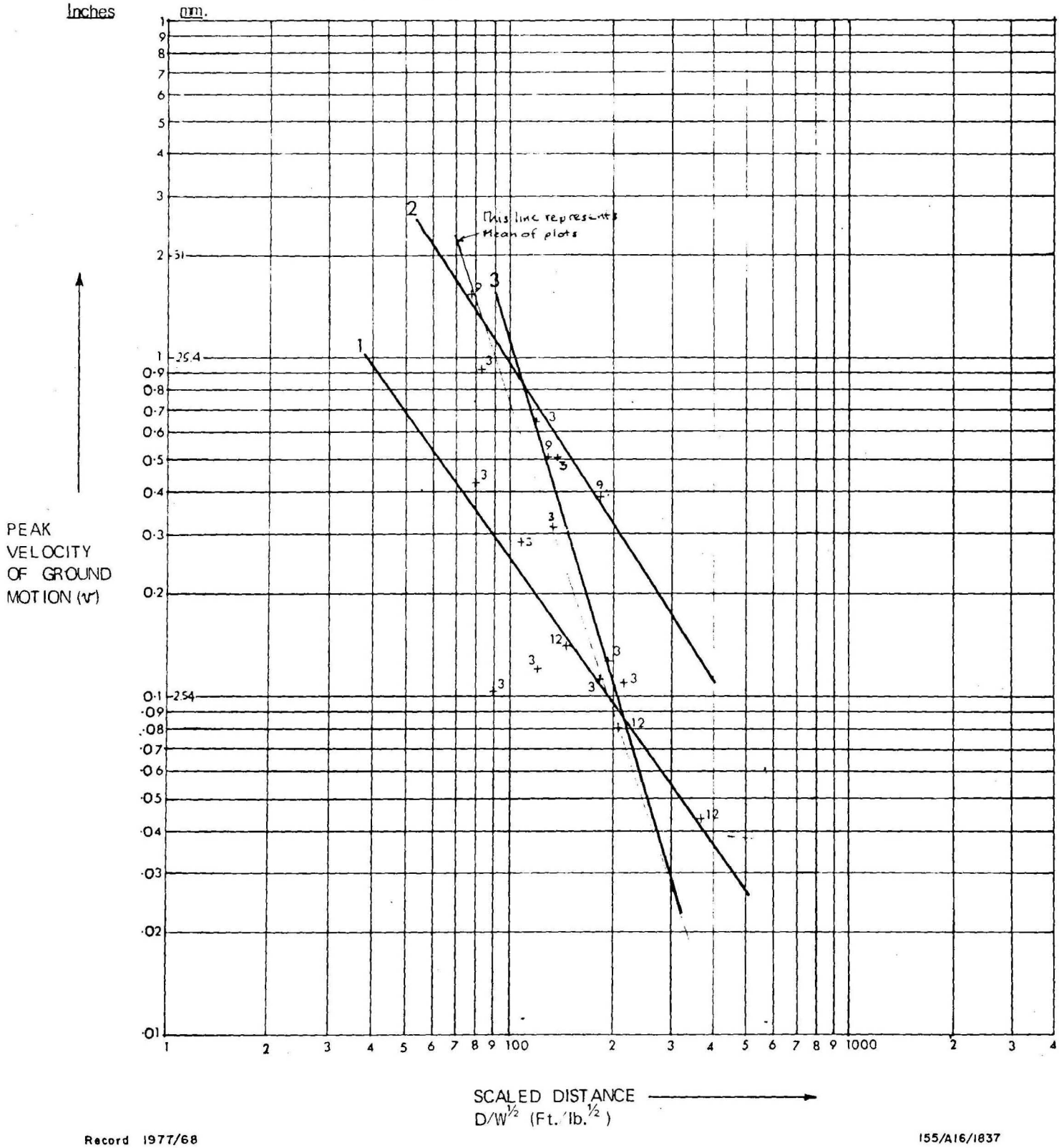
PLOT OF PEAK GROUND MOTION VELOCITY VERSUS SCALED DISTANCE ($D/W^{1/2}$)

Note:
These plots were calculated from results of experimental blasts in exploratory diamond drill-holes along tunnel line, Oct. 1971. Drillhole numbers are shown next to plots.

3 Line for TS 3 is based on first result for any one depth

2 Line for TS 9

1 Line for TS 12



Record 1977/68

155/A16/1837

FIGURE 13

VARIATION OF AMPLITUDE WITH DISTANCE

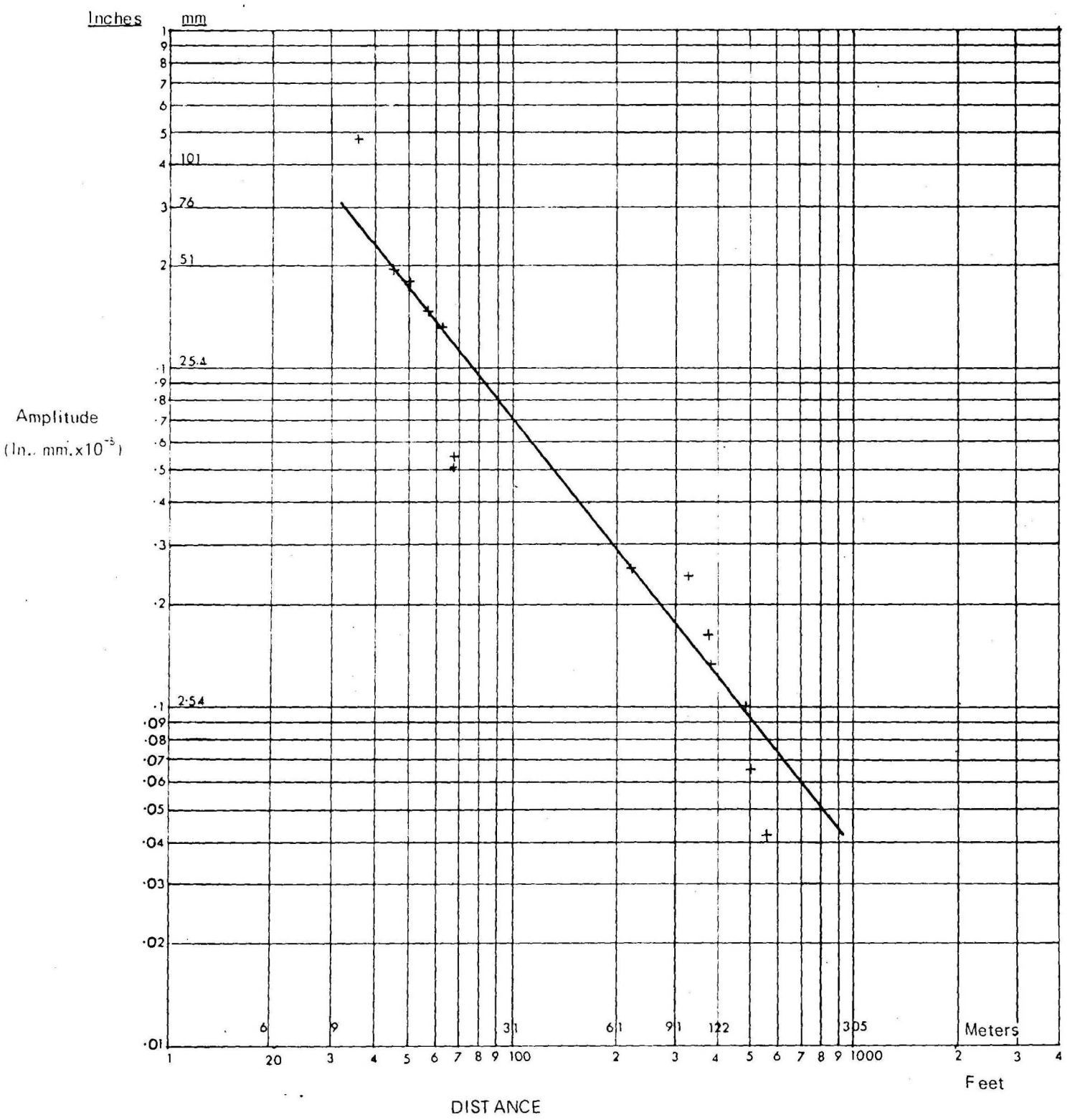
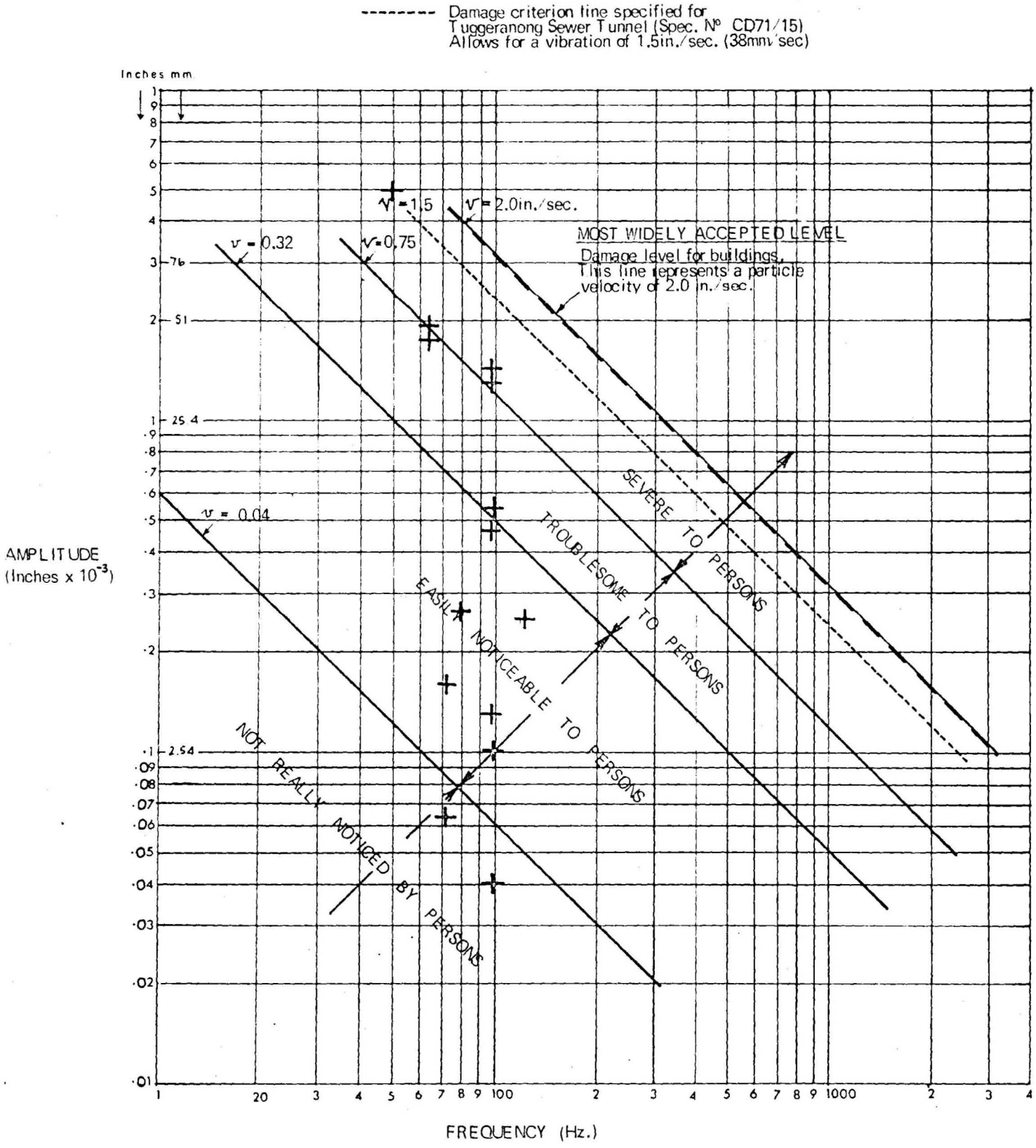


FIGURE 14

LEVELS OF HUMAN RESPONSE TO VIBRATION AND DAMAGE LEVELS VERSUS RESULTS FROM TUNNEL BLASTING.

(After Davies, 1972)



$$A = v/2\pi f$$

where: A = Amplitude half peak to half peak (in x 10^{-3})

v = particle velocity (in/sec)

f = Frequency (Hz)

Amplitude has been plotted against distance from shot in Figure 13, and peak particle velocity in Figure 14.

Site constant (K): Having determined amplitude, site constants for each test were computed. The constant K mostly depends on geological conditions; if the ground is unconsolidated alluvium K will have a high value, and if the ground is solid dacite K will have a lower value. Although other factors, such as type of explosives used will also affect the value of K, the simplistic formulae given below are thought to be adequate for practical purposes (Davies, 1970).

$$A = (K\sqrt{W})/d \quad \text{(for depth of cover of} > 70 \text{ feet)}$$

of $$A = \frac{70K\sqrt{W}}{d^2} \quad \text{(for depth of cover of} < 70 \text{ feet)}$$

Where distance from tunnel face to instrument is greater than 70 feet the relation between A and W and d is found to be in agreement. Where the distance is less than 70 feet, amplitudes are greater than those determined by theory due to wave interference (Davies, 1970).

Acceleration: Acceleration was considered by some to be a useful guide in determining the likelihood of damage but was not specified in this contract; structural damage has been found to occur when the peak acceleration equalled or exceeded 1g ($g = 32.2$ ft/sec^2 = acceleration due to gravity). Accelerations between 0.1g and 1g are generally classified as 'caution', and values lower than 0.1g as "safe" (Thoenen & Windes, 1942).

Accelerations resulting from tunnel blasting can be calculated, given values of frequency and particle velocity or amplitude from the following formulae respectively:

$$a = v\,2\pi f \quad \text{or} \quad a = 4\pi^2 f^2 A$$

where a = acceleration (in/sec^2)
f = frequency (Hz)
A = amplitude (in)
V = particle velocity (in/sec)

As the particle velocity criteria was the one used and specified for this project, accelerations were not calculated. However, it should be pointed out that the limits expressed in the project specification ($v < 1.5$ in/sec and $A < 0.024$ in for $f < 10$ Hz) are equivalent to an acceleration value of 0.25 g. The greatest particle velocity (1.56 in/sec) and lowest frequency (50 Hz) recorded in this project (stn 291 + 39, Table 4) resulted in an acceleration of 1.3 g (instrument 10 m above face in alluvium).

3.6.4. Discussion of results

Delay producing maximum vibrations. The vibration traces show that the maximum particle velocity for a particular round was not necessarily from the first delay (Table 4). In fact, maximum v resulted from the first delay on only two occasions. On 8 occasions delays 9 or 10 were responsible for maximum vibration. When delays other than the first are responsible for the maximum v, it is generally assumed that the delay hole(s) responsible are located above tunnel springline in relatively unbroken rock. For example, it is inferred that the hole(s) located on the periphery of the face near the tunnel crown ('ringers') were responsible for the maximum v recording from the last delay (no 10), (i.e. not the 'lifters').

Significance of results. Duvall & Fogelson (1962) were able to show that damage to a structure is most closely related to particle velocity which is a function of the strain or distortion in the rock mass (radial strain, $Er = v/c$, where c is the compressional seismic velocity). Figure 14 shows where the results from the 14 tests plot in relation to damage criterion and human response. Only one result plots near the damage level for buildings. Fifty percent of plots fall in the category of 'troublesome to severe' to persons; apart from one complaint, from an occupied house at 227 Badimarra Street, Fisher, the tunnel was sufficiently isolated either by surface (horizontal) distance or cover (vertical) as all other vibrations apparently passed unnoticed.

Measurements under Streeton Drive bridge. Most measurements were carried out with the instrument resting directly on ground surface. A series of four measurements was carried out on Streeton Drive bridge (crossing Weston Creek); two of the measurements were carried out on bridge piers and two on the stormwater stone-paving. Both measurements on the piers (stns 261 + 50 and 261 + 10) gave significantly lower values for V and A than those obtained on the stone-paving although values for f were the same (stns 262 + 15 and 261 + 80).

Results and geological environment. Frequencies and amplitudes of vibration generally correlated well with bedrock quality, type of overburden and relative thicknesses of each. Frequency values as low as 50 Hz and an A value of 4.98×10^{-3} inches was obtained from the measurement taken on 2/11/72 (stn 291 + 39) in extremely weathered bedrock underlying 4 m of saturated alluvial overburden. Frequencies as high as 125 Hz and A values as low as 4×10^{-5} inches were recorded where vibrations travelled through fresh bedrock with a thin weathered profile and soil cover. Frequencies and amplitudes in the intermediate ranges of 65 to 100 Hz and 1.0×10^{-4} to 1.9×10^{-3} inches respectively occurred elsewhere.

Significance of site constant K. The values of K obtained can generally be correlated fairly closely with geological conditions (for a similar weight of explosive). A relatively high value for K of 67 was obtained on alluvium (test 2/11/72) and K values between 46 and 49 on paving resting on alluvium under Streeton Drive/Weston Creek bridge. Much lower K values were calculated elsewhere, the lowest being 14, deep under the suburb of Fisher (16/1/74). Calculated K values for the tests carried out on the Streeton Drive bridge piers were about half the values calculated on the nearby paving; in other words the paving was significantly more affected by vibration than were the piers (as would be expected as the piers were founded on sound rock).

The mean value of K for all the tests is 33; the average value for tests carried out in weathered rock and/or alluvium is 47, and 27 for tests carried out in essentially fresh rock at depths greater than 30 m and with a variably weathered upper profile. (K values obtained from the bridge piers were not included in these averages).

TABLE 4. SUMMARY OF TUNNEL BLASTING VIBRATIONS*

TUNNEL STATION	DELAY CAUSING MAXIMUM VIBRATION	WEIGHT OF AN 60 IN DELAY		WEIGHT OF * AN 60 CAUSING MAXIMUM VIBRATION PEAK		DISTANCE INSTRUMENT TO TUNNEL FACE		SCALED DISTANCE (D/ W)		RESULTANT VELOCITY $a^2 + b^2 + c^2$		FREQUENCY OF MAX VIBRATION (Hz)	AMPLITUDE OF MAX. VIBRATION ($A = v/2\pi f$)		SITE CONSTANT K (Imperial unit)
		lbs	kg	lbs	kgs	ft	m	ft.lb	m/kg	in/sec	mm/sec		in $x10^{-3}$	$mmx10^{-3}$	
195 + 58	8	22	10	2	0.9	220	67	157	70	0.20	5.08	125	0.25	6.3	39
197 + 36	9	25.6	11.6	3.2	1.4	380	116	214	97	0.08	2.03	100	0.13	3.3	28
198 + 54	9	25.6	11.6	3.2	1.4	480	146	270	122	0.06	1.52	100	0.10	2.5	27
198 + 83	10	13.3	6	3.3	1.5	500	152	268	124	0.03	0.76	75	0.064	1.6	18
199 + 18	9	25.6	11.6	2.5	1.1	550	168	355	85	0.026	0.66	100	0.04	1.0	14
261 + 10	10	24	10.8	2.0	0.9	67	20.4	48	21	0.30	7.62	100	0.48	12.2	22
261 + 50	1	3.3	1.5	1.7	0.8	67	20.4	51	23	0.33	8.38	100	0.53	13.5	26
261 + 80	9	6	2.7	2.0	0.9	61	18.6	43	20	0.81	20.57	100	1.30	33.0	49
262 + 15	10	18	8.2	2.0	0.9	56	17.1	40	18	0.90	22.86	100	1.44	36.6	46
267 + 23	10	32	14.5	2.7	1.2	50	15.2	30.5	14	0.73	18.54	65	1.79	45.5	39
269 + 35	8	23.3	10.5	2.9	1.3	45	13.7	33	12	0.79	20.07	65	1.94	49.4	33
291 + 39	4	15	6.8	1.7	0.8	35	10.7	27	12	1.56	39.62	50	4.98	126.5	67
13 + 51	6	12	5.8	3	1.4	370	113	214	95	0.075	1.90	75	0.16	4.1	20
4 + 60	1	7.3	3.3	3.7	1.7	325	99	170	76	0.12	3.05	80	0.24	6.1	40

* Wherever tests were conducted, detailed tunnel sections, together with blasting patterns and vibration records have been placed in the project box.

** Total weight of AN 60 in delay divided by number of peaks.

TABLE 5. SUMMARY OF DRILL-HOLE BLASTING VIBRATIONS

TEST DATE	DRILLHOLE NUMBER	WEIGHT OF AN 60		DISTANCE CHARGE TO INSTRUMENT		SCALED DISTANCE (D/ W)		RESULTANT VELOCITY $a^2 + b^2 + c^2$		FREQUENCY (Hz)	AMPLITUDE ($A = V/2\pi f$)		SITE CONSTANT
		lb	kg	ft	*	ft/lb	m/kg	in/sec	mm/sec		in x 10^{-3}	mm x 10^{-3}	K
28/9/71	TS 3	1	0.5	126	52'	126	55	0.12	3.1	100	0.19	4.8	24
"	"	2	0.9	126	52	90	40	0.10	2.5	75	0.21	5.3	19
"	"	0.33	0.14	126	52	220	104	0.11	2.8	75	0.23	5.8	51
5/10/71	"	0.33	0.14	69	48	120	57	0.65	16.5	75	1.38	35.1	168
"	"	0.66	0.3	69	48	85	38	0.90	22.9	35	4.10	104.0	350
"	"	0.66	0.3	111	48	137	62	0.31	7.9	50	0.99	25.2	140
"	"	1	0.5	111	48	111	48	0.29	7.4	50	0.92	23.4	100
"	"	2	0.9	111	48	78	36	0.42	10.7	35	1.90	48.4	150
6/10/71	"	0.66	0.3	157	45	194	87	0.13	3.3	30	0.69	17.6	136
"	"	0.66	0.3	110	45	136	61	0.50	12.7	40	2.00	50.8	275
"	TS 8	0.66	0.3	150	20	186	83	0.11	2.8	40	0.44	11.2	82
"	TS 9	0.66	0.3	65	41	80	36	1.59	40.4	30	8.40	214	640
"	"	0.66	0.3	108	41	134	60	0.50	12.7	35	2.28	58	310
"	"	0.66	0.3	156	41	192	86	0.38	9.6	50	1.21	30.7	235
"	TS 12	0.33	0.14	210	180?	370	174	0.042	1.1	125	0.05	1.3	18
"	"	1	0.5	210	180?	210	92	0.082	2.1	100	0.13	33.0	27
"	"	2	0.9	210	180?	148	68	0.14	3.6	120	0.19	48.4	28

* Depth of charge in drill hole (below ground surface) in feet.

Drill hole stations are: TS3 (100 + 85), TS8 (271 ? 21), TS9 (276 + 50) and TS12 (approx 219 + 50)

Conditions at TS3 were: 0-40' soil and colluvium, 40'-85' completely weathered dacite, fresh below 85'

Conditions at TS8 were: 0-12' soil and alluvium, 12-22 completely weathered rhyodacite, fresh below 22'

Conditions at TS9 were: 0-15 saturated alluvium, 15- 2' " " " , slightly weathered below 52'

Conditions at TS12 were: 0-4' soil, 4'-70' completely weathered rhyodacite, 70'-251' fresh rhyodacite

Results of vibration measurements in drill holes. The purpose of these tests was to obtain vibration data from different geological environments before tunnelling commenced. Results of these tests have been plotted in Figure 12. The scatter of plots may be a result of several explosions in the one drill hole at the same (or nearly) depth; the first explosion created a cavity of broken rock which therefore affected subsequent vibrations from the same place.

Tests carried out in drill hole TS 12 (under Stirling), which has a higher-quality rock and thin soil cover, resulted in values of v, A, and K being significantly lower than tests in the other drill holes. Velocity, A, and K values obtained from drill holes TS 3, 8 and 9 were higher owing to the presence of a relatively thick layer of slopewash and alluvium (saturated in holes TS 8 and 9). The further away the instrument the greater the thickness of alluvium the ground vibration had to travel through before being recorded.

Plots of results for initial explosions in each drill hole have been plotted in Figure 12. Lines of best fit for each hole are included. Subsequent explosions in the same hole from the same depth were not considered when drawing the line of best fit.

The first test in drill hole TS 9 recorded the highest values for v (1.59 in/sec) and A (8.4×10^{-3} in) and the lowest value for f (30 Hz) of any test; subsequent tests in the same hole from the same depth recorded increasingly smaller values of v and A.

3.7 SHAFT GEOLOGY

3.7.1. General

Figures 15 to 19 summarise the engineering geology of the 5 shafts; Figure 20 compares the excavation rates.

The shafts were excavated under subcontract in the following order: shaft 1, 2, 5, 4, and 3 (some overlap in construction periods occurred). All were concrete lined and the inspection shaft at stn 282 + 00 was backfilled with tunnel spoil, and grouted near shaft-tunnel junction. Excavation was by

drill-blast methods and mucked by hand. Shafts 4 and 5 were excavated to full diameter from surface to invert level; the remaining shafts were excavated to full diameter in incompetent rock and then as 1.8 m x 1.2 m pilot shafts in sound rock to tunnel invert. The pilot shafts were stripped after tunnel breakthrough to full shaft diameter. All shafts were sunk (surface to invert) except shaft 3. Shaft 3 was sunk at full diameter from surface to RL 611 m (36 m) and then a 2 m x 1 m pilot shaft was raised (using an 8" diameter hole for raise and lower of rise cage) from tunnel invert (RL 545 m) to RL 611 m. The pilot shaft was subsequently stripped from top to bottom.

Incompetent rock was supported by 6" x 5" x 25 lb RSJ two-segment rings, close-lagged with 6" x 2" hardwood. Steel was spaced at 4' centres, hung off ¾" tie rods and pulled tight to 3" x 3" hardwood spacers. A few rock bolts were used to hold blocky ground.

