Engineering Geological Investigations in Soft Rock Terrain, Poro-o-tarao Tunnel, New Zealand

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1. INTRODUCTION

Construction of tunnels in soft sedimentary rocks and colluvium derived from them can pose problems that are not normally encountered in hard rocks or soils (Prebble, 1977). This was illustrated recently during investigations for the construction of a new 1.3 km, 6 m diameter tunnel at Poro-o-tarao for the North Island Main Trunk Railway. The tunnel replaces an adjacent existing brick-lined structure built in the late 19th century which had become increasingly unreliable due to deterioration of the lining (Webley, 1970). The various investigations undertaken are reviewed, and the effects on construction of the geomechanical characteristics of the materials encountered are described.

2. ENGINEERING GEOLOGY OF PORO-O-TARAO AREA

2.1 General Geology

The tunnel was construc[illegible] thr[illegible] [illegible] which divides the Puputaha Stream, a north flowing tributary of the Mokau River, from the Ohinemoa Stream, a south flowing tributary of the Wanganui River (Fig. 1). The ridge rises to 457 m above sea level, giving a maximum cover above the new tunnel of 115 m. To the north the ground slopes away from the ridge at 15° while to the south the slope is about 25°; further away from the ridge the slope-angles average about 10° and the surfaces are commonly hummocky, indicating that widespread landsliding has occurred in the past. Some ridges in the vicinity are capped by ignimbrite which typically forms distinctive cliffs and steep bluffs.

On the 1:250 000 Geological Map of New Zealand, Sheet 8 (Hay, 1967), the area around the site is included within the "Mahoenui Mudstone" formation of the Mahoenui Group. This group is assigned to the Upper Landon - Lower Pareora Series (Miocene). The rocks have recently been included in the Taumarunui Formation, a new unit established by Nelson & Hume (1977). For the purposes of this paper Mahoenui Group is adequate as a stratigraphic name. Bedding dips from 4° - 18° to the north or northwest. The area is mantled extensively by colluvium, which generally makes bedrock mapping difficult.

2.2 Lithology

The Mahoenui Group lithology is typically fine-grained, consisting predominantly of mudstone (as defined by Folk et al. 1970), with sporadic thinly laminated sandstone beds. Colour banding is a characteristic feature, dark grey bands alternating with light grey green bands up to 10 cm thick. The rock is commonly calcareous and ranges in hardness