3.7.2. Excavation conditions

General. Shafts 1, 2, and 3 were excavated in blue-grey dacite and shafts 4 and 5 in purple rhyodacite.

Shaft 1. Excavation conditions in shaft 1 were generally very good and only 10 steel rings were erected. Vertical shears and clay seams occurred on the northeast wall to midway down and resulted in some overbreak; some rock bolts were inserted. Water inflows were insignificant. No excavation delays occurred through poor ground conditions.

Shaft 2. Poor rock conditions and large groundwater inflows caused excavation delays; many man hours were lost in pumping water from the shaft (see 3.8.4). Twenty-seven steel rings (with timber lagging) were erected; in places large groundwater inflows constantly washed loose rock from behind timber lagging, causing additional overbreak and necessitating immediate retimbering to avert a potentially very dangerous situation from developing further.

Shaft 3. This shaft is located midway along the tunnel on a topographical high 102 m above tunnel invert. The dacite is deeply weathered and seamy and blocky to about RL 600 m (47 m below surface). Forty-one steel rings were erected and a 6-m section of mesh and rock bolts was inserted to

stabilise blocky, seamy, and sheared rock. About 60 percent of the shaft was therefore supported. Raising the pilot shaft was significantly faster than sinking the other pilot shafts. Groundwater inflows were very small and did not affect excavation conditions.

Shaft 4. Rock conditions in shaft 4 were generally blocky and seamy; the shaft was supported from just below top structure to about 5 m above tunnel-shaft junction. Groundwater inflows were small and did not reduce excavation rate although small inflows and seepages probably reduced the strength of the already weak rock mass.

Shaft 5. Shaft conditions were poor. This was due to: (i) the presence of a fractured and sheared rhyolite dyke following the shaft down to tunnel invert; (ii) 'nuisance' groundwater inflows and; (iii) the shallow depth of the shaft, which was excavated mainly in the top part of the weathered profile. The entire shaft below top structure required steep support, including the tunnel-shaft junction.

3.7.3. Shaft transitions

All transitions (shaft-tunnel junctions) except for shaft 5 were in competent rock with only the odd rock bolt required. Shaft 5 transition was poor owing to the sheared dyke mentioned above, and steel support was required. When the transition was stripped out, special transition supports were substituted for the temporary tunnel sets. There was very little overbreak at any of the transitions.

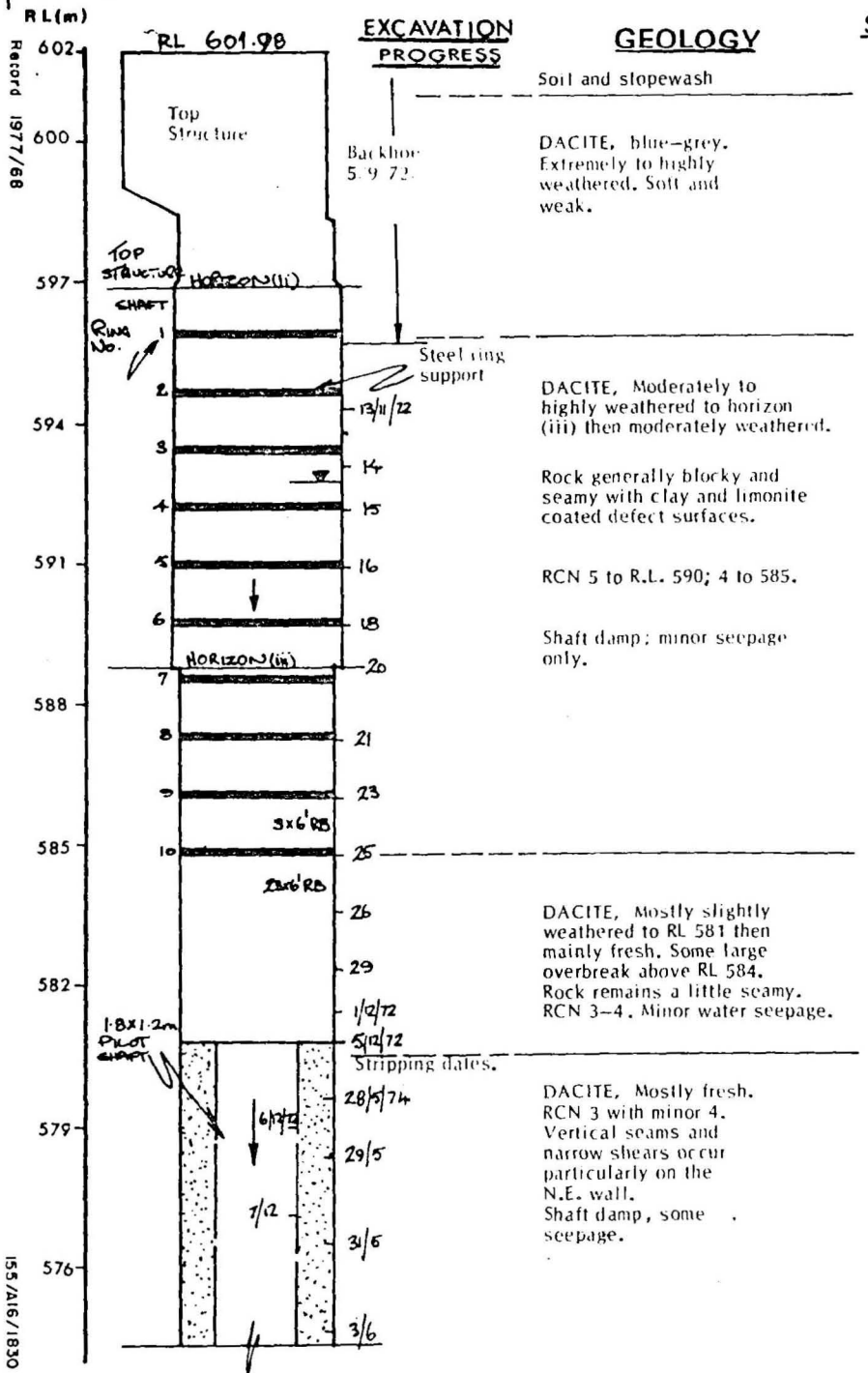
3.7.4. Concrete

A slip form method was used (see Fokkema & Noack, 1976). Before the concrete was poured, rock surfaces were washed down, and accumulated rock and debris were removed. Panning areas of water inflows was necessary, particularly in shaft 2, which required about 30 m of full panning. The timber lagging could not be removed before concreting, often for safety reasons, and as a result all supported sections were backfill-grouted, starting from the lower levels and proceeding upwards.

Concreting and backfill grout of the shafts was successful. Some groundwater did seep through joints in the concrete, particularly in shaft 1.

SHAFT 1 GEOLOGY (STN 52 + 50)

Scale V/H = 1



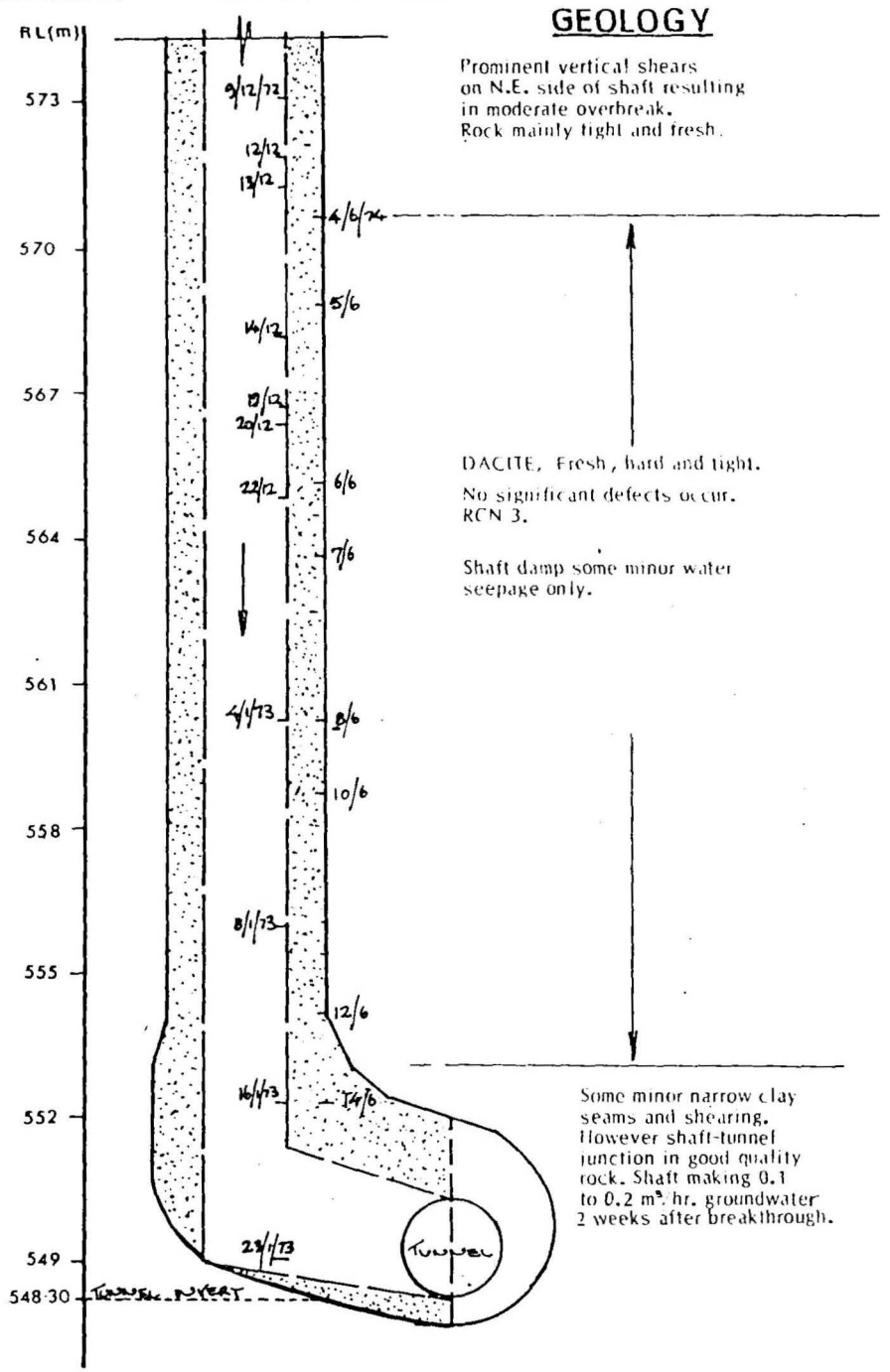


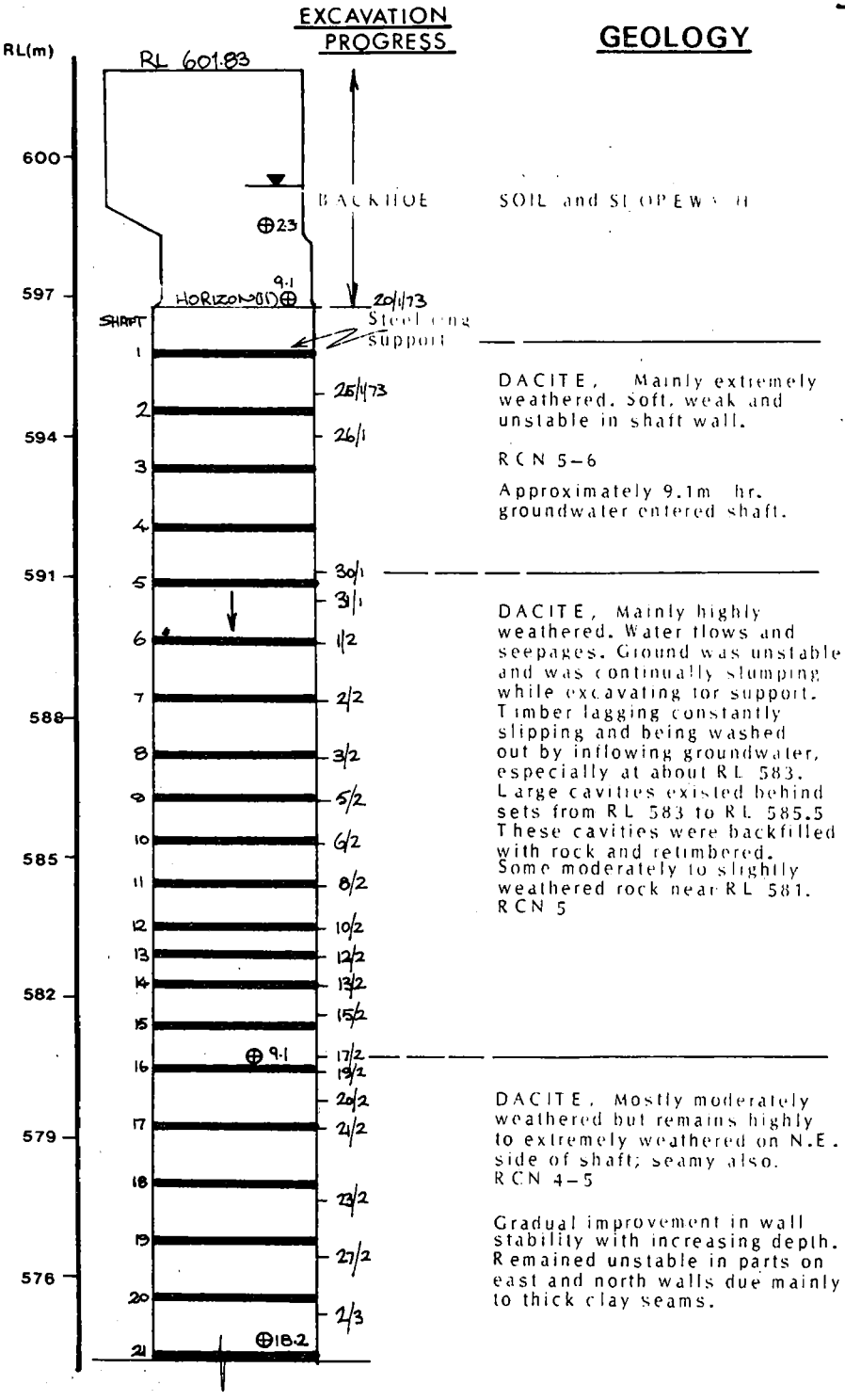
FIGURE 15

155/A16/1830

SHAFT 2 GEOLOGY (STN 94 + 00)

Scale V/H = 1

⊕ 2 Water inflow in m³/hr from surface to this point.



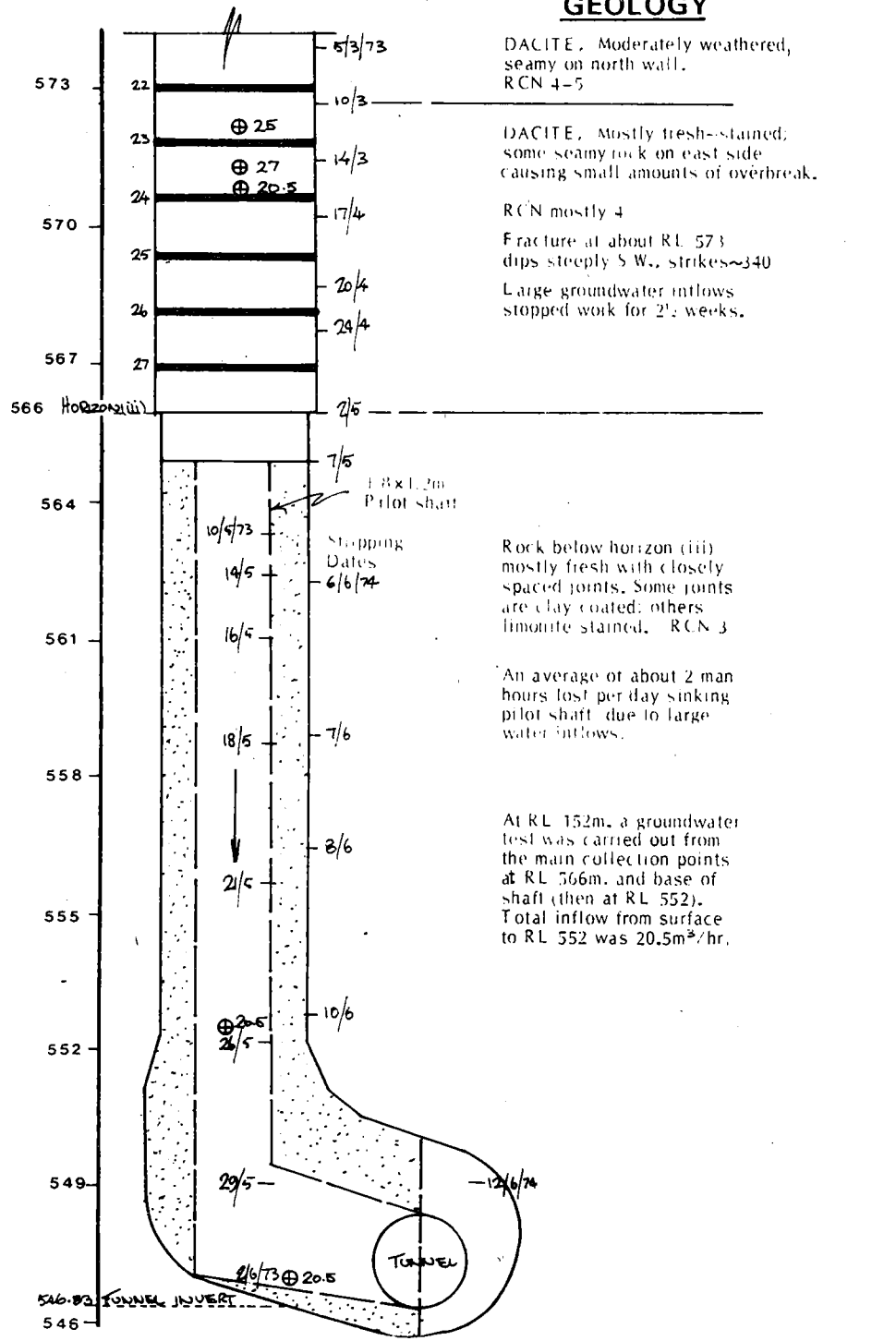
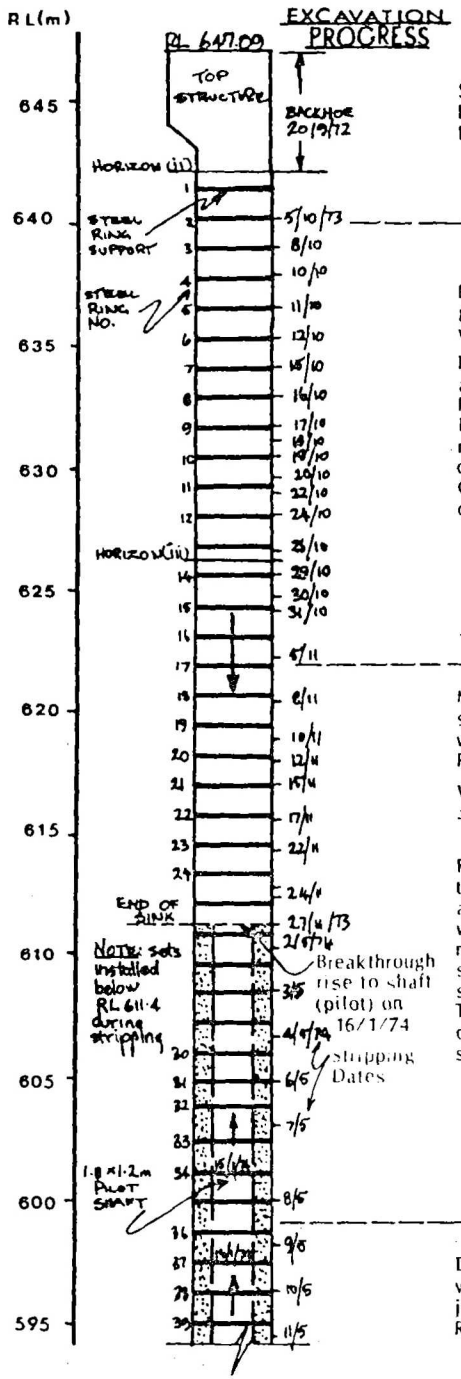


FIGURE 16

SHAFT 3 GEOLOGY (STN 145+00)

Scale V/H = 1



GEOLOGY

SOIL SLOPEWASH AND EXTREMELY WEATHERED DACITE

DACITE, highly weathered grading to mostly moderately weathered.

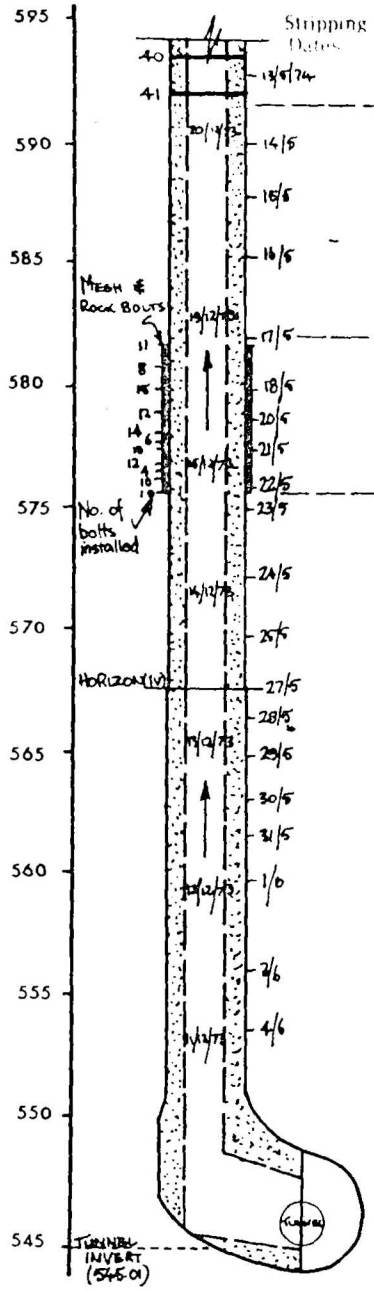
Rock generally very blocky and seamy; rock mass weak; RCN 5. Some shearing. Full lagging behind steel rings required. Mostly dry; damp in places. Gradual improvement in rock quality towards RL 622.

Mainly moderately weathered; some slightly and highly weathered. RCN mainly 4 (5 in places)

Very seamy at RL 617; clay seams 10cms. wide.

From about RL 601 to RL 609 the dacite is very blocky and seamy and highly weathered in places. Rock mass particularly weak on south side of shaft; horizontal clay seams particularly prominent. Thick seams and sheared rock caused moderate overbreak on south wall at about RL 601.

Dacite mainly slightly weathered to fresh stained; joints moderately tight RCN 3



GEOLOGY

45cm. wide clay seam at RL 593; area timbered. Horizontal (?) fractured zone at RL 591.6

Dacite fresh, hard and mostly tight. RCN 3
Minor seepages only.
Near vertical sheared zone occurs on south wall from RL 582 to 583 but stable because it is wedged in with sound rock

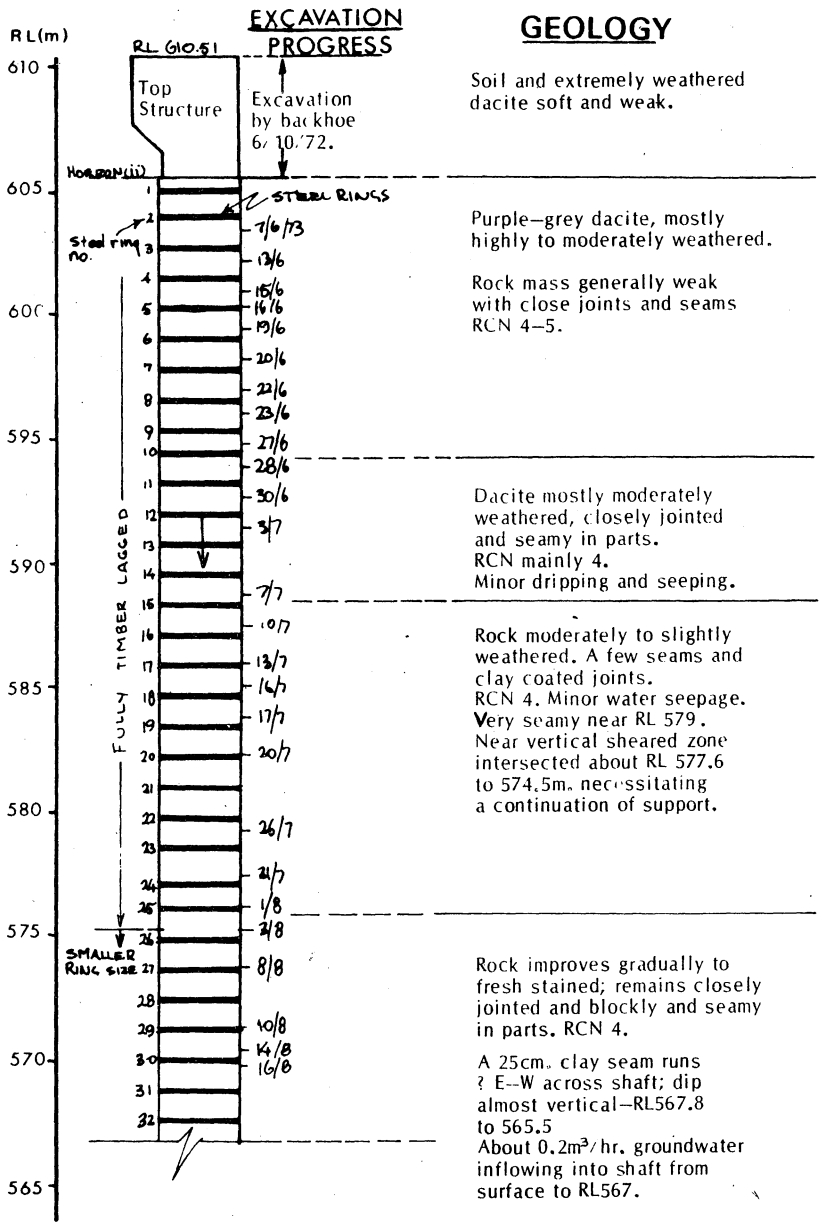
Rock material hard but mass weakened by clay seams and thin shear zones dipping at about 45° into shaft. Mesh and bolts required to hold rock in place.