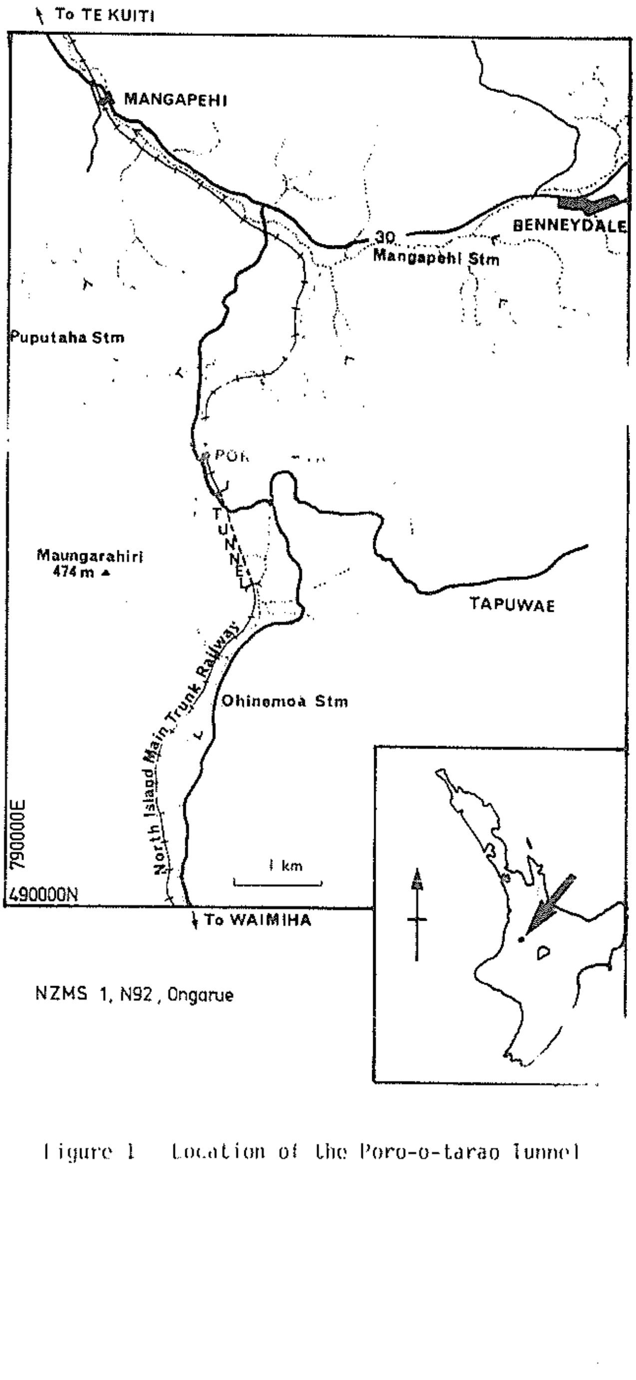

Figure 1 Location of the Poro-o-tarao Tunnel

from soft to moderately hard. Particle size analyses commonly indicate that the mudstone is composed mainly of silt, but thin section and scanning electron microscope studies suggest that the "silt" particles may actually be aggregates of clay, thus petrographically much of the mudstone might be described as claystone.

Typical whole-rock mineralogical determinations by infra-red spectrophotometry are given in Table I.

The overlying colluvium consists mostly of fragments of bedrock in a silt-clay matrix with local organic debris. It varies in grading, water content and thickness over short distances.

2.3 Investigations

Deformation of the lining of the old tunnel, thought to be possibly due to deep-seated landsliding, together with widespread topographic evidence of slope instability, indicated that careful evaluation was required to determine the influence that landsliding could have on the proposed tunnel project. Engineering geological investigations to select a suitable tunnel alignment began with aerial photograph inspection and surface mapping, and was followed by the drilling of 35 N-size cored drillholes which were concentrated around the portals and southern approaches. Diamond tipped drill bits tended to clog in both the mudstone and colluvium. The most successful core recovery was achieved using tungsten-carbide tipped bits with double or triple, split-inner core barrels (NMS or NMLC). The core retrieved contained many fine crushed and/or thin soft gouge-like zones which at first appeared to confirm the theory that the bedrock had been affected by deep-seated landsliding.

The next stage in the investigations was the excavation of a 50 m-long drive in the north portal area, to experiment with lining systems and to permit comparison of core data with in situ conditions. It became evident that the crushed zones were much less common than previously thought and that unsuitable drilling techniques were responsible for most of the defects visible in the cores. From these observations it was concluded that landsliding within bedrock was unlikely to cause problems in the construction and long term maintenance of the new tunnel. Furthermore, at Poro-o-tarao local flat areas were thought to possibly represent slumped blocks but studies beyond the site show that this is a typical geomorphic feature of Mahoenui terrain (Marwick 1946, Chandler 1978), marking changes in lithology.

Investigations then concentrated on delineating the thickness, extent and geomechanical properties of the overlying colluvium in the portal areas, especially at the south end of the tunnel. The difficulty in obtaining small diameter core representative of in situ conditions was a particular problem in this phase of the investigations. Seismic refraction surveys and penetrometer soundings to locate the bedrock-colluvium transition zone were also inconclusive. The most successful investigative tool proved to be 1 metre diameter Caldweld shafts which allowed in situ inspection of subsurface conditions. At the south portal shaft inspection identified the heterogeneous nature of the colluvium and enabled sampling for laboratory testing. The degree of weathering, water content, clay fraction, rock fragment size, interface contact and depth to mudstone-colluvium interface all varied considerably. It was shown that the thickness of colluvium was as great as 20 m, and that the colluvium-mudstone contact was either abrupt, or transitional through a zone of blocky bedrock.