Dacite fresh, hard with close but tight joints. RCN 3
Minor water seepage.
No major defects.

Shaft–tunnel junction in competent rock.

FIGURE 17

SHAFT 4 GEOLOGY (STN 202 + 90)



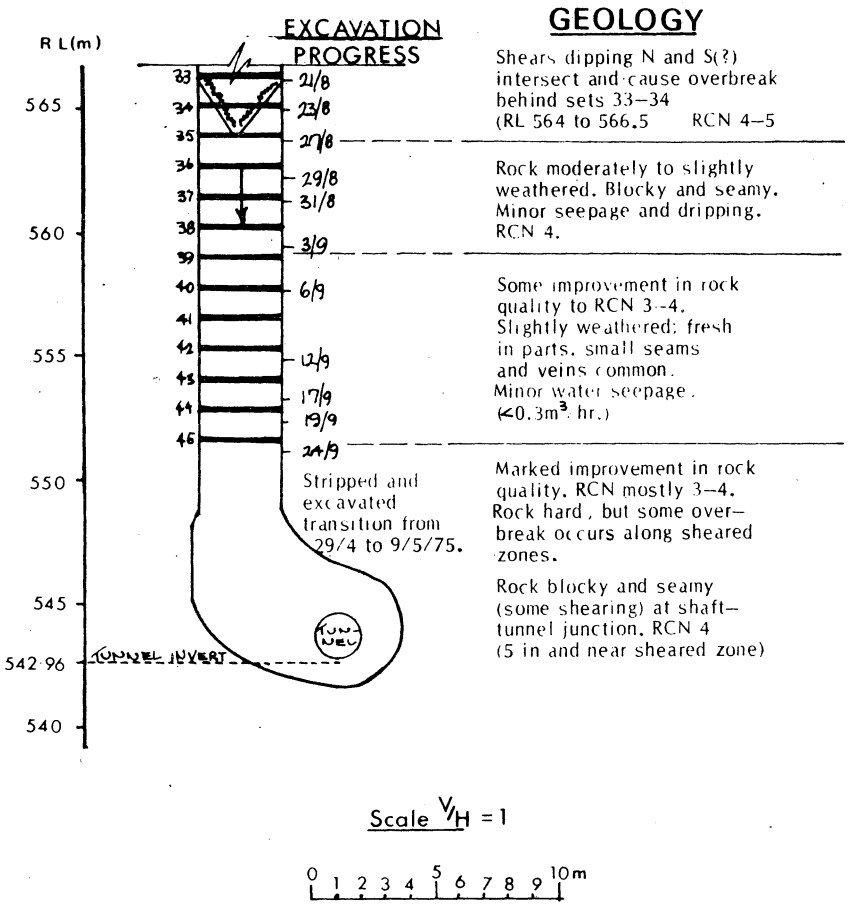
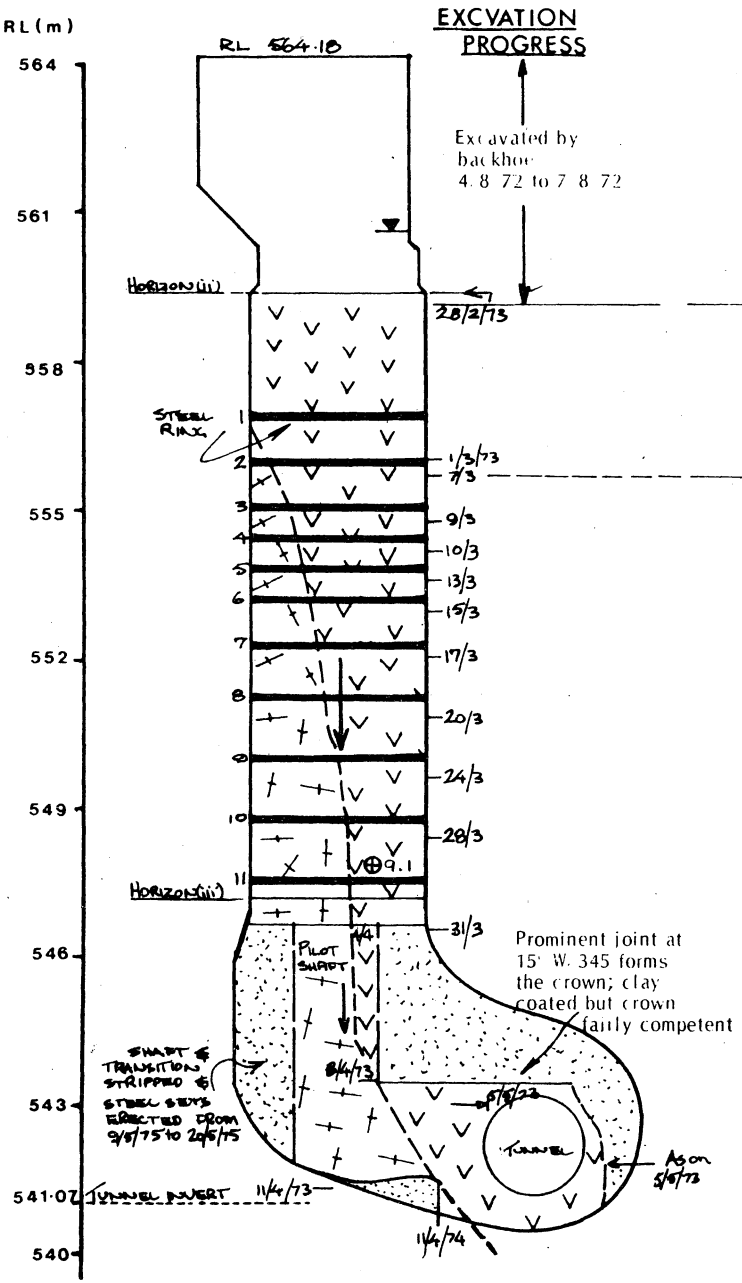


FIGURE 18

SHAFT 5 GEOLOGY (STN 255 + 90)



GEOLOGY

Alluvium overlying sandy–clay grey soil.

Groundwater intersected at RL 560.5 (3.5m below ground surface)

Highly to completely weathered purple dacite closely jointed, seamy RCN 5–6. Groundwater seepage minor.

Rock conditions variable; moderately weathered and blocky S.W. wall of shaft (RCN 4) but fractured, sheared and broken rhyolite dyke N.E. section of shaft (RCN mainly 5)

The broken rhyolite dyke is soft to moderately hard, seamy with clay coated greasy fracture plane surfaces. Rock mass very weak. It is green–grey and fine grained and frets away on exposure. The dyke has an attitude 90/115 and was supported in the tunnel.

Tunnel–shaft transition was steel supported–rock quality poor (RCN 4–5).

On 8/5/73 tunnel was 15m. past shaft; water pumped from base of shaft totalled 13.5m³/hr.

Scale V/H = 1

0 1 2 3m

⊕ 9.1 Water inflow into shaft from surface to this point (m³/hr)

Rhyolite dyke

Purple rhyodacite

FIGURE 19

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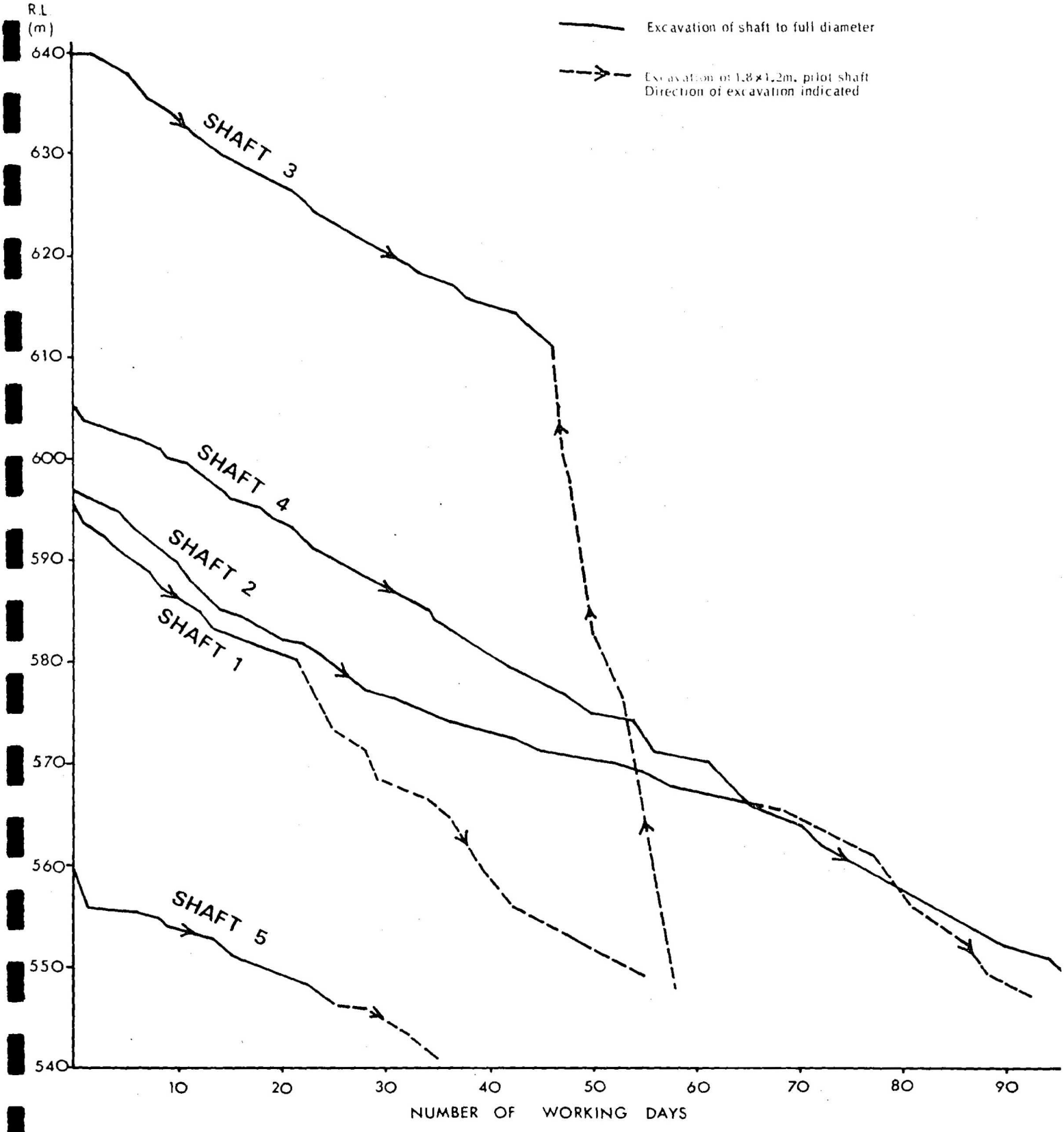
SHAFT EXCAVATION RATES

FIGURE 20



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155/A16/1835

Shaft 4 was surprisingly good (some dampness only) after concreting, considering the poor rock conditions encountered, and probably looked the pick of the shafts on completion.

3.8. TUNNELLING AND GROUNDWATER

3.8.1. General

DHC specification CD71/15 Clause 3.5.2(b) states that 'after a total progressive quantity of 200 million gallons (90×10^4 m^3) of water has been measured by the quantity measuring devices as having being pumped from the tunnel works, each further million gallons.....will be paid.....' Much of the water actually encountered during excavation was able to flow, without pumping to the outlet portal and therefore could not be considered as measured (payable) water.

Most significant groundwater inflows were confined to the north heading; the main exception was shaft 2, which made a fairly constant 18 to 20 m^3 per hour until lined with concrete.

Zones of initial high inflows generally dried up fairly rapidly once excavation had passed the point of inflow; there were exceptions, such as the contact zone between stns 178 + 00 and 179 + 00 where inflow decreased by only 50 percent; this zone also had the highest initial inflow of >50 m^3/hr along a 30 m section of tunnel. Figure 27 shows a total of about 780 000 m^3 flowed from the north heading. About 100 000 m^3 of groundwater entered the tunnel from shaft 2 between 18/6/73 (completion of shaft) and 23/4/74 (completion of tunnel excavation). Inflows into the south heading are estimated to be less than 75 000 m^3 (total outflow during construction), and are considered negligible compared with shaft 2 and north heading inflows. The total amount pumped from the south heading (via inlet portal and shaft 2) was therefore about 175 000 m^3. Total inflows into the tunnel until breakthrough (23/4/74) were 955 000 m^3 (estimated amount before construction was 910 000 m^3).

3.8.2. Water inflows - south heading

Apart from a few narrow open-jointed sections of tunnel through which small to moderate initial inflows occurred, the rock mass was very tight.

About 30 percent of this heading was dry; dripping or seepage occurred from most closely jointed and fractured zones. Clayey sheared zones were often dry or damp, and significant inflows (upwards of 200 1/h) were recorded from only a few. Most inflows recorded from this heading were less than 200 1/h and decreased fairly rapidly. The most significant inflows were recorded from:

- narrow fault at stn 52 + 20 (near shaft 1), 1600 1/h; reduced gradually over a few days to less than 100 1/h.
- open joint at stn 59 + 80 (west wall), 2700 1/h; very gradually reduced to about 250 1/h one month later
- sheared zone at stn 97 + 75, 2700 1/h; reduced to 450 1/h by the following shift and to dripping and seepage 5 days later.

Initially high inflows were also recorded when diamond-drill holes were intersected; for example, 1400 1/h from TS13 (reduced to 500 1/h after 7 days) and 6800 1/h from TS3 (reduced to 20 1/h after 10 days).

3.8.3. Water inflows - north heading

The north heading (including shafts 4 and 5 and the inspection shaft) accounted for about 80 percent of all groundwater drained by the tunnel. Individual inflows up to 1 m^3/h were numerous; most decreased to dripping or seepage. The wettest sections of the north heading were:

(i) Stns 178 + 00 to 179 + 20 - contact zone between purple rhyodacite and grey-blue dacite; water inflows through open joints were initially up to 45 to 50 m^3/h in this section, decreasing gradually to about 18 m^3/h.

(ii) Stns 180 + 00 to 184 + 00 - water inflows up to about 20 m^3/h occurred from within 5-10 m of the face in this section. These inflows decreased fairly rapidly to seepage and dripping zones; occasional small inflows of less than about 0.2 m^3/h persisted from some places.

(iii) Stns 226 + 00 to 227 + 40 - individual initial flows up to 14 m^3/h occurred over this section. These flows gradually reduced over a few days to generally less than 0.5 m^3/h. Most flows were associated with open sheared and fractured zones. Most sheared and fractured zones from stn

227 + 00 southwards to about stn 200 + 00 recorded moderate inflows (0.5 to 2.5 m^3/h) and reduced to generally less than 0.2 m^3/h.

(iv) Stns 250 + 00 to 255 + 00 - initial water inflows exceeding 20 m^3/h (from a 5 m length of tunnel) were recorded from open (1 to 3 cm) near horizontal joints in this section. These flows decreased rapidly to less than about 0.1 m^3/h.

Some sections of the rhyolite produced initial water inflows in excess of 3.0 m^3/h gradually reducing to less than 1.5 m^3/h.

Water inflow into the tunnel via drill hole TS7 was initially about 2.3 m^3/h, reducing to about 0.25 m^3/h (see plot, Fig. 22).

3.8.4. Water inflows - shafts

Shaft 1 - Groundwater inflows into this shaft were very small and did not affect excavation in any way. On completion of excavation the shaft was making less than about 0.2 m^3/h (see Fig. 15).

Shaft 2 - Groundwater inflows into this shaft were high enough to delay shaft excavation; many man-hours were lost due to excessive water. Figure 16 shows inflows measured at various points during excavation. Large water inflows occurred directly from the soil-slopewash horizon and through open fractures and joints, primarily above horizon (iii; RL 566). Most of the water apparently originated from the deep soil and slopewash aquifer covering most of the Kambah area. An observation bore (TS 17, Fig.26) located only 50 m downhill from the shaft recorded a drop in water-level of only 4.3 m (only 0.8 m lower than the observation bore first intersected water). On completion of the shaft, water inflows remained fairly constant at between 10 and 13.6 m^3/h.

Shaft 3 - Groundwater inflows into this shaft were very small and did not affect excavations in any way. During and after excavation the shaft never made more than about 0.5 m^3/h.

Shaft 4 - Groundwater inflows into this shaft were small, the total at any one time never exceeding 0.5 m^3/h.

Shaft 5 - Groundwater inflows into shaft 5 generally increased with depth, and on completion yielded a fairly constant 13.5 m^3/h; this decreased gradually to less than about 5 m^3/h. The shaft is situated in blocky and partly fractured open rock accompanying a rhyolite dyke (Fig. 19) which allowed easy and fairly rapid groundwater drainage into the shaft. The water-levels in observation bore TS16, and water inflows into the tunnel restricted to near invert level, indicate that the tunnel and shaft have lowered the groundwater-table effectively and rapidly (Fig. 25).

Water pumped from shafts 4 and 5 and the inspection shaft (stn 282 + 00) have been included in the cumulative groundwater plot (Fig. 27).

3.8.5. Groundwater effect on tunnelling and shaft excavation

The larger groundwater inflows were generally confined to sections of tunnel classified as RCN 4, 5, or 6. Progress was slowed by water inflows in some sections of the tunnel, but at no time did they result in a cessation of tunnelling operations, and at no time was it considered necessary to pilot drill ahead of the face. Most significant groundwater inflows occurred in the north heading, which was free-draining. On some occasions (especially over holiday periods) groundwater built up near the face of the south heading, and some time was lost while pumping was in progress.

Sinking of shaft 2 was affected by excessive water inflows as noted previously. The exact number of man-hours lost is not known.

A small amount of time was lost through pumping groundwater from shaft 5 during excavation. Figures on lost man-hours are not available.

Groundwater certainly decreased rock stability in RCN 5 and 6 and therefore indirectly slowed excavation progress. Overbreak in RCN 5 and 6 was generally greater where groundwater inflows occurred.

Although groundwater inflows in the rhyolite sections were not large, enough water was present to reduce rock stability considerably. Once the groundwater-table was lowered to near invert level some sections of rhyolite appeared much stronger, and able to stand up without support.

OBSERVATION BORE TS 5

LOCATION: N 595990 E 203883
On tunnel centre-line stn. 201+00
Approx 60 m from shaft 4

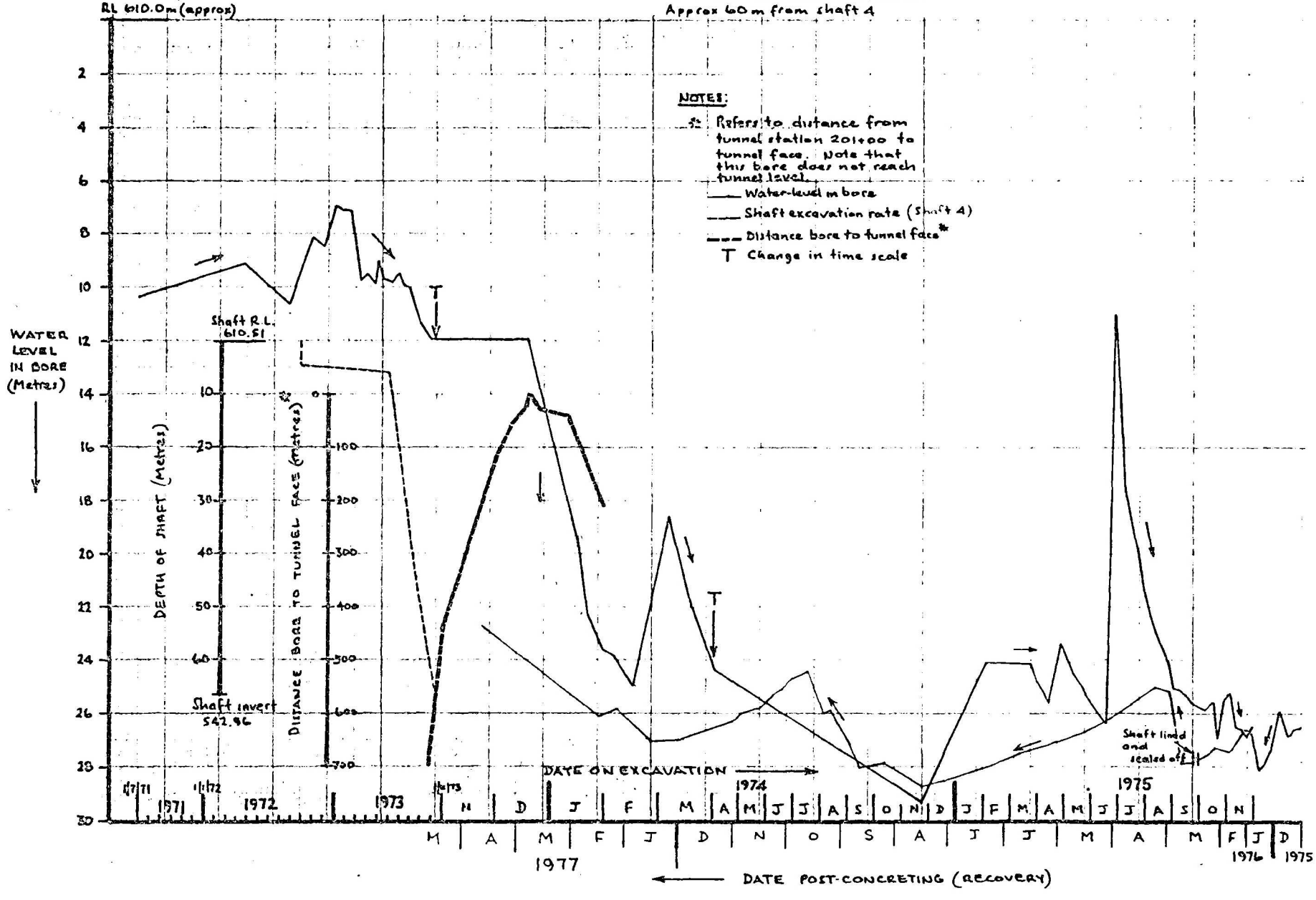


FIGURE 21

OBSERVATION BORE TS 7

LOCATION: N 597781
E 203806
TUNNEL STATION 259+80

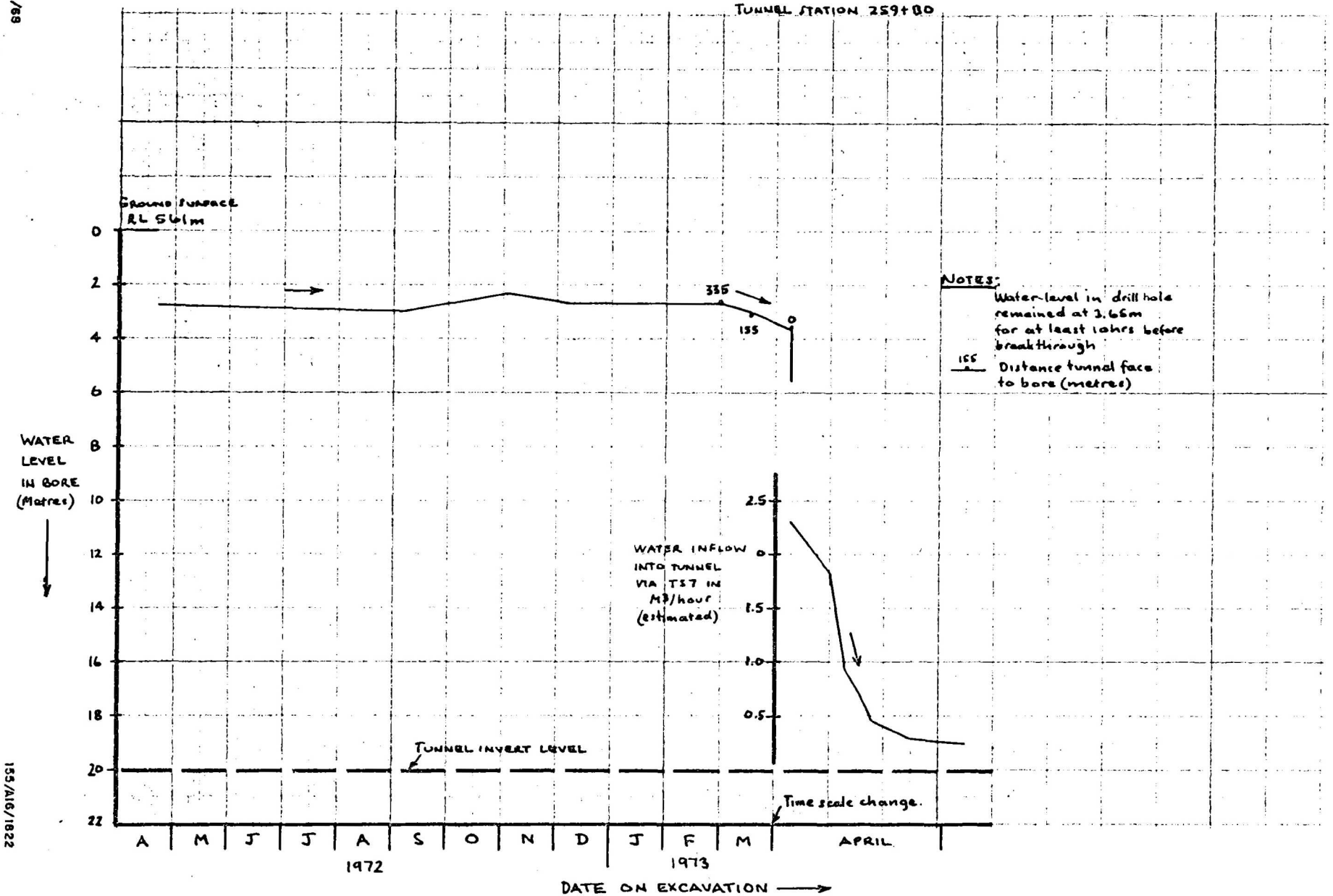


FIGURE 22

OBSERVATION BORE TS 14

LOCATION: N 596992 E 203771
69m west of tunnel station 233+97

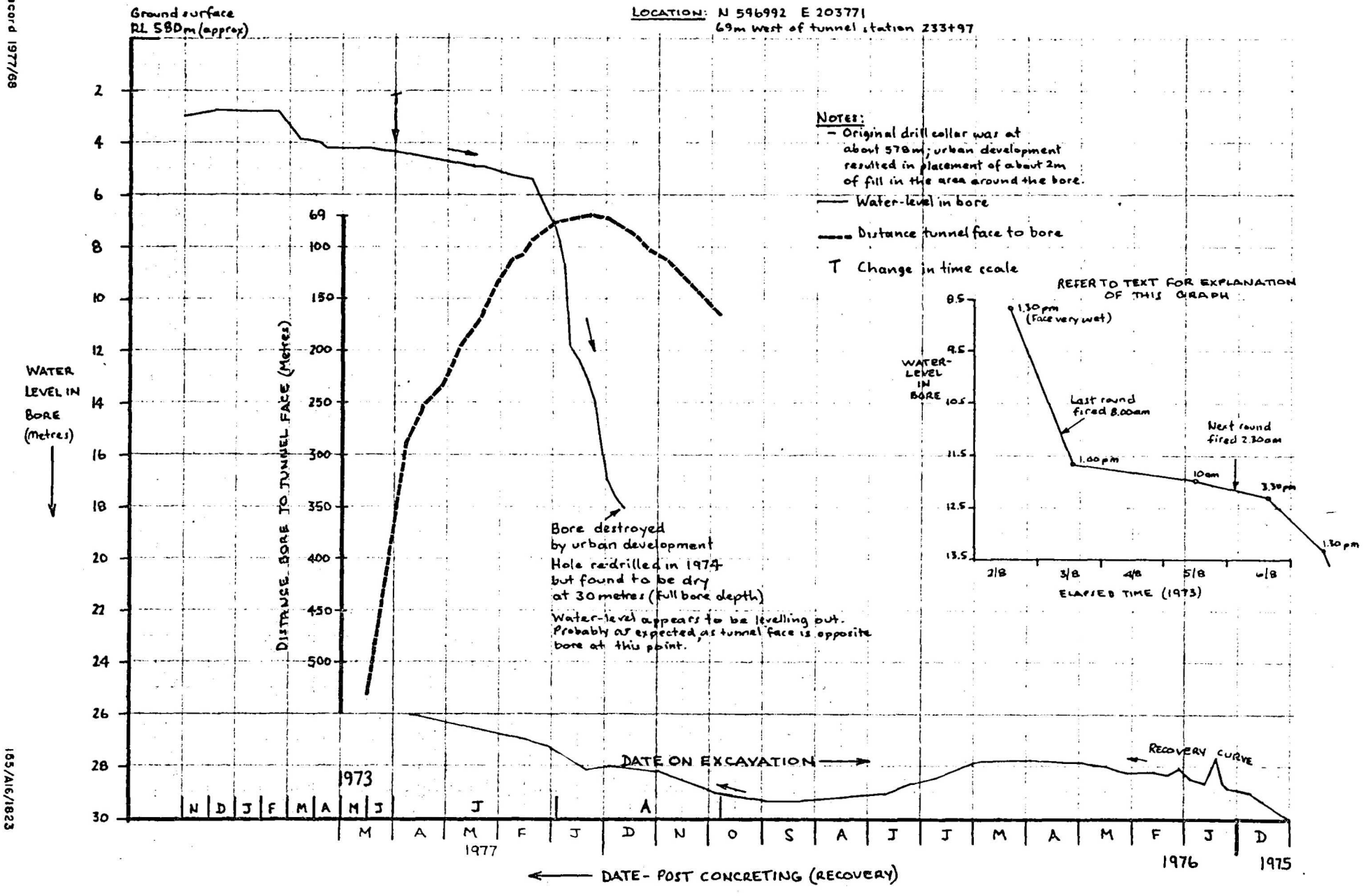


FIGURE 23

OBSERVATION BORE TS 15

LOCATION: N 594401 E 203981
33m E of tunnel station 148+80

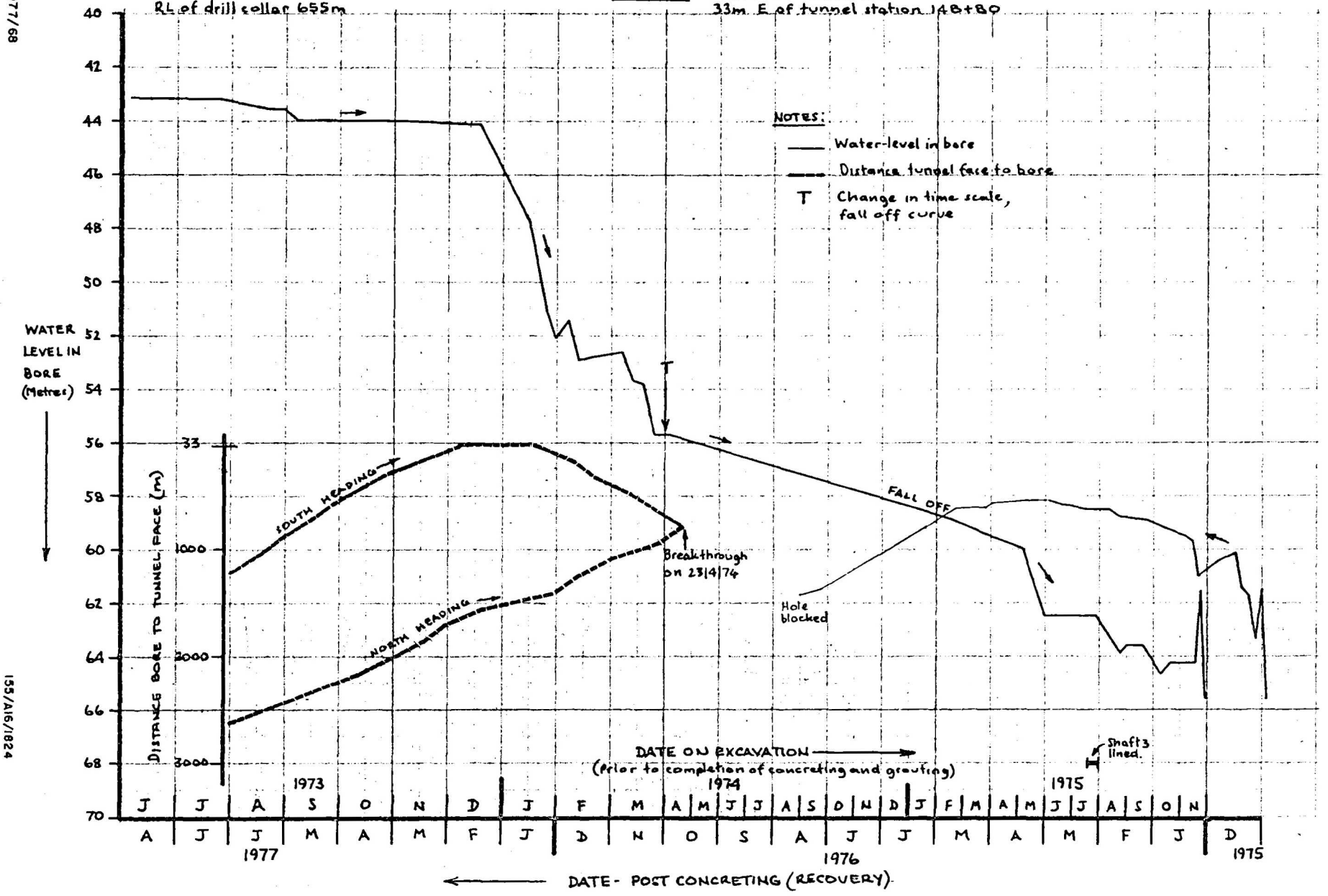


FIGURE 24

OBSERVATION BORE TS 16

LOCATION: N 597841 E 203865
53m E of tunnel stn 261+70 and 183m north of shaft 5

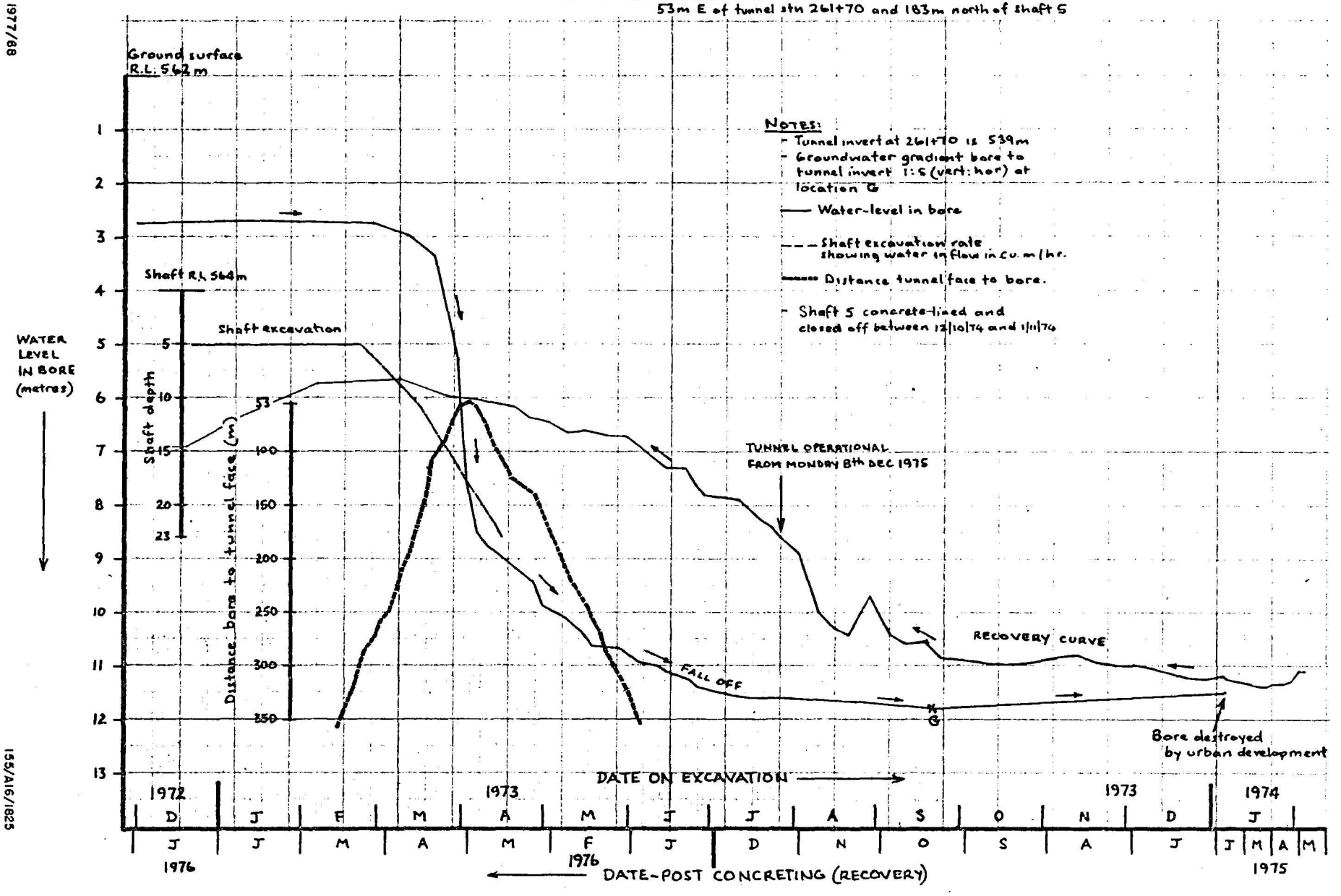


FIGURE 25

OBSERVATION BORE TS 17

LOCATION: N 592694 E 204063
38m E of tunnel station 92+80
50m SE of Shaft 2 centre-line.

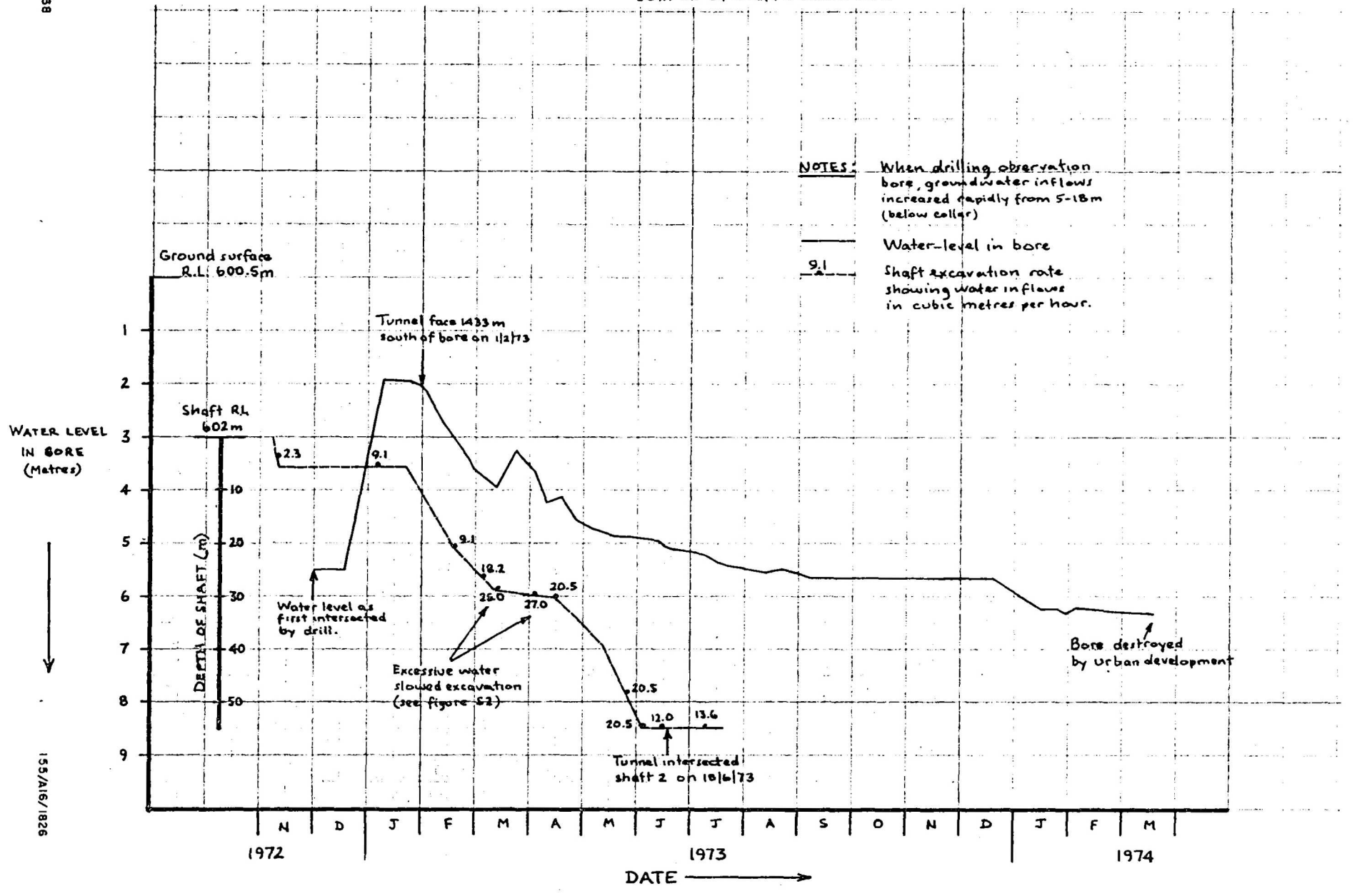
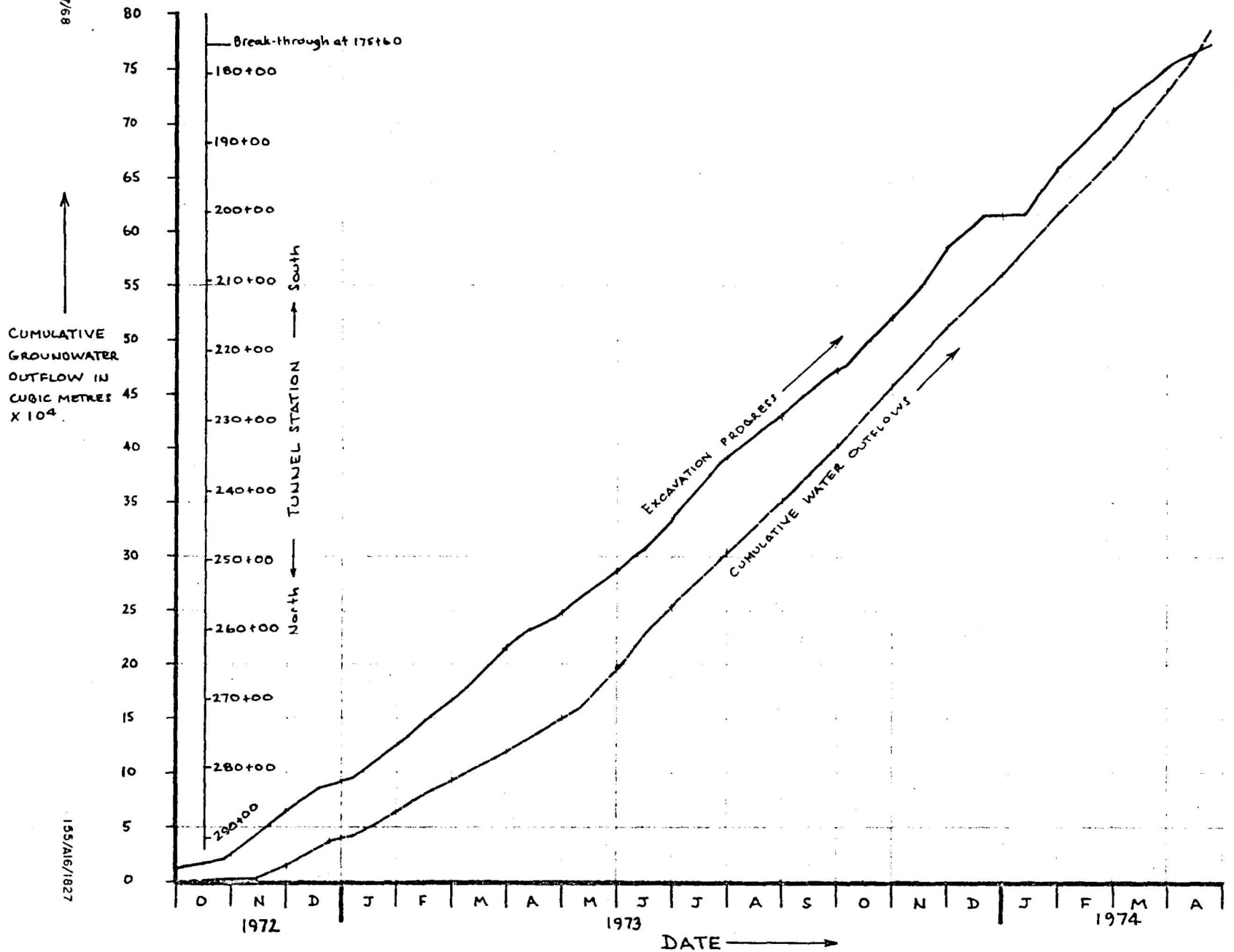


FIGURE 26

Record 1977/68

NORTH HEADING CUMULATIVE GROUNDWATER OUTFLOW AND EXCAVATION RATE



155/A16/1827

FIGURE 27

3.8.6. Groundwater before concrete lining

Most groundwater inflows were reduced to seepage and dripping by the time concrete-lining operations commenced. Inflows that were in existence were generally fairly easily panned (temporarily diverted away from concrete lining operations). Panning difficulties occurred in a few sections sucn as the contact zone (stn 178 + 00 to 179 + 20), where significant quantities of groundwater were still flowing into the tunnel.

3.8.7. Tunnelling and shaft sinking - effect on groundwater regime

The environment around the tunnel is a crystalline fractured rock aquifer with no significant intergranular porosity. The aquifer properties depend almost solely on permeability along joints and fractures. Most water inflows into the tunnel occurred via open fractured or jointed rock. Clay-coated joints and clay-filled sheared zones generally yielded negligible water inflows.

The observation holes were drilled specifically to permit study of the effect of the excavation of the tunnel, and its subsequent lining. The locations of the holes are shown in Fig. 1.

Groundwater-levels along the tunnel line were monitored at regular intervals in exploratory drill holes and groundwater observation bores offset from the tunnel line. All diamond drill holes except TS5 were destroyed by urban development or rendered useless by tunnel intersection. One observation bore, TS17 was destroyed during urban development of Kambah. Observation bores TS14 (Hindmarsh Drive) and TS16 (Streeton Drive) were partly destroyed on several occasions. Each time a hole was destroyed it was redrilled as soon as possible. Continuous records are therefore not available for all bores although recording during critical periods were generally obtained.

Results of groundwater monitoring are shown in Figures 21 to 26. Two graphs are shown in each Figure (except Figs. 22 and 26) one for dewatering during tunnel excavation and one for recovery after concrete lining and grouting.

Groundwater information gathered from observation bores to 1/4/76 is summarised in Table 6. It should be noted that bore TS5 is located

directly above tunnel centre-line; the other bores are located from 33 to 69 m off tunnel centre-line. Rainfall readings during 1972-1976 are shown in Figure 29.

Water-level in bore TS15, (Fig. 24) situated on the groundwater divide (Kambah-Weston Creek) was not affected by tunnelling until the south heading face was at its closest point to the bore (33 m). There followed a gradual lowering of the water-table from 44 m to 65.5 m. Soon after the tunnel became operational in December 1975 the water-level in the bore gradually rose, at about the same rate as fall-off. By April 1976, the bore had risen about 7 m but had dropped off 4 m by August and the hole had caved from that date. The bore had risen 7 m in 4 months.

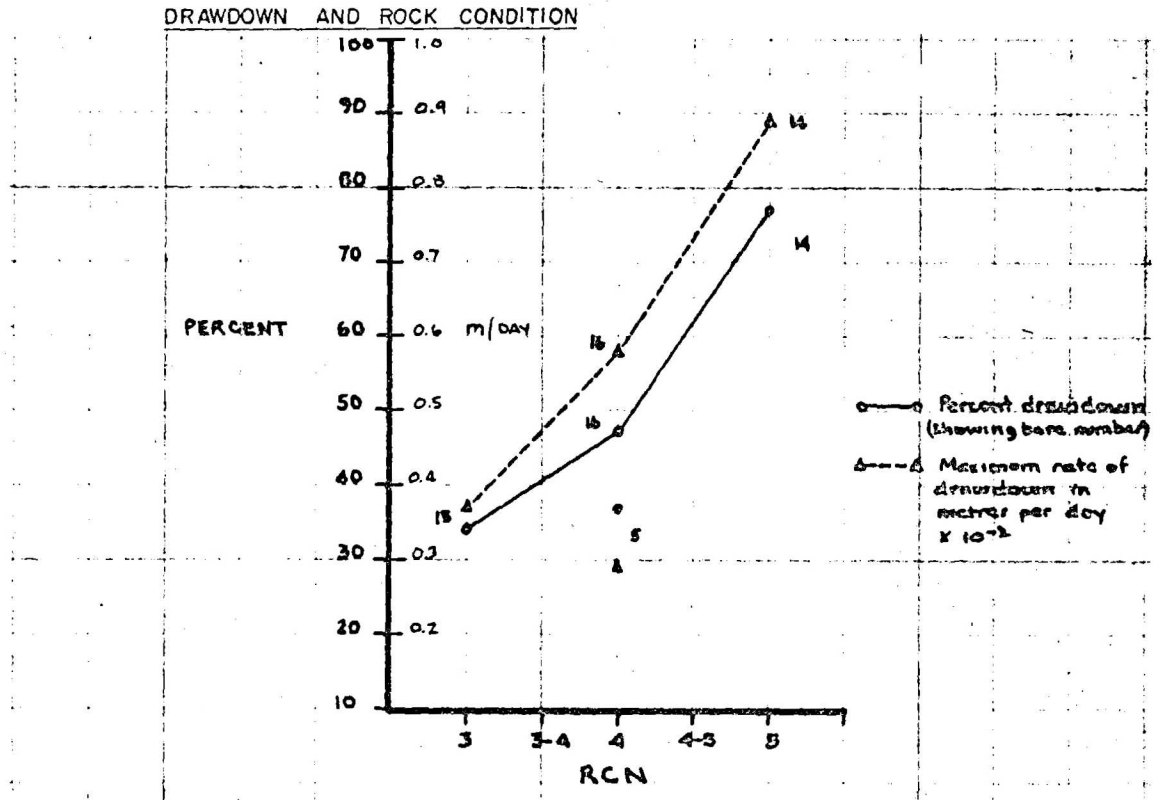
Observation bore TS5 is the only remaining exploratory diamond-drill hole (Fig. 21). TS5 is about 60 m from shaft 4, and the initial drop in water-level to 12 m is attributed to sinking of this shaft. TS5 is directly above the tunnel; its base is 30 m above tunnel invert. Sudden lowering of the water-level occurred immediately the tunnel face reached the same vertical plane along which the bore occurs. A sudden upturn in the curve (Fig. 21) occurred in June-July 1975, for which there is no obvious explanation. By December 1976 the water-level had only recovered a maximum of about 5 m up from its base level reached in December 1974. For some reason this bore is particularly sensitive to groundwater fluctuations.