However the site investigations at the north portal did not detect a colluvium-filled channel that intersected the tunnel between construction distances 98 m and 112 m. This feature was identified during installation of the north portal settlement gauges after tunnel construction had commenced thus emphasising the necessity for continual site assessment throughout construction.

2.4 Geomechanical Characteristics

A summary of selected properties of Mahoenui Group rocks and its colluvium is given in Table II.

3. CONSTRUCTION CHARACTERISTICS OF COLLUVIUM AND BEDROCK

3.1 Colluvium

The extent and marginal stability of the colluvium at the southern portal lead to the design of stabilization measures consisting of a contiguous piled concrete retaining structure founded into bedrock and a series of vertical sand drains linked to a 300 m drainage drive to lower piezometric levels upslope (Parton 1974). As excavations proceeded bedrock profiles inferred from interpolation between borehole and shaft data required modification. This changed the input data for slope stability analyses and hence cut slope design was continually reassessed and modified as further geologic information became available (Ramsay 1980). A combination of wet weather and local overstressing at the toe of a cut slope initiated some progressive slope failures which resulted in careful control of the rate and method of subsequent excavation of this inherently unstable material.

Due to the variability of the colluvium in situ, the long term effectiveness of the sand drains in dewatering the south portal slopes is as yet uncertain. However short-term observations of Geonor and standpipe piezometers indicate that, allowing for seasonal fluctuations, overall groundwater levels are being lowered (Fig.2). Significant

TABLE I

WHOLE-ROCK MINERALOGY
(Analyst: Mr C.W.R. Soong)

Tunnel Distance (m)	Qtz	Plag.	Calcite	Illite	Montmorillonite	Kaolinite
670	20	8	7	14	42	3
1350	20	6	9	15	44	2

TABLE II
SUMMARY OF ENGINEERING GEOLOGICAL PROPERTIES

	Mahoenui Group	Colluvium	Base of the Colluvium
Lithology	Mudstone with minor sandstone beds	Mudstone fragments in a silt-clay matrix	Clay with rare mudstone fragments
Colour	Dark grey/grey green	Blue-grey to yellow-brown	Light grey
Weathering	Unweathered	Moderately to highly weathered	Moderately weathered
Hardness	Soft - mod. hard (calcareous bands hard)	Very soft	Very soft
Consistency		Firm to stiff with medium plasticity	Very soft to firm
Bedding (i) Bedding Plane separation	< 0.1 mm	-	-
(ii) Dip	Between 4^{o} - 18^{o} N to NW	-	-
Defects (i) Sets	2 plus random	-	-
(ii) Spacing	0.3 to 1.0 m	-	-
(iii) Continuity	2.0 to 3.0 m	-	-
(iv) Amplitude of Waviness	2 - 4 cm	-	-
(v) Surface Roughness	Slightly rough	-	-
(vi) Fracture separation	Mainly < 0.1 mm Always < 1 mm	-	-
(vii) Water conditions	Generally dry	Dry to moist	Moist to wet
Uniaxial Compressive Strength	Average: 6 MPa	-	-
Point-Load Index	0.1 - 0.3 MPa	-	-
Schmidt Hammer Rebound No.	Average ⊥ Bedding = 38 Average II Bedding = 23	- -	-
NCB Cone Indenter No.	0.63 - 0.88	-	-
Angle of Shearing Resistance and Apparent Cohesion			
Triaxial ⊥ Bedding	27 - 58^{o}, 300 - 2000 kPa	-	31^{o} - 37^{o}, 10 - 20 kPa
Direct Shear ⊥ Bedding	58^{o}, 1200 kPa	-	-
II Bedding	45^{o}, 1000 kPa	-	-
Remoulded Residual Strength	-	-	$\phi' = 16^{o}$
Remoulded Cohesion	-	-	C' = 0 kPa
Slake Durability (2nd cycle %)	Hard sandstone bands: 84 - 96 Mod. hard mudstone: 16 - 25	-	-
Bulk Density	1.92 - 2.22 tonnes/m^3	1.65 tonnes/m^3	1.75 tonnes/m^3
Particle Size (Average values) Clay %	22 - 38 (35)	17 - 37 (26)	
Silt %	55 - 78 (60)	48 - 70 (65)	
Sand %	0 - 10 (5)	0 - 20 (9)	
Clay Mineralogy	See Table I		
Moisture Content % (Average Values)	8 - 12 (11)	21 (29)	-
Plasticity Index (Average)	22	28	-
Mean Plastic Limit (No. samples)	22 (57)	27 (16)	-
Mean Liquid Limit (No. samples)	44 (57)	55 (16)	-