Observation bore TS14 (Fig. 23) was destroyed by urban development and was out of operation for over one year. When the hole was redrilled it was found to be dry at 30 m. The ultimate depth to which the water-table fell is not known. Indications are that, by December 1976, the bore had recovered only a few metres from its base level (i.e., about one year after the tunnel became operational). Rock permeabilities in the area of TS14 and 16 are higher than indicated by drill hole pressure testing. A comparison of water inflows into the tunnel and changes in bore water-levels suggests rock permeabilities of up to 150 m per day. Investigation drill holes TS6 and 7 yielded maximum Lugeon values or 9 to 40, which is about equivalent to 30 to 120 m per year permeability or 0.1 to 0.3 m per day.

A notable increase in water inflows into the tunnel occurred 3 days before sudden drawdown occurred in bore TS14, and while the tunnel was still

FIGURE 28

DRAWDOWN VERSUS ROCK CONDITION, AND RECOVERY VERSUS ROCK CONDITION



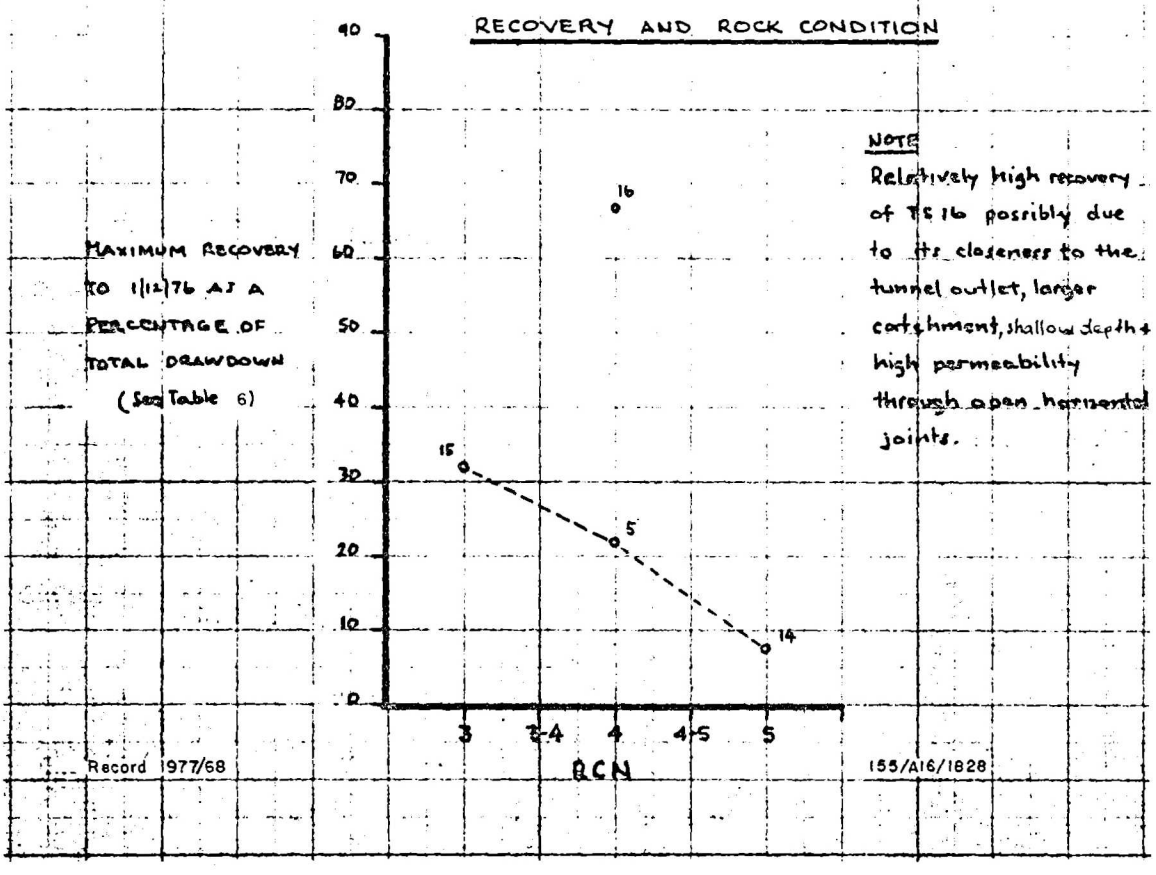


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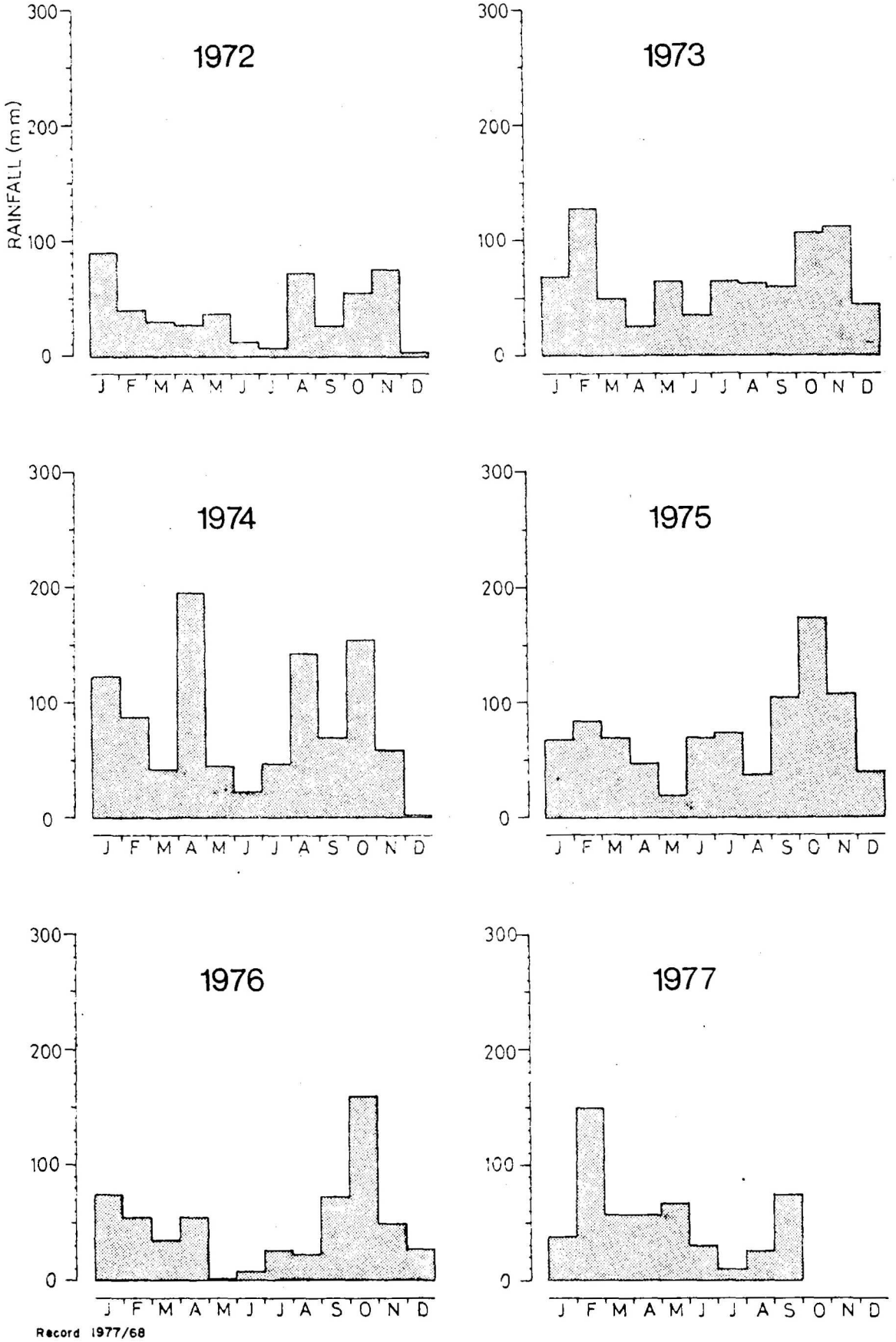
TABLE 6. ANALYSIS OF OBSERVATION BORE READINGS

MEASUREMENT	Hole Number			
	TS5	TS14	TS15	TS16
ROCK TYPE IN BORE AND TUNNEL	Rhyodacite	Rhyodacite (near southern contact with rhyolite)	Dacite	Rhyodacite
GROUNDWATER HEAD ABOVE TUNNEL INVERT (m) (PRE-TUNNEL CONSTRUCTION)	60	35	67	19.3
INTERPRETED ROCK CONDITION (FROM TUNNEL LOGS) BETWEEN TUNNEL AND OBSERVATION BORE	RCN 4	RCN 5	RCN3	RCN4
DISTANCE TUNNEL FACE TO BORE AT TIME OF FIRST SIGNIFICANT DRAWDOWN (m)	31 m (vertically) Zero m (hor.)	90	33	100
TOTAL DRAWDOWN (m)	22.4	>27 (bore destroyed before base reached)	22.6	9.0
TOTAL DRAWDOWN AS A PERCENTAGE OF ORIGINAL GROUNDWATER HEAD ABOVE TUNNEL INVERT	37%	77%	34%	47%
MAXIMUM RATE OF DRAWDOWN PER DAY (m) MEASURED OVER A PERIOD OF AT LEAST 3 DAYS)	0.29	0.89	0.37	0.58
AVERAGE RATE OF DRAWDOWN (m per day)	0.05	?	0.06	0.05
MAXIMUM RECOVERY TO 1/12/76 (m)	5.0	2.0(?)	7.3	6
MAXIMUM RECOVERY TO 1/12/76 AS A PERCENTAGE OF TOTAL (BASE) DRAWDOWN	22%	>7.5%	32%	67%

FIGURE 29

YARRALUMLA RAINFALL 1972–1977



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about 200 m from the bore. On 4 and 5 August 1973 no tunnel excavation was undertaken as all effort was spent retimbering from stn 234 + 40 (tunnel face) to stn 237 + 00. On the preceding day the water-level in the observation bore fell by 3.0 m (Fig. 23); immediately excavation ceased, the rate of drawdown decreased to about 0.3 m until excavation resumed. A return to excavation brought about an immediate sharp drawdown in the water level (Fig. 23). The initial drop from 2.5 to 3.25 m can probably be attributed to both tunnel and shaft excavation.

Observation bore TS16 (Fig. 25) began to be significantly affected when the tunnel face approached within 100 m of the bore. This happened when the tunnel reached about stn 264 + 45, and coincided with a deterioration of rock conditions and an increase in water inflows into the tunnel.

Tunnelling had no effect on observation bore TS17 (Fig. 26). Shaft 2 excavation affected the water-level in the bore to a certain extent. The reason is that the aquifer is well above tunnel crown, and at tunnel level the rock is tight with low permeability.

A correlation between RCN and percent drawdown resulting from tunnelling is shown in Figure 28. An increase in RCN from 3 to 5 generally produces a greater drawdown and indicates an increase in permeability. Drawdown is less and recovery is slower for bore TS5 than for TS16 although the rock in each is RCN 4. Vertical permeability (essentially) occurs at TS5 which lies directly above tunnel centre-line and the rate of drawdown was much slower than for TS15, 16, and 14. The rock mass in the region of TS5 is seamy but much tighter than the rock near bores TS14 and TS16, where open horizontal (and vertical) joints result in an increase in rock-mass permeability.

If percentage drawdown for holes TS14, 15, and 16 is plotted against distance from bore to tunnel it would show an unexpected increase in drawdown with increasing distance from the bore to tunnel. This is explained by comparing distances and RCN:

Bore	Distance bore to tunnel(m)	Percent drawdown	RCN
TS 15	33	34	3
16	53	47	4
14	69	77	5

The groundwater-table was lowered to tunnel invert level in the section of tunnel from about Hindmarsh Drive (near observation bore TS14) to the outlet portal. Over the remainder of the tunnel route (except at shaft sites), groundwater-levels were not lowered to tunnel level owing to the general tightness (very low permeability) of the rock mass.

The lateral extent of drawdown resulting from tunnel construction is not accurately known but between Hindmarsh Drive and the outlet portal the area affected is estimated to be in excess of 300 m either side of the tunnel (from TS14 and TS16 data).

No complaints regarding lowering of the water-table were received from rural or suburban landowners during construction of the tunnel, even in suburbs close to the tunnel.

3.9. CORRELATION OF SEISMIC PROFILES WITH GEOLOGY

Geological information obtained from tunnel mapping has been summarised onto seismic profiles. Some of these sections are shown in Plates 7 and 8 and the remainder are filed in BMR with the project records (in Project Indexing System).

For correlation purposes the seismic profiles have been divided into two parts: (i) where the depth from surface to tunnel crown is less than 38 m; (ii) where the depth of cover exceeds 38 m. It must be stressed that in situ measurements are required for a true assessment to be made as these correlations required a projection of velocity down to tunnel level.

3.9.1. Where depth of cover is less than 38 m (Plate 8).

The tunnel section to be considered here is from near the north portal (stn 288 + 50, where the cover is 10 metres) to stn 232 + 00 (Hindmarsh Drive). Correlations between seismic profile and the geology as mapped in the tunnel were generally very good. Individual geological features such as narrow faults or sheared zones were not always detected by the seismic data, but the overall rock mass condition was generally well indicated by seismic velocities.

From the north portal southwards to stn 265 + 00 the highest recorded seismic velocity boundary generally occurs close to tunnel level; the tunnel was therefore partly driven through the intermediate velocity layer and partly through the highest recorded velocity layer.

Correlating a particular degree of weathering with a narrow range of seismic velocities can be misleading. Information from the tunnel shows that a large overlap in velocity ranges sometimes exists between degrees of weathering:

Material	Seismic velocity (m/s)
Extremely weathered rock	1000-1500
Highly weathered rock	1500-2000
Moderately weathered rock	1700-2000
Slightly weathered rock	3400-4500
Fresh rock	4100

Explanations for these large overlaps in velocities can probably be explained in two ways: (a) many of the correlations were made near velocity boundaries, and exact velocities at a particular level in the profile are difficult to determine without in situ measurements: (b) the uneven distribution of narrow shears, seams, fractured, or weathered zones and open joints can effectively change the velocity of the layer from place to place, resulting in a wide range of velocities.

Experience gained in other similar volcanic rocks in the Canberra area shows an 'average' velocity/degree of weathering correlation as being:

Material	Seismic velocity (m/s)
Soil or slopewash	300-1000
Saturated soil or slopewash	1500
Extremely weathered	600-1700
Highly weathered	1000-2000
Moderately weathered	1200-3500
Slightly weathered	3000-4500
Fresh or fresh stained	>3500

Seismic velocities were generally good indicators of rock condition, and horizontal changes (anomalies) in the rock velocity were found to be

equally significant in tunnelling; for example, the velocity changes accompanying the change from rhyodacite (4100 to 4300 m/s) to rhyolite (3200 to 3400 m/s) between tunnel stations 232 + 00 and 250 + 00 (Plate 8).

An approximate correlation between seismic velocities and RCN condition based on the tunnel logs is as follows:

Rock condition number	Dacite-rhyodacite	Rhyolite
2 or 3	> 4500	No data
3 or 4	2000 -4300*	3400-3700
5	1700-2000	3200**

Notes

* In practice, considerable overlap exists between the two RCNs and their velocities. In two sections of tunnel the seismic velocity of the rock may be similar but the allotted RCN is different owing to differences in jointing (spacing, orientation, etc.).

** This velocity is considerably higher than that for the dacite equivalent but the rhyolite is generally more unstable owing to clay-coating of most defects and numerous clay shears; some RCN 6 occurs at this velocity also.

Seismic velocities were generally good indicators of tunnel sections requiring steel support; however, some narrow sheared or fractured zones not detected by the seismic survey in otherwise competent rock required steel support. Seismic velocity/steel-support data in the section of tunnel where the cover is less than 38 m are summarised below and also in Figure 25.

Velocity (m/s)	Total length of section (m)	Length steel supported (m)	Percentage
1700 - 2000	518	518	100
3200 - 3700	503	358	70
4100 - 4300	198	76	38
4500 - 5000	244	10	4

The relatively high figure of 38 percent for the 4100-4300 m/s velocity layer can probably be attributed to the fact that at the Western

Creek end of the tunnel, the crown is close to (5 m in places) the boundary with the intermediate velocity layer.

The 4 percent figure for the 4500-5000 m/s layer represents tunnel support for narrow fractured or sheared zones in otherwise very sound rock.

3.9.2. Where depth of cover is greater than 38 m (Plate 7)

The tunnel south from stn 232 + 00 has a cover of more than 38 metres except close to the south portal. This section of tunnel was only partly covered by the seismic survey as the tunnel passed under established Canberra suburbs.

Predictions of tunnelling conditions in the areas of deep cover are based not only on velocity values, but to a large extent on horizontal changes in bedrock velocities and seismic profile irregularities. South from stn 232 + 00 the tunnel is below the highest recorded velocity boundary. Bedrock velocity is generally in excess of 4000 m/s, and tunnelling conditions in sections covered by the seismic survey were generally very good (RCN 2 to 3).

Correlations between actual tunnel conditions and seismic anomalies (abrupt velocity changes) were generally not good. However, experience gained from this project has enabled far better interpretation and consistently good correlations during construction of the Ryan, Pine Ridge, and Ginninderra Sewer Tunnels.

4. CORRELATION OF AIRPHOTO-LINEAMENTS AND ROCK CONDITION

There were two problems associated with airphoto interpretation of major defects along this tunnel line: (i) south of Fisher (Kambah area) bedrock is covered by up to 15 m of soil and slopewash; and (ii) the rhyodacite and rhyolite north of stn 228 + 00 is generally poor in quality; the rock is in many places randomly sheared and fractured and detection of the many larger sheared zones is difficult, especially as many of them appear to be discontinuous.

Table 7 is a correlation of airphoto-lineaments with rock conditions

as mapped in the tunnel. Many of the major defect zones intersected during tunnelling were not detected by airphoto-interpretation, and most of these could not be observed on re-examination.

Greater success has been obtained in prediction of major defects crossing the Ryan Sewer Tunnel.

Of the 27 lineaments crossing the tunnel alignment nine were classified as being weak and 18 as strong; 60 percent of all lineaments successfully predicted poorer quality rock within about 100 m of the surface trace of the lineament. About 45 percent of weak lineaments and 70 percent of strong lineaments could be correlated with major rock defects.

5. CORRELATION WITH GEOMECHANICS CLASSIFICATION

An engineering classification of jointed rock masses - the Geomechanics Classification (GC) - has been proposed by Bieniawski (1974). It is based on six parameters, each of which is given numerical importance ratings, and the sum of the ratings is an indication of stand-up time. The higher the total rating the better the rock mass conditions. For the calculations appearing in the tables a tunnel diameter of 2.5 m has been used. Data from the tables have been presented graphically as figures 30 and 31.

The GC is a classification of rock mass, and not of individual sections of potential rock failure due to one or two seams or shears for which a separate rock stability analysis would be required.

The figures presented in Tables 8 and 9 are obtained from drill-core, field, and underground measurements and are presented here as 'back-calculations' in order to test Bieniawski's classification. The number of calculations made for this tunnel is small and therefore any conclusions must be reserved until additional information becomes available. In logging future tunnels, more notice should be taken of rock failures (small and large) and actual stand-up times obtained, so that accurate back-calculation can be made. Prediction of tunnelling conditions and overbreak may be possible from GC ratings, taking into account other factors such as tunnelling methods, support used, etc.

TABLE 7.

CORRELATION OF AIRPHOTO-LINEAMENTS WITH ROCK CONDITION

Details of lineaments			Apparent significance of lineament
Surface station	strength	strike	
256 + 00	S	110°	Rhyolite dyke (3 m wide); sheared rock in vicinity
249 + 00	W	040°	Geological boundary; north margin of rhyolite
237 + 00	S	020°	" " ; south " " "
229 + 00*	S	135°	NONE
223 + 00*	S	070°	Possibly associated with shearing, 224+00 to 227+50
218 + 00	W	080°	Sheared and fractured zone 216+00 to 218+00
200 + 00*	S	150°	Sheared section of tunnel, but difficult to correlate with particular shears
193 + 50	S	050°	Possibly associated with shearing, 194+50 to 195+50
189 + 00	W	050°	Sheared between 189+00 and 190+20
173 + 00	W	040°	NONE
166 + 50	W	140°	NONE
163 + 00	S	050°	NONE
158 + 00	W	060°	NONE
143 + 00	S	145°	NONE
131 + 00	W	150°	Possibly correlates with sheared zones, 127+00 to 138+00
118 + 00	W	130°	NONE
99 + 00	S	160°	Shear 1.5 m wide, blocky rock up to 6 m wide
98 + 00	S	110°	NONE
95 + 00	S	145°	All of these lineations correlate with sheared and fractured rock from 91+00 to 93+20
92 + 00	S	150°	
90 + 00	S	135°	
83 + 50	S	110°	Possibly correlates with blocky and partly sheared rock, 83+00 to 88+00
71 + 00	S	070°	NONE
64 + 00	S	110°	Possibly correlates with shears between 62+00 and 64+00
61 + 00	S	090°	Sheared zone, 12 m wide
55 + 00	W	050°	NONE
44 + 00	S	110	Fractured zone, partly sheared, 60 m wide

* Cannot definitely correlate with any particular (or group) of major defects as all the rock in the immediate vicinity is generally of poor quality.

** W = weak S = strong

Several points arise from the comparison between the Geomechanics Classification and actual conditions encountered in the tunnel.

(1) Rock with a GC of 33 or less (equivalent RCN 4-5) was steel-set supported. Rock with a GC of 45 (equivalent RCN 4) was supported in 40% of cases.

(2) Prediction of percent overbreak from GC values or RCN can possibly be done, but data collected so far needs to be supplemented; this can be seen in the examples calculated below:

Example 1: Given a GC rating of 30, Figure 30 would indicate a percent overbreak of 76 and from Figure 31 a RCN of 4-5

However, from Figure 3, we would expect an overbreak of 56 percent for rock with RCN 4-5, and a percent overbreak of 76 would correspond to RCN 5.

Conclusion: Some inconsistency is apparent between percent overbreak, RCN and GC.

Example 2: Given a GC rating of 70, Figure 30 would indicate a percent overbreak of 44, and from Figure 31 a RCN of 3.

From Figure 3 we would expect 44 percent overbreak for rock with RCN 3.

Conclusion: The relationship between GC and percent overbreak as presented in Figure 30 is considered approximate and further data are required. The discrepancy noted towards the lower values of GC rating in example 1 would be negligible if curve A was used; more data in this area of the figure is required.

(3) Some doubts about Bienawski's 'limits of applicability' lines exist. According to figure 2 of his paper a GC rating of <37 is not applicable to a 2.5 m diameter tunnel; however correlations between GC values of <37 and actual tunnel conditions seem to exist here.

(4) Table 9 shows that rock at stn 242 + 60 has a stand-up time of about 1 hour; however, on excavation, stand-up time was close to zero. Blasting may have loosened rock around the opening more than would be expected with careful excavation. Therefore, in using Bienawski's classification some allowance may have to be made for the method of excavation (blasting, manual, or boring machine).

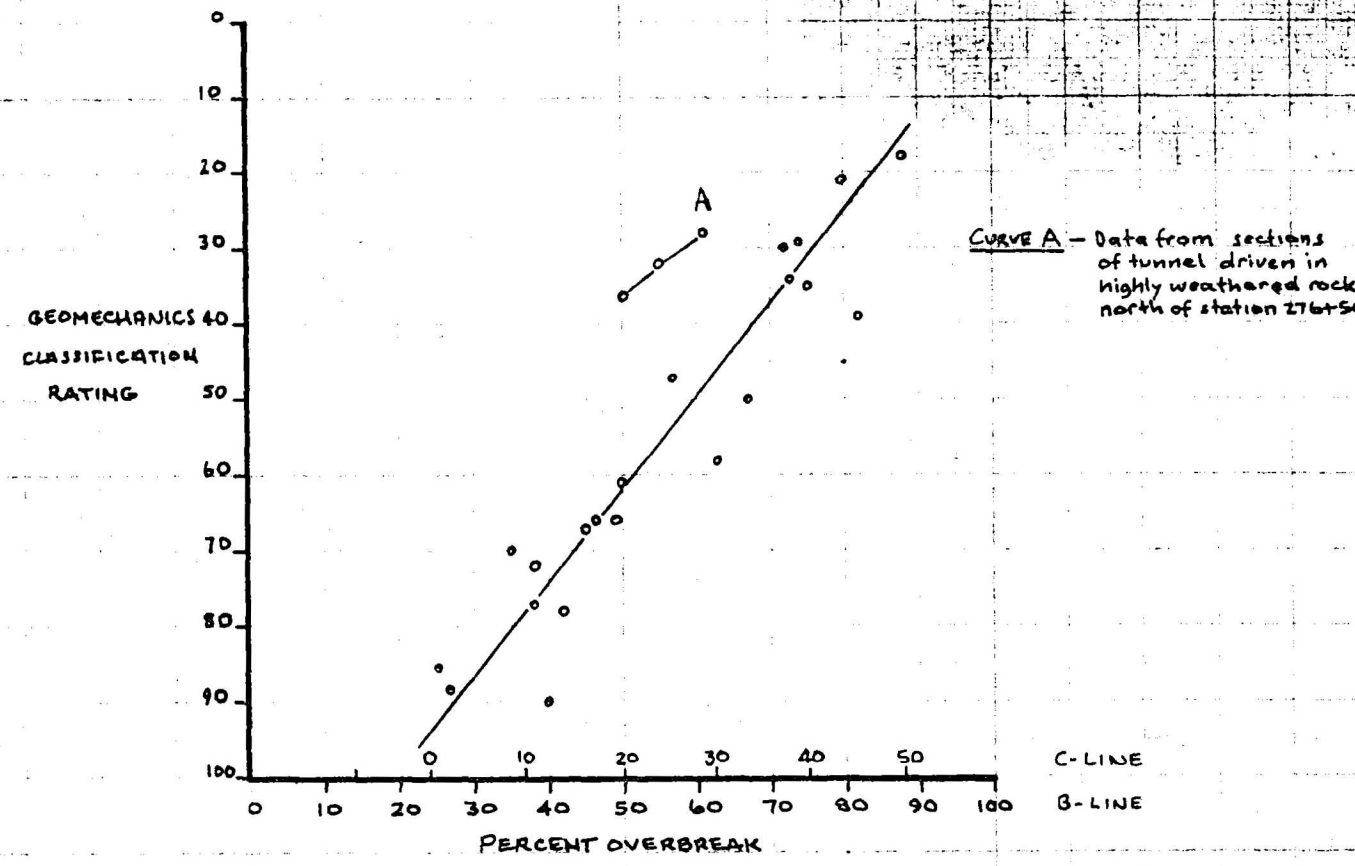
TABLE 8. ACTUAL TUNNEL STABILITY VERSUS GC PREDICTIONS BASED ON DRILL-HOLE INFORMATION

CLASSIFICATION PARAMETER	DRILLHOLE NUMBER AND PARAMETER RATINGS						
	TS 3 (100+85)	TS 6 (249+93)	TS 7 (259+80)	TS 8 (271+21)	TS 9 (276+50)	TS 10 (288+46)	TS 12 (220+07)
UNIAXIAL COMPRESSIVE STRENGTH OF INTACT ROCK	10 (200 Mpa)*	5 (150 Mpa)	10 (200 Mpa)	10 (200 Mpa)	10 (Probably 25 Mpa)	0	10 (290 Mpa)
DRILL CORE QUALITY RQD	19 (81)	8(34)	17 (76)	17 (76)	3 (8)	3	19 (81)
JOINT SPACING	20 (0.3-1m)	10(50-300mm)	15 (300 mm)	12 (150-300mm)	10	7	15
STRIKE AND DIP ORIENTATION OF JOINTS	10 (Fair)	10	10	5 (very unfavourable)	8	6	10
CONDITION OF JOINT SURFACES	10 (tight)	12	7 (some open, some gouge)	12	7	7	10
GROUNDWATER INFLOWS	8 (25 1/min)	2 (initially 125 1/m)	8	10 (none)	8 (25 1/min)	5 (25-125 1/m)	8
TOTAL RATING	77	47	67	66	36	28	72
STAND-UP TIME CALCULATION FROM TOTAL RATING	6 Months	1 Day	1 Week	1 Week	3 Hours	1 Hour	4 Months
ACTUAL CONDITIONS ENCOUNTERED IN TUNNEL	Not supported No failure in 2 years	Not supported Some barring down necessary. No failure in 2 years	Not supported Tunnel supported 7 m away. No major failure in 2 years	Some rock bolts installed to tie loose blocks.	Steel supported. Small blocks fell - some barred down.	Steel supported. Small blocks fell - some barred down.	Not supported. No failure in 2 years.
MEASURED PERCENT OVERBREAK (B-LINE)	38	57	45	46	50	61	38

* Actual figures used to obtain rating value in brackets.

FIGURES 30 & 31

1. GEOMECHANICS CLASSIFICATION VERSUS OVERBREAK



2. GEOMECHANICS CLASSIFICATION VERSUS RCN

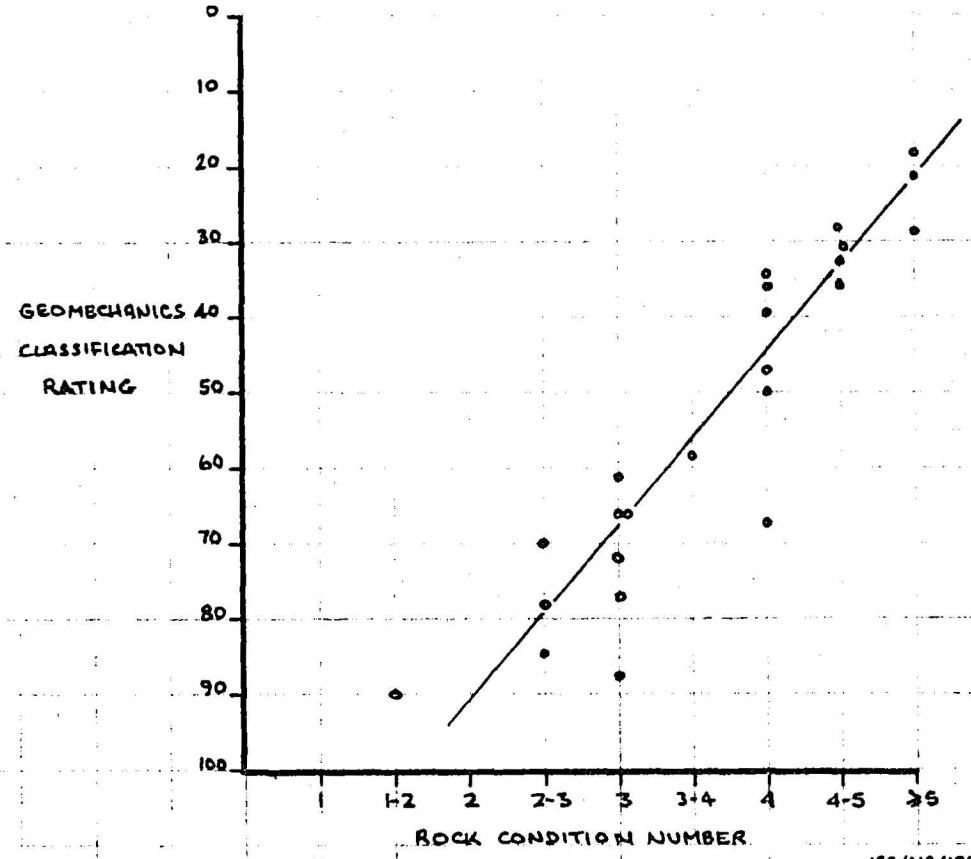


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TABLE 9. ACTUAL TUNNEL STABILITY VERSUS GC PREDICTIONS BASED ON DATA FROM TUNNEL LOGS

CLASSIFICATION PARAMETER	TUNNEL STATION																
	9+00	23+00	29+00	45+50	56+00	63+10	82+50	91+75	95+40	230+50	234+50	236+80	237+20	242+60	250+80	254+30	292+00
UNIAXIAL COMPRESSIVE STRENGTH(EST.)	8	8	8	2	8	8	5	5	10	5	1	0	1	0	5	5	0
RQD (EST.)	17	15	15	10	20	20	20	3	20	17	3	3	3	3	17	15	3
JOINT SPACING	10	10	10	10	20	20	15	5	20	10	5	10	5	10	10	10	10
JOINT ORIENTATION	10	13	10	14	12	15	13	13	15	8	4	6	4	6	3	3	6
CONDITION OF JOINT SURFACES	15	12	10	6	15	15	15	0	15	10	0	5	0	3	0	0	5
GROUNDWATER INFLOWS	10	8	8	8	10	10	10	8	10	8	5	5	8	8	2	2	8
TOTAL RATING	70	66	61	50	85	88	78	34	90	58	18	29	21	30	37	35	32
RATING STAND-UP TIME	4 Months	1 Month	2 Weeks	2 Days	2 Years	4 Years	11 Months	3 Hours	4 Years	1 Week	10 Min	1 Hour	10 Min	1 Hour	5 Hours	3 Hours	2 Hours
ACTUAL CONDITIONS IN TUNNEL	Hard intact rock. No failure in 2 years. NOT SUPPORTED			Blocky steel sup-port	Hard intact rock. No failure in 2 years. NOT SUPPORTED			Fault Steel Sup-port	Rock strong not sup-ported	Blocky NOT SUP-PORTED	Soft weak rock. Steel supported. Blocks fell in on blasting and while erecting support				Blocky NOT SUPP-ORTED	Blocky. Steel supported	
MEASURED % OVERBREAK (B-LINE)	35	49	50	67	25	26	42	73	40	63	88	74	80	72	82	75	55

6. REVIEW OF GEOLOGICAL SERVICES

Investigation stage

Refinements of geophysical interpretation by comparison with actual conditions in Tuggeranong tunnel and other recent projects have enabled tunnelling conditions to be more accurately predicted in later tunnels (Ryan, Pine Ridge, and Ginninderra). Calibration of seismic refraction results by correlation with diamond-drill hole data has obvious advantages in reducing the number of drill holes required. Reduction in the number of holes drilled for this tunnel and relying more on the seismic profiles would have resulted in much the same degree of accuracy in predicting tunnelling conditions; substantial monetary savings would have been achieved in the investigations. Shafts and portals and areas of complex or unknown geology must be drilled.

In the particular geological environments tunnelled in Canberra 1971-77 (volcanics, interbedded sediments), water-pressure testing of drill holes has not been worth the effort. Inspection of the drill core and obtaining an understanding of the general hydrological conditions in the tunnel area have been more useful in predicting groundwater conditions in the Tuggeranong, Ryan, Pine Ridge, and Ginninderra tunnels.

Construction stage

Tunnel logging procedures were found to be satisfactory in obtaining information useful in the concreting and grouting stage of construction. For the first few months of construction some difficulty was experienced in gaining entry into the tunnel for mapping purposes, as three shifts per day were worked. As a result, mapping was done between 3 a.m. and 8 a.m. Sunday mornings; however, when the tunnel face to portal distance exceeded about 2 km, too much time was wasted in foot travel, and logging had to be done during normal shift hours. It was found that the best time for logging was near the end of the drilling and loading part of the tunnelling cycle, and entry to and from the tunnel was by muck-train or locomotive. Logging did not interfere with mining operations at all.

Recording locations and quantities of water inflows must be the

responsibility of the client's supervisors as well as the project geologist. An accurate log of groundwater conditions is invaluable during concrete placement and grouting operations, as inflows of groundwater into the completed tunnel affects the operational economics of sewer tunnels. Records of total groundwater outflow should also be recorded during excavation and after concreting, and the most practical method of monitoring outflows is by V-notch weirs. Automatic recorders suffer from fouling due to a high sediment content, and several recorders are often necessary to monitor portals and shafts, whereas properly installed V-notch weirs are cheap and easy to read. Care should be taken not to read the weir when abnormal pumping operations are in progress (such as after temporary pump breakdown).

Estimating overbreak and stand-up times is important, and the geologist needs to have the co-operation of the client supervisors and contractor's shift bosses to obtain information for the time the geologist is not on site.

Logging of shafts is a problem owing to the cramped working conditions. Logging during crib times is the only time available, apart from brief inspections with the client's supervisor. Time is only available during shift hours if poor ground conditions are met. Logging a shaft being raised is even more difficult until the shaft is completed, particularly if heavy support has been erected.

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APPENDIX; DEFINITIONS OF TERMS

ROCK CONDITION NUMBERS

1. Descriptions of the Rock Condition Numbers (RCN) have been modified after Terzaghi (1946) and Deere, Merritt, & Coon (1969) to suit geological conditions encountered in tunnelling operations in the ACT since 1971. To date these tunnels have passed mainly through acid volcanics and sedimentary rocks derived from them.

2. Predicted support requirements for each RCN should be used only as a guide, as very narrow but poorly oriented defects in an otherwise long section of competent rock (e.g., RCN 2) may require 2 or 3 steel sets or a few rock bolts for stabilisation. The predictions of support assume an excavated tunnel diameter of up to 4m.

3. It should also be noted that RCN 7 and 8 have not been recorded to date in the ACT.

ROCK CONDITION

Number

Description of the Rock Mass

1. HARD INTACT ROCK: Rock massive, very hard and very strong, with no significant joints or other defects. Breaks across sound rock when blasted. No support necessary.

2. HARD WIDELY JOINTED ROCK: As above, but may be foliated or bedded with a fairly high resistance to separation of surfaces. Prominent continuous joints spaced 1-3 m are tight; joints usually not continuous for more than a few metres. No support required.

3. MASSIVE, MODERATELY JOINTED: Rock mostly hard and strong. Continuous joints generally spaced 0.5-1 m are usually fairly tight, but some water seepage along joints may occur. Rock may be partly blocky in places, and generally breaks along joint surfaces when

blasted. Steel or rock-bolt supports generally not required in 3 m diameter tunnel; in a 4 m tunnel, some rock-bolts may be required where blocky or poorly oriented defects cross the tunnel.

4. MASSIVE, MODERATELY JOINTED, SEAMY: As above but defect surfaces generally clay-coated and loose. Clay seams and sheared or fractured rock with clay common. Rock may be moderately weathered or altered and soft in parts. Steel-set support (1-1.3 m spacing) sometimes required in tunnels up to 3 m diameter; more often in 4 m tunnel. Rock-bolts may be preferable in places.

5. CLOSELY JOINTED AND SEAMY: Closely jointed, seamy, and fractured rock; joints and fractures are loose and open (where not clay-filled), and may result in large water inflows into the excavation; includes highly and extremely weathered (or altered) rock. May exert considerable weight on steel-set supports; steel supports spaced at 1 m (or less) with heavy timber lagging. Rock-bolts not usually effective.

6. SEAMY AND CRUSHED ROCK: Includes unconsolidated sand, slopewash, etc. Refers to fault zone material (gouge) or shattered rock where clay and gravel make up the greater percentage of the material mass. If water content is high, these materials may run or flow and exert significant side pressures. Stand-up time near zero. Rock-bolting not effective. Steel sets 0.5 m centres, invert struts, and possibly linear plates. Shotcrete or gunite often effective in containing running ground.

7. SQUEEZING GROUND: Slow movement of rock into the tunnel without perceptible volume increase (rock with clay minerals with low swelling characteristics).

8. SWELLING GROUND: Material expands in volume upon exposure to water (e.g. montmorillonite clay, serpentinite, anhydrite, etc.).

DEGREES OF ROCK WEATHERING

FRESH : No discolouration or loss in strength.

FRESH STAINED	:	limonitic staining along fractures; rock otherwise fresh and shows no loss of strength.
SLIGHTLY WEATHERED	:	Rock is slightly discoloured, but not noticeably lower in strength than the fresh rock.
MODERATELY WEATHERED	:	Rock is discoloured and noticeably weakened; N-size (54 mm) drill core generally cannot be broken by hand across the rock fabric.
HIGHLY WEATHERED	:	Rock is discoloured and weakened; N-size (54 mm) drill can generally be broken by hand across the rock fabric.
EXTREMELY WEATHERED	:	Rock is decomposed to soil, but the original rock fabric is mostly preserved.

ROCK SUBSTANCE

This is defined as intact, effectively (for engineering purposes) homogenous rock. Repeated mechanical tests on the material would give acceptable coefficients of variations (e.g., uniform results).

ROCK MASS

Rock mass is a body of material which is not effectively homogenous, that is, the rock substance is crossed by natural defects such as joints, faults, seams etc.

SHEARED ROCK

Consists of rock intersected by close (<1 cm), slightly curving intersecting fracture planes; the fracture surface may be smooth, polished, slickensided, or coated with clay.

CRUSHED ROCK

Consists of rock which is mechanically disintegrated but not obviously chemically decomposed.

FRACTURED ROCK

Consists of rock which is intensively jointed in several directions. Fracture surfaces are often clay-coated.

FAULTED ROCK

Faults can be sheared, crushed, or fractured rock, and where relative displacement of rock can be seen. Unless evidence for faulting is quite definite the term should not be used.

JOINT SPACING

Very close - joints spaced < 5 cm
Close - joints spaced 5 to 30 cm
Moderately close - joints spaced 30 cm to 1 m
Wide - joints spaced 1 to 3 m
Very wide - joints spaced > 3 m

JOINT APERTURE

This describes the amount of separation of the joint surfaces. Joints may be open or tight. If two joint faces fit perfectly it is probable that the joint in the rock mass was tight (or closed). However, if they do not fit it probably means that the joint was open; or possibly filled with clay that has been washed away during drilling.

BEDDING

Laminated - <10 mm thick
Thinly bedded - 10 mm to 100 mm thick
Thickly bedded - >100 mm thick

GRAINSIZE

Coarse-grained - 1 mm to 4 mm in diameter
Medium-grained - $\frac{1}{4}$ mm to 1 mm in diameter
Fine-grained - $< \frac{1}{4}$ mm in diameter

ROCK QUALITY DESIGNATION (RQD)

RQD is the ratio (expressed as a percentage) of length of core recovered to the total length of core run, counting only those pieces of hard and sound rock 10 cm in length or longer.

REFERENCE

DACITE

SHEARED, FRACTURED OR FAULTED ZONES

LOCATION

Accurate

Inferred

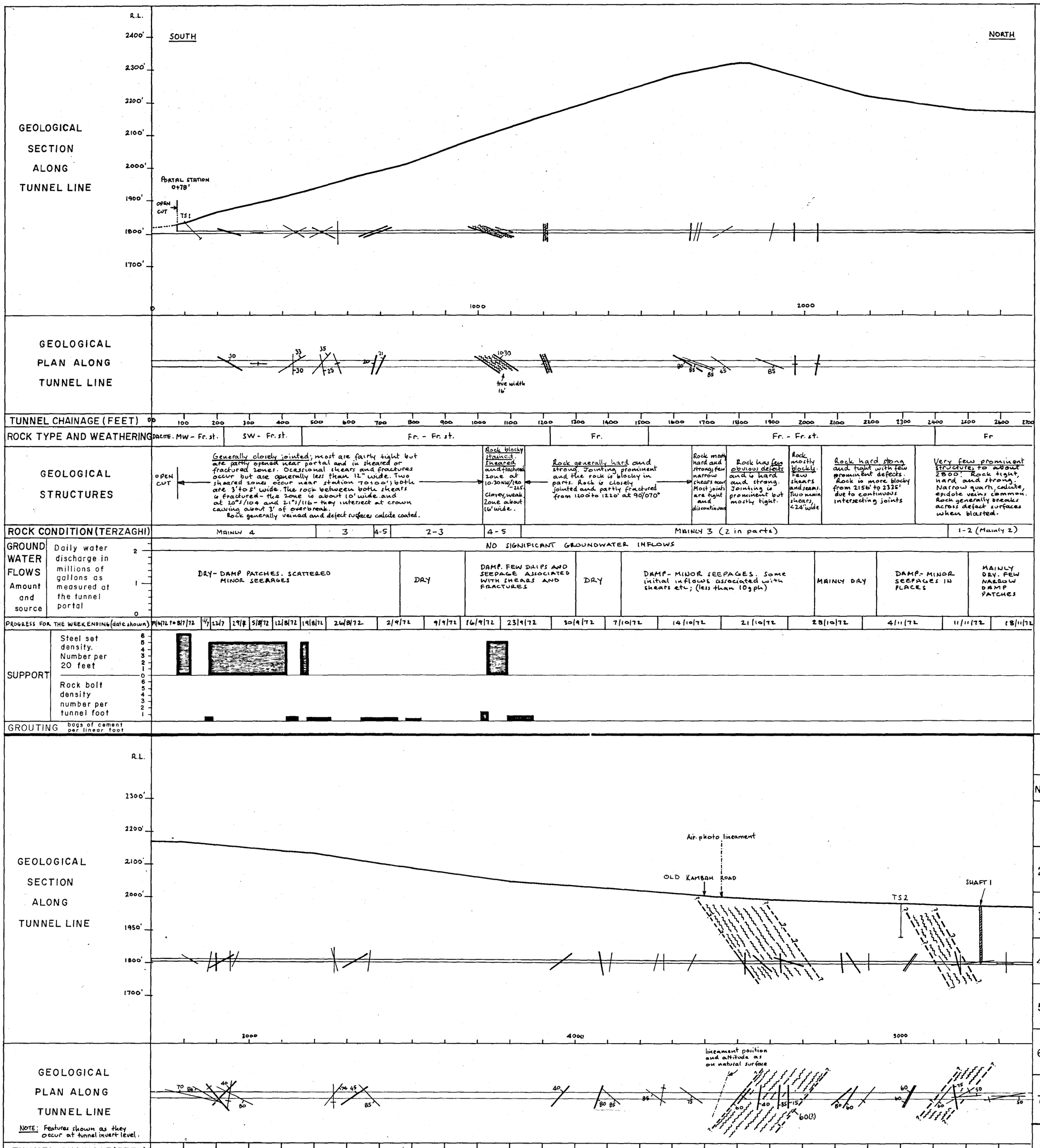
WIDTH AND ATTITUDE (Attitude in plan only)

45 — 3"-11" wide, showing dip.

85 — 12"-5' wide, showing dip.

10 — ≥ 5' wide, showing dip.

TS2 — Diamond drill hole



Record 1977/68

AMENDMENTS

No	DESCRIPTION	AUTHOR / CHECKED	DATE
1			
2			
3			
4			
5			
6			
7			

GEOLOGY BY D. C. PURCELL

COMPILED AND CHECKED: D. C. Purcell — PROJECT GEOLOGIST

CHECKED AND APPROVED: D. C. Purcell — SENIOR GEOLOGIST

SUPERVISING GEOLOGIST

Scale (1/4 = 1)

HORIZONTAL 0 100 200 300 400 Feet

VERTICAL 0 200 400 Feet

TITLE
SUMMARY GEOLOGICAL LOG
CHAINAGE 0+00 TO 54+00

PROJECT
TUGGERANONG SEWER TUNNEL

TO ACCOMPANY RECORD — SHEET 1 OF 6

DRAWING NUMBER 155/A16/1847 — PLATE 1

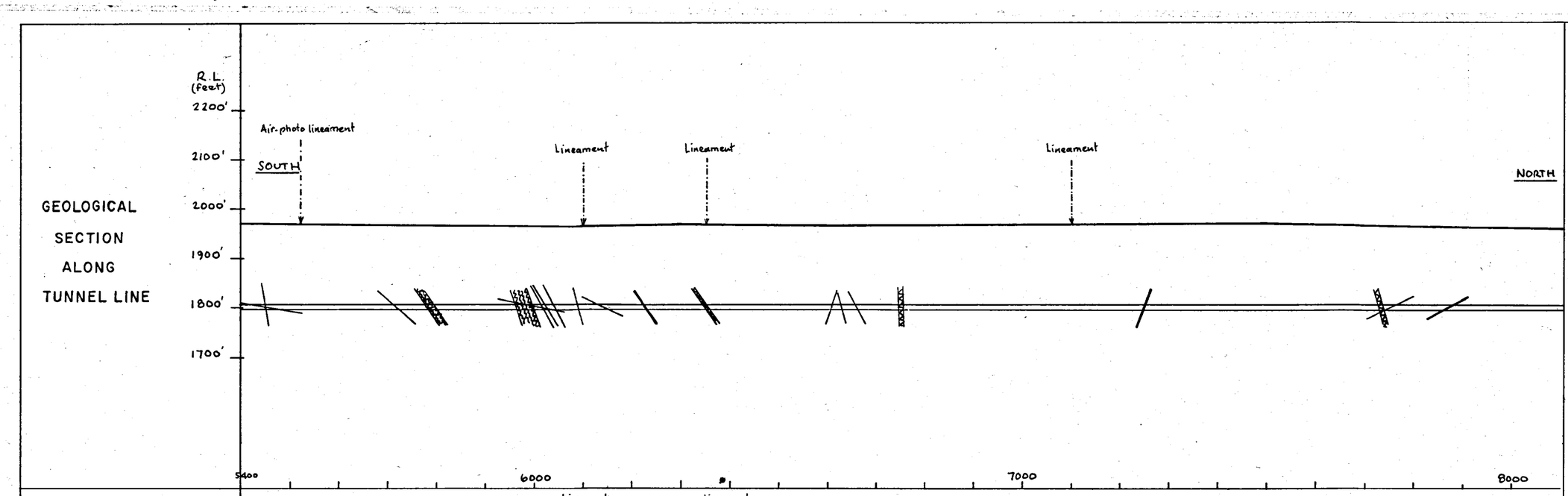
REFERENCE

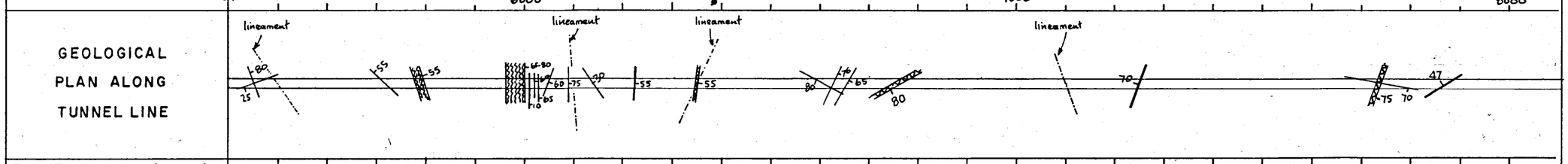
DACITE

SHEARED, FRACTURED OR FAULTED ZONES

LOCATION
— Accurate
?—?— Inferred

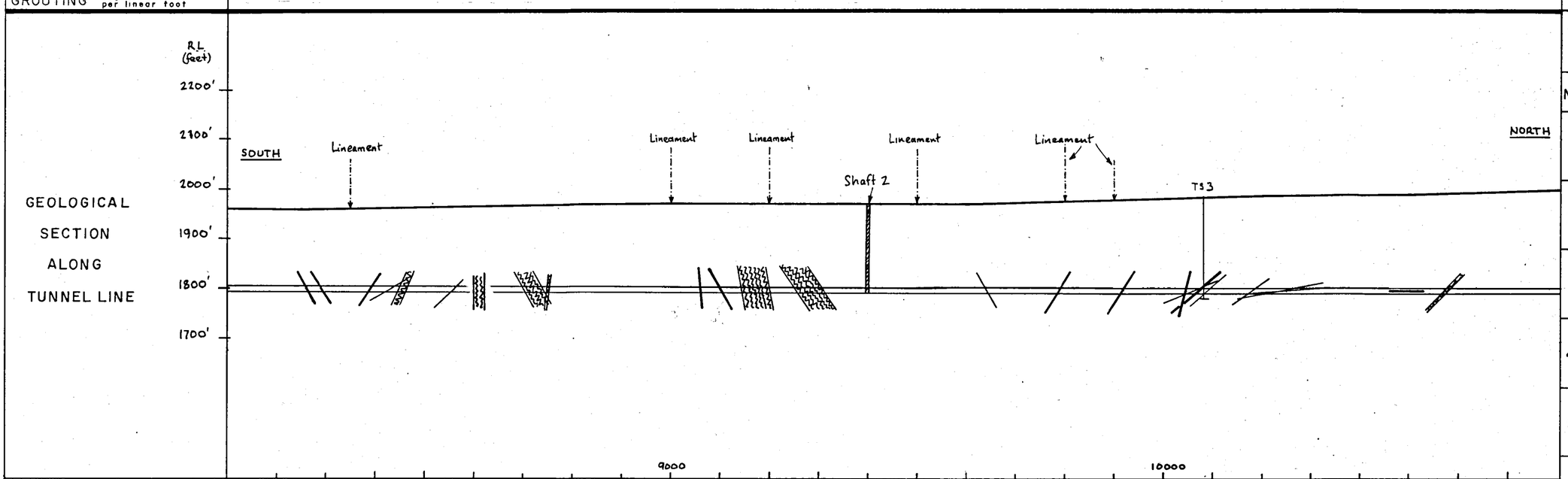
WIDTH AND ATTITUDE (Attitude in plan only)
45 — 3"-11" wide, showing dip
60 — 12"-5' wide, showing dip
80 — ≥ 5' wide, showing dip