reductions of localised water-table levels were achieved by well point pumps during the excavation and concreting of piles for the south portal retaining structure (Fig. 2).

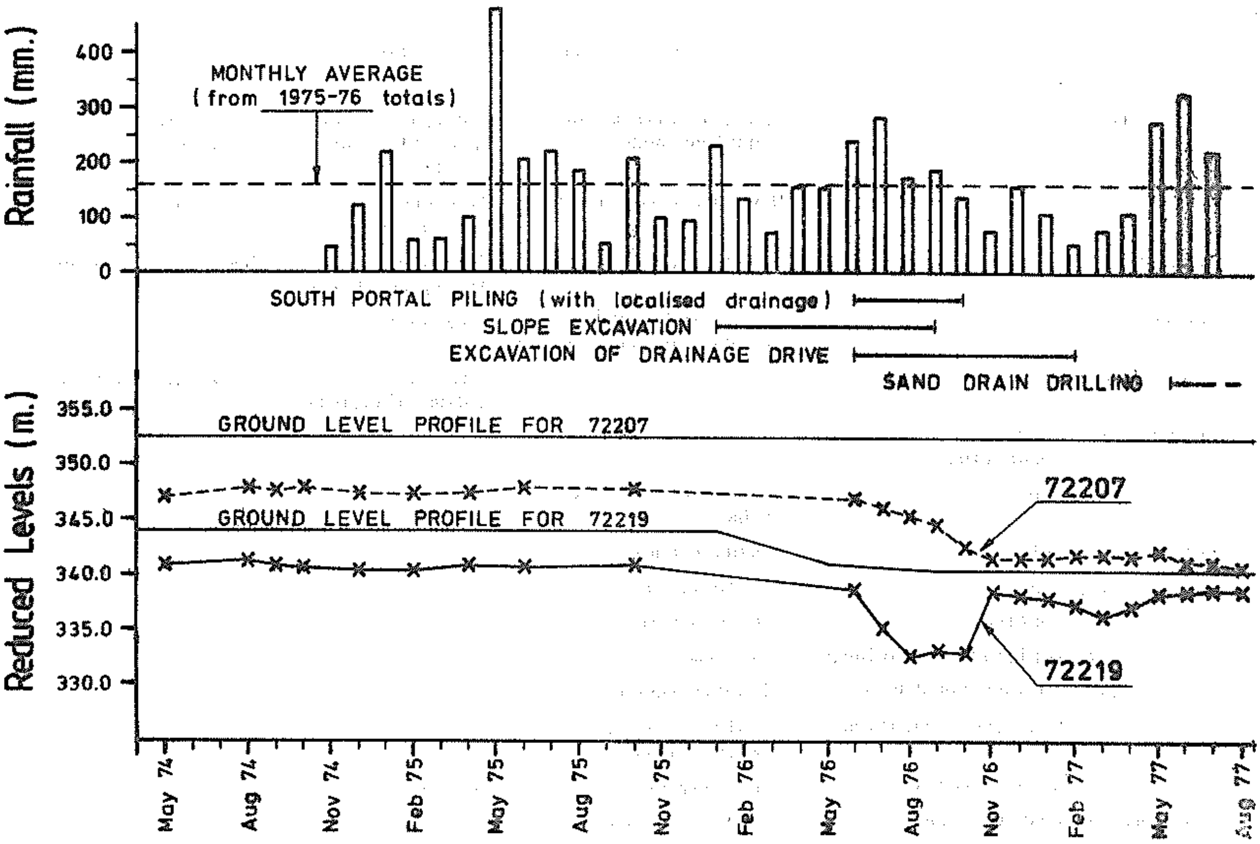

Figure 2 Groundwater levels as recorded in south portal geonor piezometers

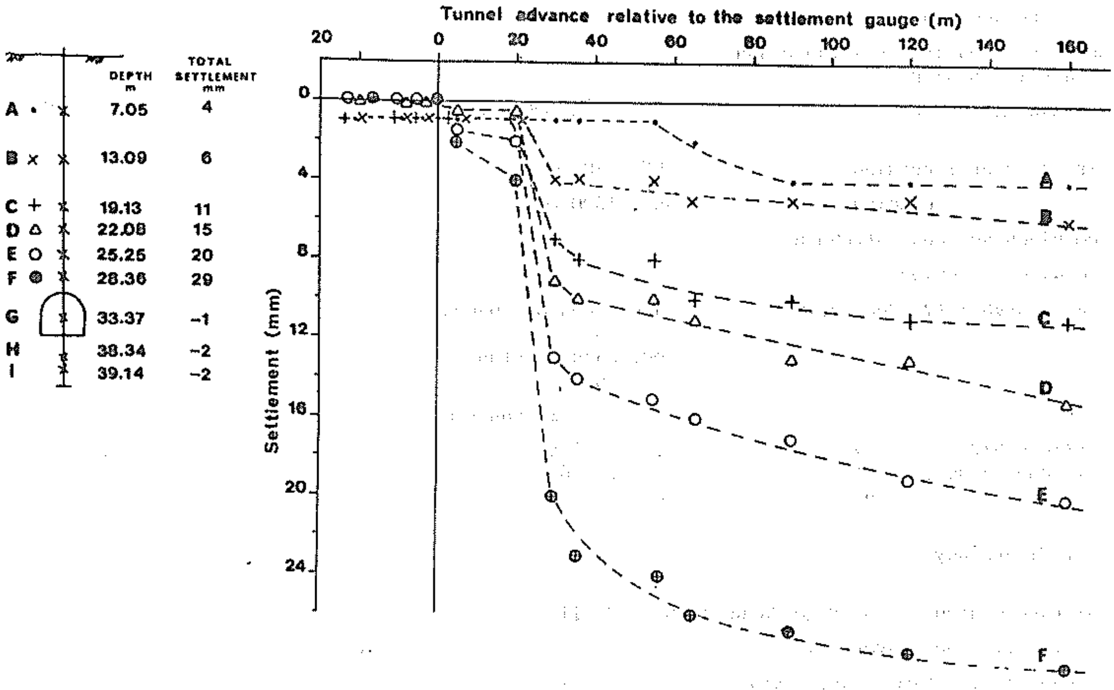

Figure 3 North portal borehole settlement gauge at tunnel construction distance of 191 m

Near the north portal the tunnel intersected the base of a trough of colluvium which reduced full face excavation to heading and benching. Prelining and spilling support ahead of the excavation face were required (Borrie, 1977).

3.2 Bedrock

The response to excavation of soft rock may be controlled both by the intact material properties and rock mass defects. The influence that rock mass defects had on excavation was minimised by the use of a Dosco Roadheader rather than using drill and blast techniques. The rock was noticeably harder towards the middle of the tunnel but this did not affect construction.