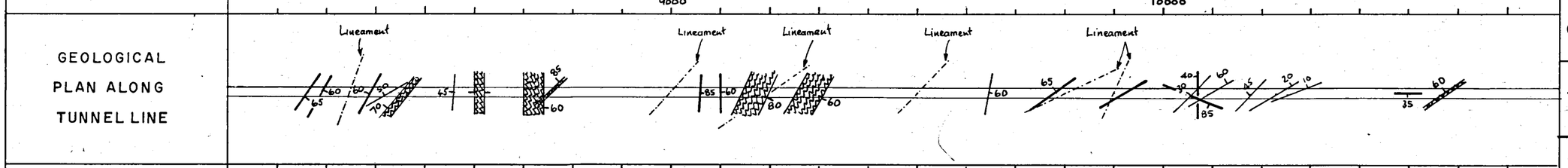




TUNNEL CHAINAGE (FEET)	5400	5600	5800	6000	6200	6400	6600	6800	7000	7200	7400	7600	7800	8000
ROCK TYPE AND WEATHERING	DACITE - FRESH (Blue-grey color)		Fresh to Fresh-stained	FRESH	Fresh-stained	DACITE - FRESH	Fresh to Fresh stained	FRESH	Fr. St.	DACITE - FRESH. (Minor alteration in main sheared zones.)	DACITE - FRESH TO FRESH STAINED		Fresh to MW	Fresh to Fresh stained	FRESH
GEOLOGICAL STRUCTURES	5200-5350 (See comments sheet 1)	Rock condition very good. Closely jointed in parts but rock tight. Joints mostly not continuous for more than a few feet. Jointing generally random.	Blocky; seamy parts. Sheared & fractured zone between 5778-5800 at 55°N/072	Mostly strong, hard rock. Tight. No major defects	Sheared 5951-6000 at about 65-80N 090 clayey. Blocky & seamy to 6026	Rock mostly strong hard and tight although blocky and closely jointed in narrow sections. A 4' wide shear at 6230' at 55N/095 and a 5' shear at 6350' at 55N/098. The rock is particularly blocky and seamy between 6430' and 6475'			Generally blocky and closely jointed. Occassional sheared zones (mostly <6" wide) occur. Sheared and fractured zone occurs at 6750' (about 10' wide) with an attitude 85°E-90°/150°.	Condition improves from 6860'. Mostly moderately-closely jointed but tight and strong. Blocky and partly seamy from 7020'-7050'. Also sheared and fractured at 7155' (4' wide and at 70°/290)	Rock more blocky than previous section. Rock mass is loose in parts and surfaces are often clay coated		Fractured and blocky 7650'-7765'; zone probably at 70°/140 to 75N/110°. Main shear at 7725 (10' wide) at 75N/110. A further shear (5' wide) occurs at 7870' at 47°SW/147. This section mostly blocky to closely jointed		Moderately to closely jointed but mostly tight to 8220. Some sections (narrow) are a little blocky.
ROCK CONDITION (TERZAGHI)	2 TO 3	3-4	3	4	3		3-4	3	3/4	MOSTLY 3 (Varies to 2 to 4 in narrow sections only)	3-4		3	3-2
GROUND WATER FLOWS (Daily water discharge in millions of gallons as measured at the tunnel portal)	NO SIGNIFICANT WATER INFLOWS.													
	DAMP	DRY	DAMP	SEEPAGE, MINOR DRIPPING IN PLACES	DRY	SEEPAGE SOME DRIPPING OPEN FRACTURE WITH 400 gph INITIALLY. ABOUT 100 gph CONTINUE.	MAINLY DRY	SOME SEEPAGE - SOME DRIPPING IN PLACES.	DRY	DAMP	DRY	DAMP	DRY	DAMP	DAMP - SOME SEEPAGE. DRIPPING IN ISOLATED PLACES.	MAINLY DRY (Scattered small damp patches)
PROGRESS FOR THE WEEK ENDING (date shown)	3/3/73	10/3/73	17/3/73	24/3/73	31/3/73	7/4/73	14/4/73	21/4/73	28/4/73	5/5/73	12/5/73		19/5/73	
SUPPORT: Steel set density. Number per 20 feet														
SUPPORT: Rock bolt density number per tunnel foot														
GROUTING (bags of cement per linear foot)														

NOTE: EASTER 20/4/73 to 26/4/73





TUNNEL CHAINAGE (FEET)	8200	8400	8600	8800	9000	9200	9400	9600	9800	10000	10200	10400	10600	10800
ROCK TYPE AND WEATHERING	DACITE - FRESH		Fresh to fresh stained		FRESH		Fr. St to slightly weathered. Some alteration.	FRESH					FRESH TO FRESH STAINED	
GEOLOGICAL STRUCTURES	Same as from 7873'	More blocky and seamy from 8220. Narrow shear and fracture occur. Fractured and partly sheared from 8380' to 8400'	TIGHT, STRONG ROCK	Conditions deteriorate from 8440'. Rock is closely jointed and fractured zones occur more frequently. Partly fractured 8600-8620 at about 90/090°. Seamy and partly fractured zone 8700-8740 at 60N/090. Chloritic shear at 8750 3' wide at 85N/135 (but rock severly affected to a width of about 8'. Rock seamy between shears.	A gradual improvement occurs North from 8820; More marked from 9000. Rock partly blocky where joints continuous but the rock mass is mostly hard, strong and tight.		SHEARED & FRACTURED. Many individual shears and fractures occur. Particularly N of 9255 the rock is seamy, blocky & heavily limonite stained. Two main zones: i) 9145-9200 at 75-85N/110-115 and ii) 9255-9210 at 60°N/120(?)	Dacite strong and tight. Few narrow shears or fractures - some limonite stained. Partly blocky and seamy from 100+40 to 100+90 and 101+60 to 103+00 due to shears.				Rock blocky - seamy in parts. Many narrow (<12" wide) shears but rock mass remains fairly strong.	
ROCK CONDITION (TERZAGHI)	3-2	3-4	3	3-4	3	3-4	3	3-4	3	4	4-5	2-3	Mostly 3	2-4
GROUND WATER FLOWS (Daily water discharge in millions of gallons, as measured at the tunnel portal)	NO SIGNIFICANT WATER INFLOWS. (Tunnel making constant 2500 gph through shaft 2)													
	DRY	DAMP MINOR SEEPAGE DRIPPING IN PLACES	MAINLY DRY (Minor damp patches)	DAMP - MINOR SEEPAGE DRIPPING IN PLACES (in shears, fractures, etc)		DAMP - DRY IN PLACES	DAMP - SEEPAGE AND DRIPPING IN PLACES.	DRY IN PLACES - MINOR SEEPAGES OR DRIPPING FROM SHEARS OR FRACTURES						
PROGRESS FOR THE WEEK ENDING (date shown)	19/5/73	26/5/73		2/6/73		9/6/73	16/6/73	23/6/73	30/6/73	7/7/73	14/7/73	21/7/73	28/7/73	
SUPPORT: Steel set density. Number per 20 feet														
SUPPORT: Rock bolt density number per tunnel foot														

Record 1977/68

AMENDMENTS

No	DESCRIPTION	AUTHOR / CHECKED	DATE
1			
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GEOLOGY BY D.C. PURCELL

COMPILED AND CHECKED	CHECKED AND APPROVED
D.C. Purcell	DCPurcell
PROJECT GEOLOGIST	SENIOR GEOLOGIST

SUPERVISING GEOLOGIST

Scale 1/H = 1
HORIZONTAL 0 100 200 300 400 Feet
VERTICAL 0 200 400 Feet

TITLE
SUMMARY GEOLOGICAL LOG
CHAINAGE 54+00 TO 108+00

PROJECT
TUGGERANONG SEWER TUNNEL

TO ACCOMPANY RECORD — SHEET 2 OF 6

DRAWING NUMBER 155/A16/1847 — PLATE 2

REFERENCE

SHEARED, FRACTURED OR CRUSHED ZONES

LOCATION

— Accurate

?— — —? Inferred

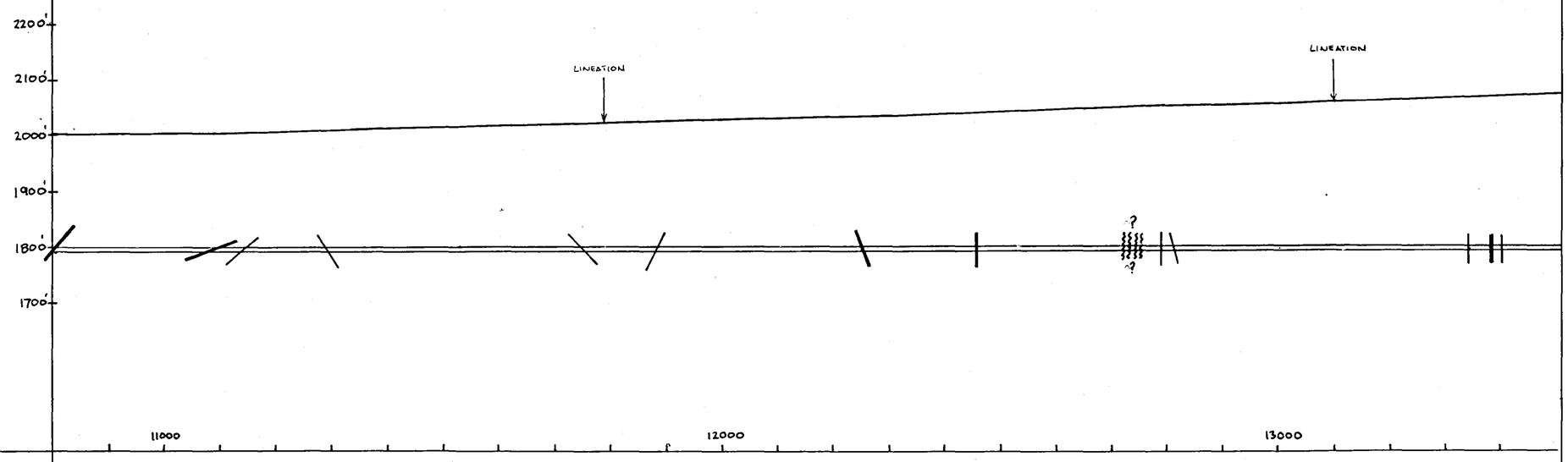
WIDTH + ATTITUDE

80 — 3"-11" wide

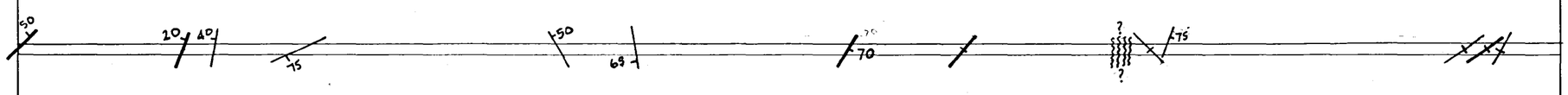
50 — 1'-5' wide

≥ 5' wide; actual width to scale.

GEOLOGICAL SECTION ALONG TUNNEL LINE



GEOLOGICAL PLAN ALONG TUNNEL LINE



TUNNEL CHAINAGE (FEET)	10800	11000	11200	11400	11600	11800	12000	12200	12400	12600	12800	13000	13200	13400

ROCK TYPE AND WEATHERING: DACITE - FRESH (Minor limonite staining on open fractures or shears)

GEOLOGICAL STRUCTURES:
- Rock mass strong and tight. No major defects - joints randomly orientated. Rock sheared and fractured at 10800 (6' wide) and 11090 (2' wide)
- Rock partly blocky with ragged tunnel profile. Rock strong. No major defects
- Rock tight and very strong. Rock blocky near main shear zone at 12250 (2' wide)
- Rock partly blocky but remains fairly tight. Narrow sheared zones common. Rock particularly blocky and seamy with overbreak from 12320 + 12440

ROCK CONDITION (TERZAGHI): 2-3 (3-4 in sheared zones) | 2 | 3-2 | 2-3 | 3 | 3 and 4

GROUND WATER FLOWS — Amount and source — Daily water discharge in millions of gallons as measured at the tunnel portal (2, 1, 0)

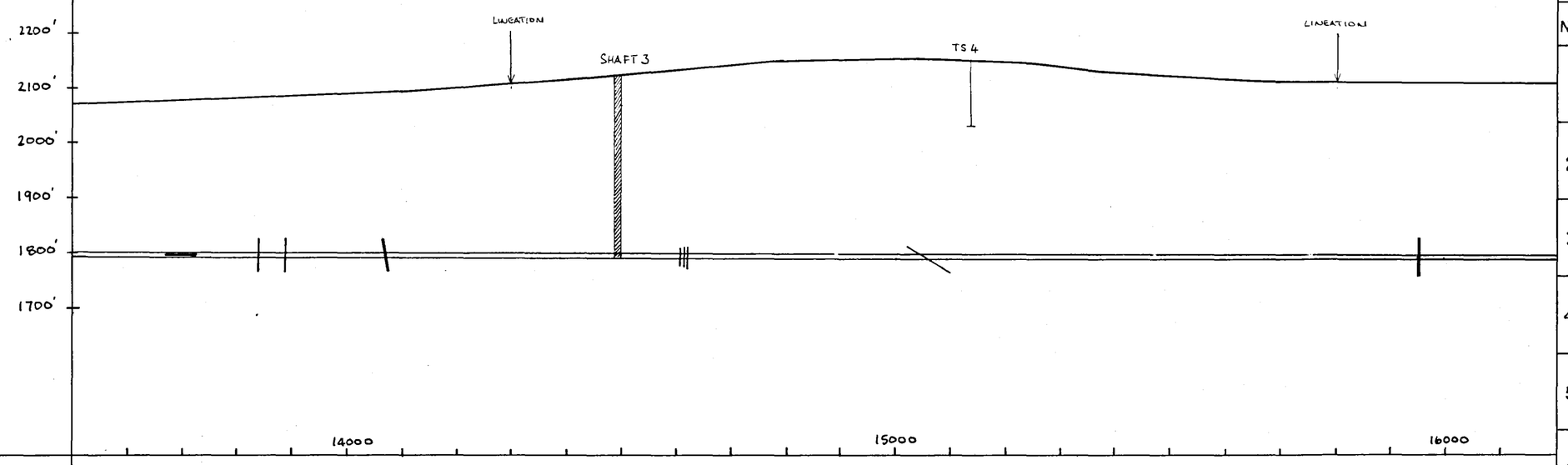
- Mainly damp. Dry places
- Damp - some dripping
- Mainly dry - few seepages or drips near shears or fractures only.
- Mainly dry - few damp patches
- DAMP WITH SEEPAGES
- Mainly dry
- Some seepage and dripping
- Damp - some dry patches
- Some seepage and dripping
- Mainly dry - few damp patches

PROGRESS, HEADING, DATE: 4/8/73 | 11/8/73 | 18/8/73 | 25/8/73 | 1/9/73 | 8/9/73 | 15/9/73 | 22/9/73 | 28/9/73 | 6/10/73 | 13/10/73

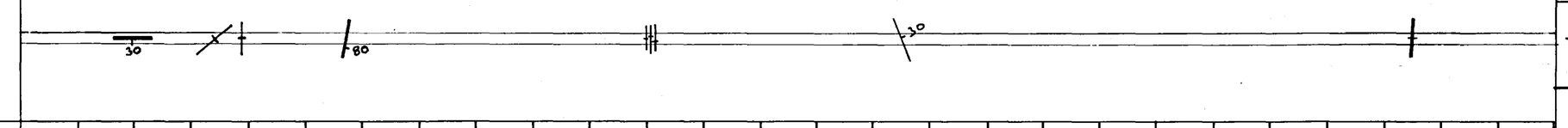
SUPPORT — Steel set density. Number per 20 feet (6, 5, 4, 3, 2, 1, 0); Rock bolt density number per tunnel foot (6, 5, 4, 3, 2, 1)

GROUTING — bags of cement per linear foot

GEOLOGICAL SECTION ALONG TUNNEL LINE



GEOLOGICAL PLAN ALONG TUNNEL LINE



TUNNEL CHAINAGE (FEET)	13600	13800	14000	14200	14400	14600	14800	15000	15200	15400	15600	15800	16000	16200

ROCK TYPE AND WEATHERING: DACITE - FRESH (minor limonite staining on open fractures or shears) - Partly altered 159+40 to 161+80

GEOLOGICAL STRUCTURES:
- As for previous section
- Rock mostly tight, clay in some joints
- Rock blocky + seamy near 4' wide shallow dipping shear zone
- Rock generally tight although seepage through horizontal joints and some sheared zones occurs in places.
- Joints mostly discontinuous, rock tight and strong with no significant defects.
- Rock generally fairly tight, hard and strong. Blocky + partly open in places. Quartz-epidote veining common.

ROCK CONDITION (TERZAGHI): 3+4 | 2 | 3+4 | 3-2 (4 in shears) | 2 | MAINLY 3. Some 2 or 4 in narrow sections

GROUND WATER FLOWS — Amount and source — Daily water discharge in millions of gallons, as measured at the tunnel portal (2, 1, 0)

- Damp - some seepage and dripping
- DRY
- Generally damp; Dry in places. Some minor inflows (250 gph at 149+45')

XMAS BREAK 8/12/73 to 17/1/74

PROGRESS, HEADING, DATE: 20/10/73 | 27/10/73 | 3/11/73 | 10/11/73 | 17/11 | 24/11 | 1/12 | 8/12 | 19/1 | 26/1/74 | 2/2/74 | 9/2 | 16/2 | 23/2 | 2/3

SUPPORT — Steel set density. Number per 20 feet (6, 5, 4, 3, 2, 1, 0); Rock bolt density number per tunnel foot (5, 4, 3, 2, 1)

Record 1977/68

AMENDMENTS

No	DESCRIPTION	AUTHOR / CHECKED	DATE
1			
2			
3			
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GEOLOGY BY D.C. PURCELL + G.B. SIMPSON

COMPILED AND CHECKED: D.C. Purcell — PROJECT GEOLOGIST

CHECKED AND APPROVED: D.C. Purcell — SENIOR GEOLOGIST

SUPERVISING GEOLOGIST

Scale
HORIZONTAL 0 100 200 300 400 Feet
VERTICAL 0 200 400 Feet

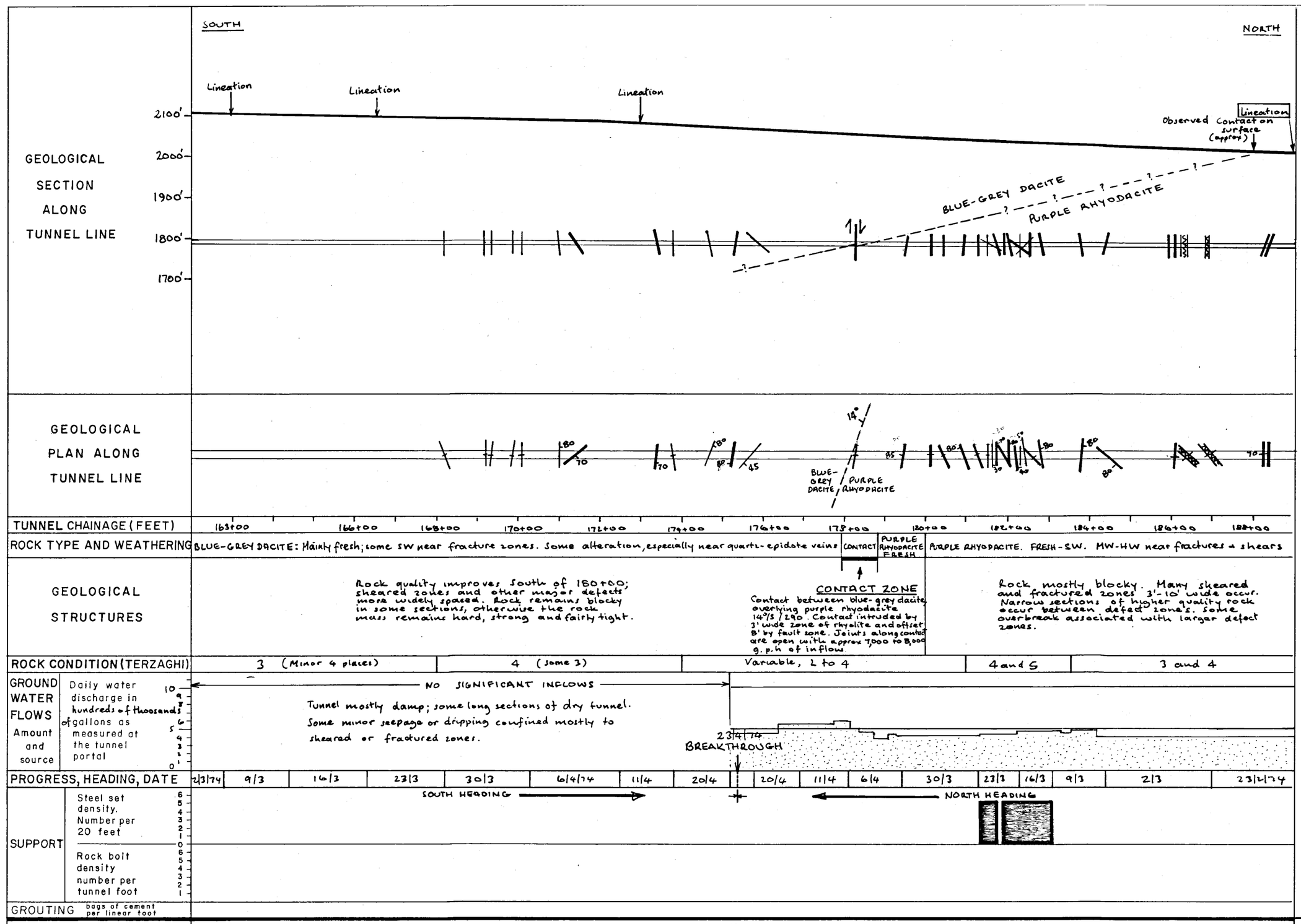
TITLE
SUMMARY GEOLOGICAL LOG
CHAINAGE 108+00 TO 162+00

PROJECT
TUGGERANONG SEWER TUNNEL

TO ACCOMPANY RECORD — SHEET 3 OF 6

DRAWING NUMBER 155/A16/1847 — PLATE 3

M(Pf)144



REFERENCE

SHEARED, FRACTURED OR CRUSHED ZONES

LOCATION

Accurate

Inferred

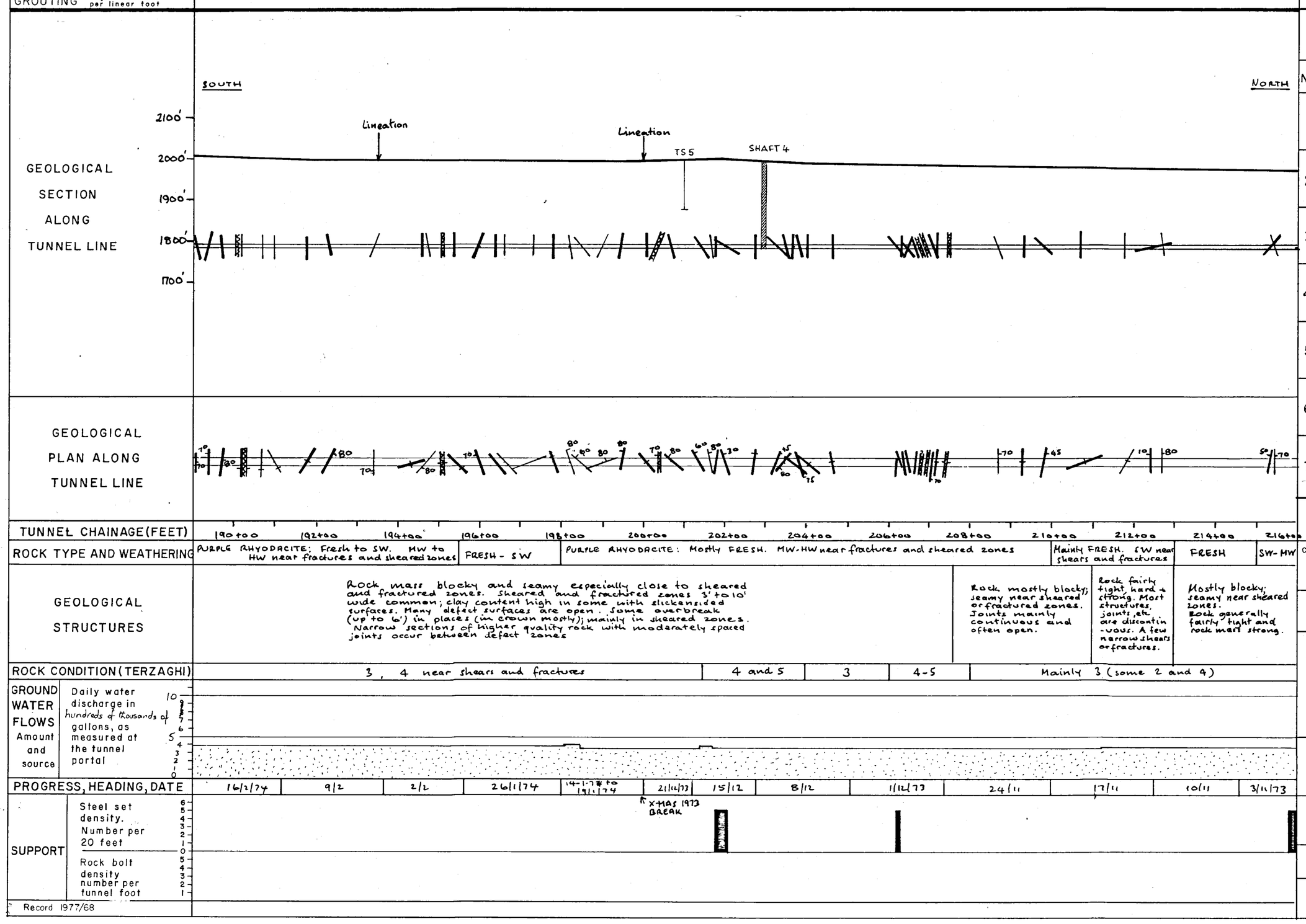
WIDTH AND ATTITUDE

2"-11" wide

1'-5' wide

≥ 5' wide; actual width to scale

Relative movement along fault zone.



Record 1977/68

AMENDMENTS

No	DESCRIPTION	AUTHOR / CHECKED	DATE
1			
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4			
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GEOLOGY BY
D.C. PURCELL + G.B. SIMPSON

COMPILED AND CHECKED: D.C. PURCELL, PROJECT GEOLOGIST

CHECKED AND APPROVED: D. Purcell, SENIOR GEOLOGIST

SUPERVISING GEOLOGIST

Scale V/H = 1

HORIZONTAL 0 100 200 300 400 Feet

VERTICAL 0 200 400 Feet

TITLE
SUMMARY GEOLOGICAL LOG
CHAINAGE 162+00 to 216+00

PROJECT
TUGGERANONG SEWER TUNNEL

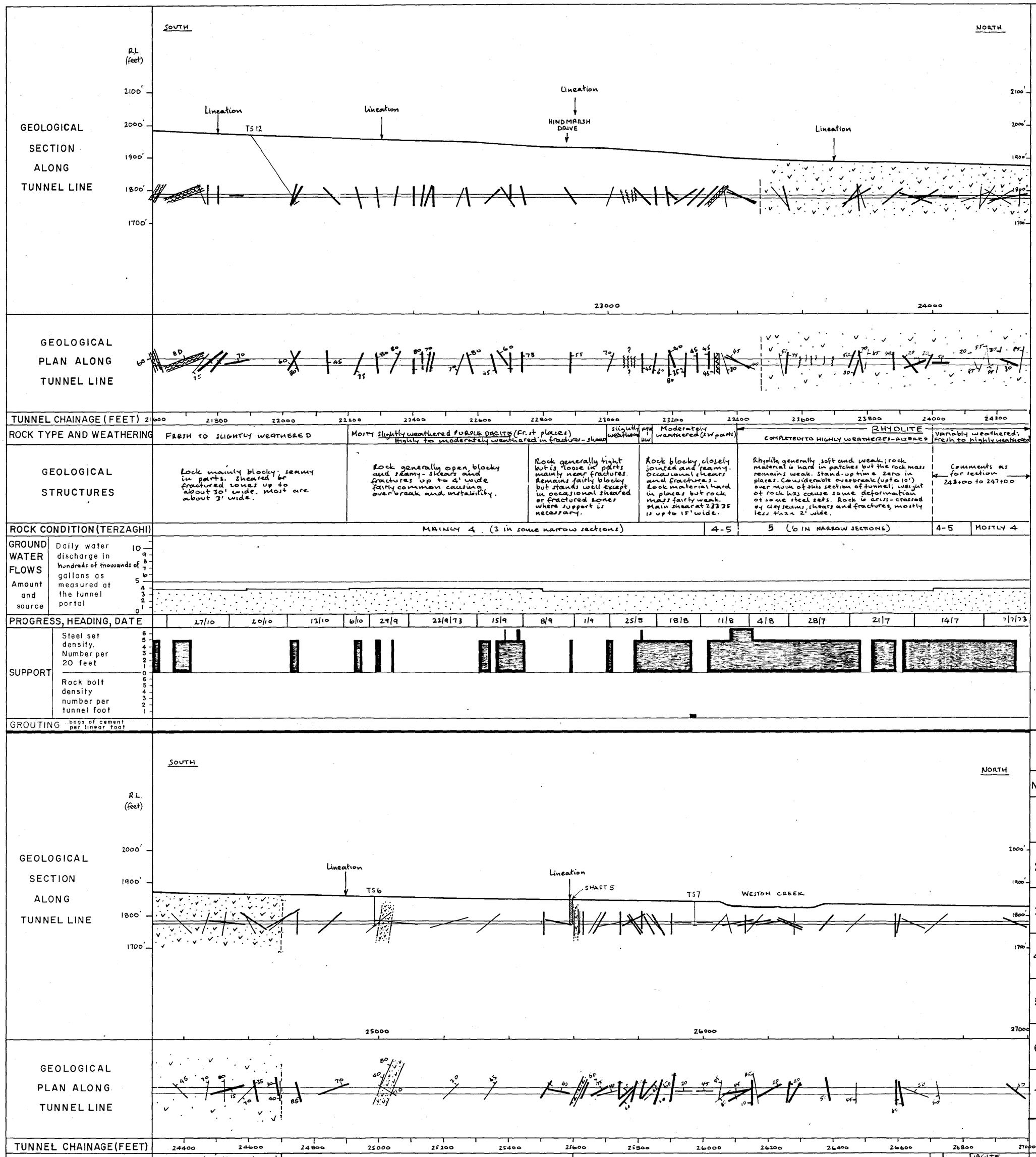
TO ACCOMPANY RECORD

SHEET 4 OF 6

DRAWING NUMBER 155/A16/1847

PLATE 4

M(Pf)144



REFERENCE

DACITE, RHYODACITE

RHYOLITE

SHEARED, FRACTURED OR FAULTED ZONES

LOCATION

Accurate

? - - - - ? Inferred

WIDTH AND ATTITUDE (Attitude in plan only)

30 — 3"-11" wide, showing dip

60 — 12"-5' wide, showing dip

80 — ≥ 5' wide, showing dip

§§§ Zones where many narrow shears (mainly random attitudes) occur.

AMENDMENTS

No	DESCRIPTION	AUTHOR / CHECKED	DATE
1			
2			
3			
4			
5			
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7			

GEOLOGY BY D.C. PURCELL

COMPILED AND CHECKED: D.C. PURCELL, PROJECT GEOLOGIST

CHECKED AND APPROVED: D.C. Purcell, SENIOR GEOLOGIST

SUPERVISING GEOLOGIST

Scale V/H = 1

HORIZONTAL: 0 1·00 2·00 3·00 4·00 Feet

VERTICAL: 0 2·00 4·00 Feet

TITLE
SUMMARY GEOLOGICAL LOG
CHAINAGE 216+00 TO 270+00

PROJECT
TUGGERANONG SEWER TUNNEL

TO ACCOMPANY RECORD

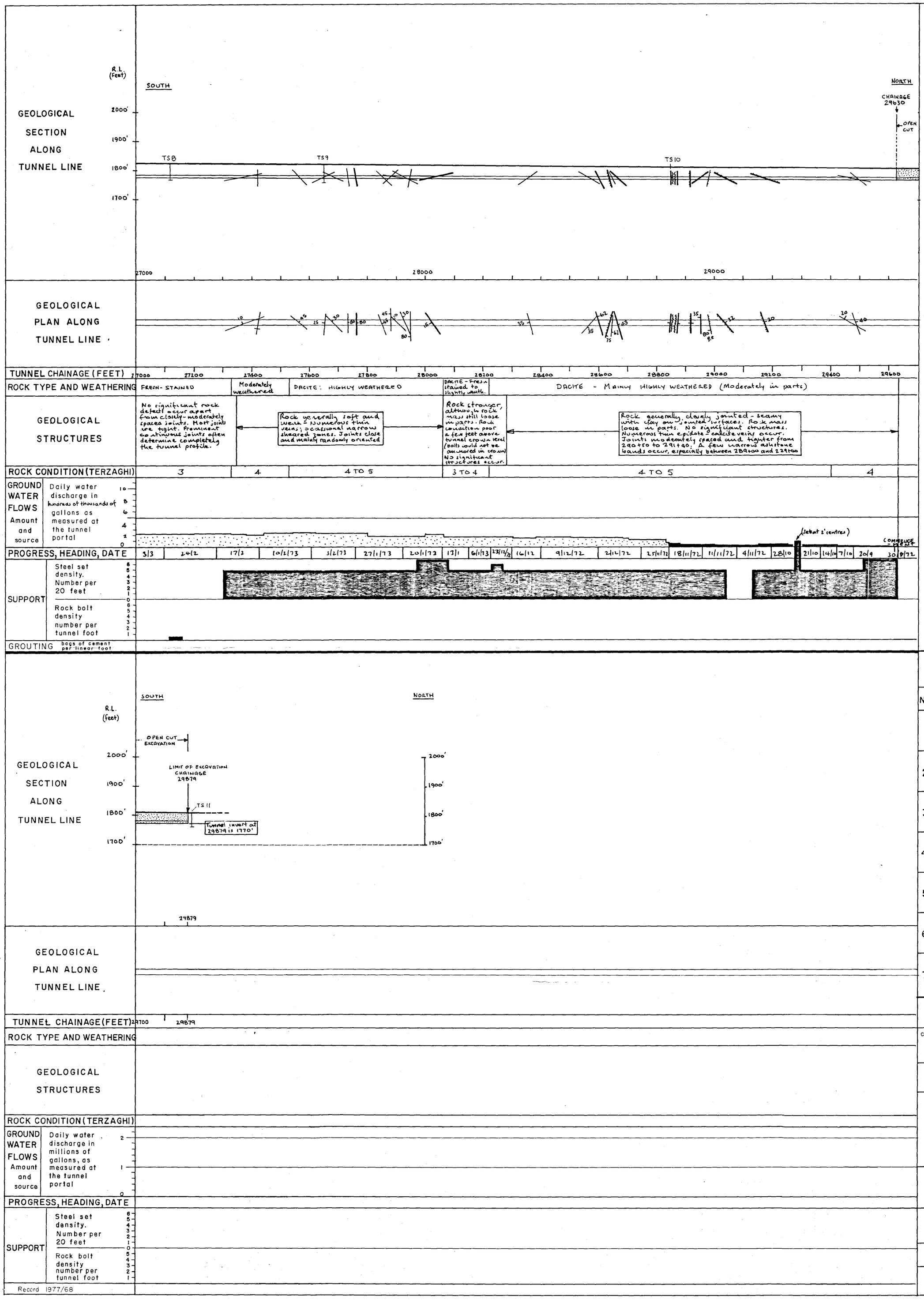
SHEET 5 OF 6

DRAWING NUMBER 155/A16/1847

PLATE 5

Record 1977/68

M(Pf)144



REFERENCE

DACITE, RHYODACITE

SHEARED, FRACTURED OR CRUSHED ZONES

LOCATION

Accurate

Inferred

WIDTH AND ATTITUDE (Attitude in plan only)

3"-11" wide, showing dip

12"-5' wide, showing dip

≥ 5' wide, showing dip

VEINS, DYKES

3"-11" wide, showing dip

12"-5' wide, showing dip

AMENDMENTS

No	DESCRIPTION	AUTHOR / CHECKED	DATE
1			
2			
3			
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7			

GEOLOGY BY D.C. Purcell

COMPILED AND CHECKED: D.C. Purcell, PROJECT GEOLOGIST

CHECKED AND APPROVED: D.C. Purcell, SENIOR GEOLOGIST

SUPERVISING GEOLOGIST

Scale V/H = 1

HORIZONTAL 0 100 200 300 400 Feet

VERTICAL 0 200 400 Feet

TITLE
SUMMARY GEOLOGICAL LOG
CHAINAGE 270+00 TO 298+79

PROJECT
TUGGERANONG SEWER TUNNEL

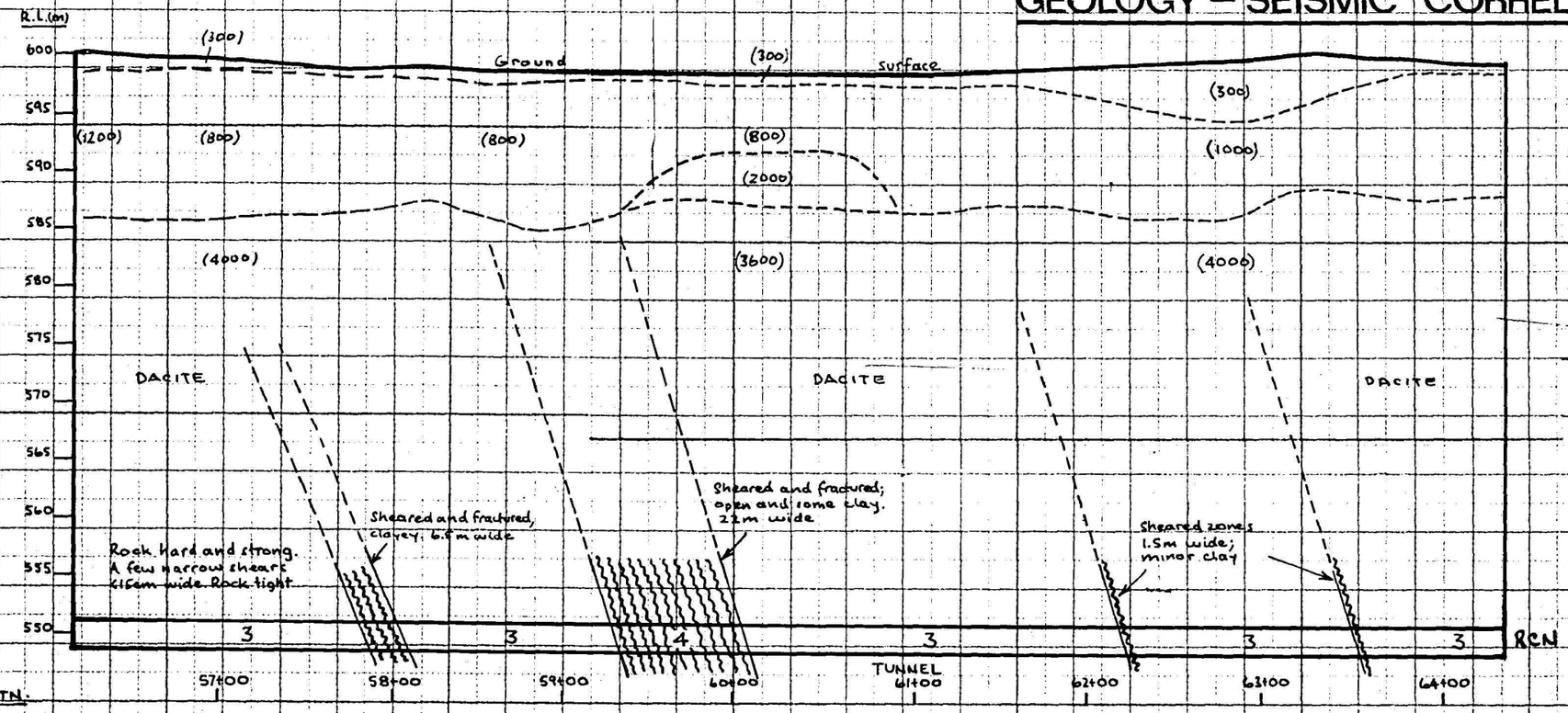
TO ACCOMPANY RECORD: SHEET 6 OF 6

DRAWING NUMBER 155/A16/1847

PLATE 6

GEOLOGY – SEISMIC CORRELATIONS

PLATE 7



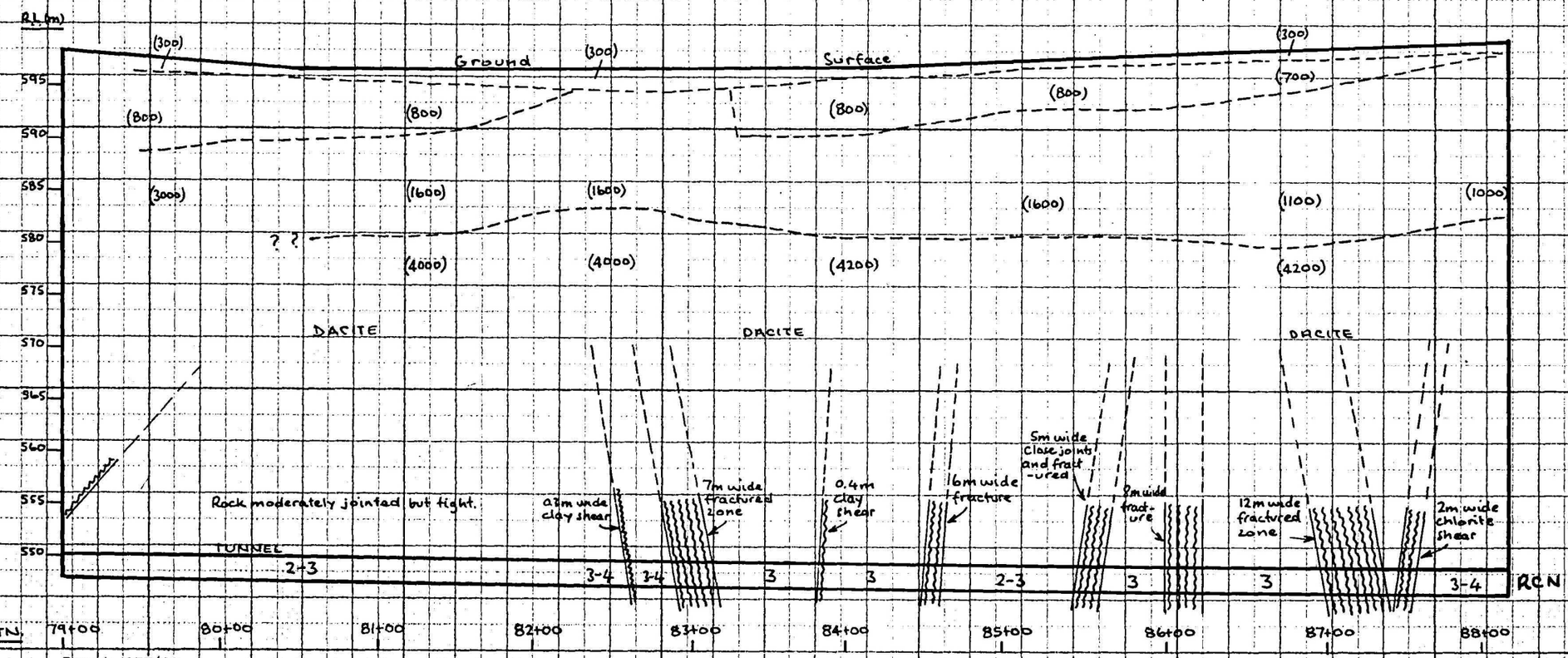
NOTES:

(4000) Seismic velocity in metres/second

– – – – Seismic refractor

Sheared/fractured zone; Rock in vicinity usually seamy and closely jointed.

– Tunnel stations are in hundreds of feet

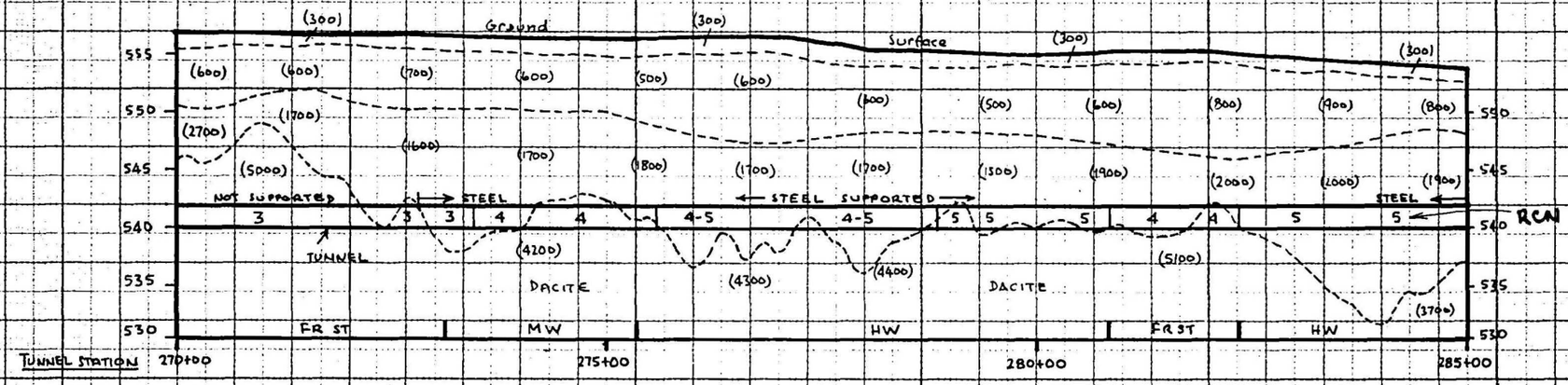


Record 1977/68

155/A16/1848

GEOLOGY – SEISMIC CORRELATIONS

PLATE 8



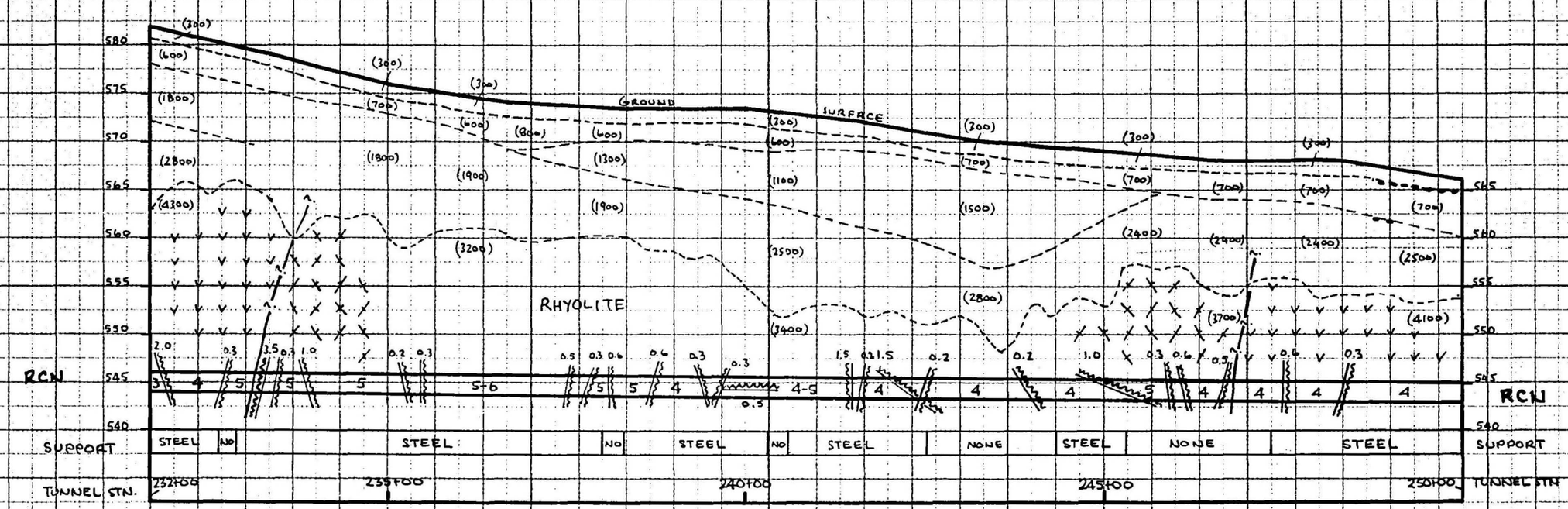
NOTES:

(4000) SEISMIC VELOCITY IN METRES/SECOND

SEISMIC REFRACTOR

0.6 SHEARED ZONE SHOWING WIDTH (m)

TUNNEL STATIONS ARE IN HUNDREDS OF FEET



Record 1977/68

155/A16/1849