Overbreak was low compared to the conventionally excavated drainage drive, due to less peripheral rock mass disturbance. The overbreak that did develop was caused by randomly orientated joint sets intersecting subhorizontal bedding defects resulting in slabbing in the crown and the occasional large wedge failure at the face. The former required extensive use of steel mesh installed between the sets to stop slabbing of small blocks.

Settlements of up to 30 mm occurred in the crown with perceptible movements being detected 2 - 3 tunnel diameters away. The most significant settlements occurred at tunnel construction distance of 191 metres when the tunnel face had been advanced a further 3 - 4 tunnel diameters past the multi-position magnetic borehole settlement gauge (Fig. 3). The movements are attributed to downward movement of bedding slabs, resulting in a zone of loosened material developing above the tunnel crown. Heavy rain may have accentuated the process.

Predictions of support requirements using rock mass classification schemes (Wickham, Tiedmann and Skinner, 1974, Bieniawski, 1975, Barton, Lien and Lund, 1975) proved to be reasonably reliable (Rutledge, 1977, Borrie, 1977). Limitations are evident when predicting support requirements for smaller openings and for tunnels in softer sedimentary rocks where rock mass breakdown occurs. However the use of an active, rock mass sealing, support system such as shotcrete applied immediately after excavation rather than the conventional passive steel sets, would have eliminated much of the fretting and bedding plane slabbing that occurred in the tunnel crown.

A factor that was not anticipated during the investigations was the large quantity of dust generated by roadheader excavation. Initially dust caused industrial problems due to the health risk that it imposed. The tungsten carbide cutters liberated dust particles of which about 10% were quartz fragments within the respiratory range (< 5 μm). Formation of dust size particles depends on two main factors; grain size of the rock, and its mineralogy. The dust problem with Mahoenui Group rocks is probably the result of very fine grain size coupled with relatively high quartz-feldspar content (Dr G.A. Challis, personal communication). The problem was alleviated using steam and/or water dust suppression units and extraction fans near the face.

Little water was encountered in the tunnel but the combination of water resulting from dust suppression and rolling stock caused the invert to disintegrate to a muddy slurry. When tunnel advance was slower the problem compounded as construction machinery was more concentrated within the affected area. The Mahoenui Group rocks are prone to shrinkage and fretting on drying. Simple experiments show that a block of mudstone will break down to chips and friable particles when left outside for a few days. The effect underground is to cause unlined tunnel walls to fret and hence, with time, leading to increased overbreak. A sealing primary support system would have eliminated this problem.

4. CONCLUSIONS

Various difficulties with subsurface investigations in Mahoenui Group rocks and its overlying colluvial weathering products and related engineering problems encountered during the construction of the new Poro-o-tarao tunnel included:

- Recovery of small diameter drill core representative of subsurface _in situ_ conditions was difficult. Core drilling _in_ soft rock terrain should therefore be interpreted with caution, at least until the technique is proved reliable.
- The more meaningful investigative methods were those which allowed _in situ_ inspection, such as large diameter bucket auger holes.
- Extension of field studies beyond the site which has limited outcrop proved worthwhile.
- Because of the variable nature of the colluvium care was needed in selection of geomechanical design parameters and excavation techniques.
- Bedding plane failures migrated above the tunnel crown to cause settlement when active support systems were not installed soon after excavation.
- Machine excavation resulted in neater excavation profiles but excessive dust was liberated by the cutter header action.
- Invert breakdown due to rolling stock and wet conditions often caused construction difficulties. Concrete lined drains and sump holes would have been necessary to channel water away from the face and invert effectively.
- Shrinkage on drying causes fretting, leading to overbreak in unlined tunnel walls. Sealing primary support systems can eliminate this problem.

These factors will not necessarily be relevant to other Tertiary soft rocks but provide guidelines as to the kinds of investigations which may be appropriate for civil engineering projects in similar materials.

5. ACKNOWLEDGEMENTS

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