AN ENGINEERING GEOLOGICAL INVESTIGATION
OF GROUND SUBSIDENCE ABOVE THE
HUNTLY EAST MINE AREA

A thesis
submitted in partial fulfilment
of the requirements for the Degree
of
Master of Science in Engineering Geology
in the
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by
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"The process of coal mining, as indeed the mining of any stratified mineral, is inevitably followed by some degree of sinking of the superjacent strata and consequently of the surface"

Ground subsidence above the Huntly East Mine at the N.Z.E.D. Hostel has affected an area of approximately seven hectares with measured settlements of over 800mm. Extensive damage was suffered by most buildings and services of the hostel complex (a $2M development) with remedial measures being estimated at approximately $450,000.

To determine the cause(s) and mechanism(s) of the subsidence, site and laboratory investigations were undertaken. Site investigations included core and wash drilling, geophysical borehole logging, dutch cone penetrometer soundings plus piezometer installation and minotoring. Laboratory investigations included one dimensional consolidation and permeability testing, SEM fabric studies, XRD and chemical tests for clay mineralogy, and determinations of Atterberg Limits and grain size distributions.

The mine overburden geology at the site consists of a 35 to 60m thick sequence of mudstones and coal seams of the Te Kuiti Group (Eocene to Oligocene), and overlain by a 50 to 70m thick succession of saturated sands, silts and gravels of the Tauranga Group (Pliocene to Holocene). Within the Tauranga Group three aquifers are present. Drilling, combined with laboratory testing has defined a 4 to 10 metre thick, laterally extensive, highly compressible ignimbritic silt aquitard above the lowest aquifer.

The engineering geological model considered most likely to explain the subsidence is mine roof collapse causing void migration to near the top of the Te Kuiti Group sequence resulting in drainage and depressurising of aquifers at the base of the Tauranga Group. Aquifer depressurisation is considered likely to cause consolidation within both the aquifer and aquitards associated with it.

Back-analyses of the dewatering consolidation model in terms of both magnitude and rates of settlement are consistent with observed values. A finite difference numerical analysis was developed for estimations of settlement rates.

Further field verification of the dewatering consolidation model requires monitoring of piezometers in areas of potential ground movement and inspection of workings under subsided ground.
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NOTATIONS

\( a_v \) coefficient of compressibility
\( C \) constant of compressibility
\( C_c \) compression index
\( c_v \) coefficient of consolidation
\( e \) void ratio
\( k \) coefficient of permeability
\( m_v \) coefficient of volume compressibility
\( n \) porosity
\( N \) newton
\( Pa \) pascal
\( q_c \) cone resistance
\( f_s \) sleeve friction
\( w \) water content
\( w_l \) liquid limit
\( w_p \) plastic limit

\( \rho \) density of soil
\( \rho_s \) density of solid particles
\( \rho_w \) density of water
\( \rho_{sat} \) density of saturated soil material
\( \rho_b \) buoyant density
\( \rho_d \) density of dry soil material
\( \bar{c} \)

preconsolidation pressure

\( \bar{\sigma} \)

effective intergranular stress

\( \sigma \)

total stress

\( \mu \)

pore water pressure (neutral stress)
ACKNOWLEDGEMENTS

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I would like to thank my wife Liz, for her continual support and encouragement throughout the course of this study.
CHAPTER I

INTRODUCTION

1.1 Subsidence at Huntly

Ground movement at the N.Z.E.D. (New Zealand Electricity Division) Hostel, Burke Place, Huntly was first noticed in early January, 1983. Within two months the area affected had extended to five hectares with settlements\(^1\) up to 550mm. In April 1983 the affected area had enlarged to approximately seven hectares with maximum settlements of 800mm.

The subsidence caused extensive damage to buildings and services of the hostel (a $2M development) with lesser damage to nine private homes. Williams (1985) estimates remedial measures for the hostel and homes to cost approximately $450,000 and $14,000 respectively. These figures do not include the decline in capital value of an adjacent 30 section subdivision, or the loss of accommodation in the hostel.

Since the hostel subsidence two more areas of significant ground movement have occurred in the Huntly Borough over the East Mine. Settlements up to 1140mm have been recorded in these areas.

1.2 Thesis Objectives

This study was initiated as a result of recommendations made by S.C.M. (State Coal Mines) and M.W.D. (Ministry of Works and Development) in preliminary reports addressing the subsidence problem (Deplege 1983a; McInally, 1983a and Williams, 1983).

Thesis objectives are:

1) to determine the cause(s) and mechanism(s) of subsidence above the south headings of the Huntly East Mine.

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1. See Appendix One for definitions of settlement and subsidence in this thesis.
2) to develop an engineering geological model for the subsidence.

3) to provide a numerical analysis of the model where calculated and observed settlements are compared.

4) to define implications of the study with respect to existing and future coal mining in the area.

Although this study focuses on the first case of significant subsidence above the Huntly East Mine, observations from more recent occurrences are incorporated for the overall analysis.

It is beyond the scope of this study to propose or detail preventative or remedial measures for the subsidence problem.

1.3 Investigation Methodology

The approach adopted for the subsidence investigation is illustrated in Figure 1.1. This objective-oriented procedure, originally developed by an International Society of Rock Mechanics Commission (I.S.R.M., 1975), has been widely applied to site investigations (e.g. Stapledon, 1979; Bell and Pettinga, 1983) and was considered to be a suitable framework for this study.

1.4 Local Setting

1.4.1 Site Location

The Huntly Borough is situated on the banks of the Waikato river 80km south of Auckland (Figure 1.2). The N.Z.E.D. Hostel is part of the Kimihia subdivision located in Huntly East. The N.Z.E.D. Hostel subsidence is the most southerly of the three subsidence areas with settlements greater than 100mm above the Huntly East Mine.
Arrows indicate iterative feedback process to solve additional problems found during main investigation program.

Figure 1.1: Objective orientated approach to site investigations (based on I.S.R.M., 1975; Bell and Pettinga, 1983). This conceptual framework has been adopted for the subsidence investigation.
Figure 1-2: Locality map showing the extent of the Huntly East Mine workings and areas of subsidence.
1.4.2 Coal Mining

The Huntly East mine was opened in 1977 and is presently the largest producing coal mine in New Zealand with an annual production of approximately 300,000 tonnes. From the worked out Kimihia Opencast, East Mine drives extend west under the Huntly township (Figure 1.3a). The area north of the drives, known as the northern headings is presently being developed for longwall mining which is due to commence in 1987.

The south headings, which extend under the Huntly township, are currently mined using the room and pillar method. South headings development involves the excavation of main access and service drives to the southeast with a later retreat phase, extracting the coal in a series of panels. Original design dimensions of room and pillar workings were drives 5m wide and 3.5 or 6m high, depending on floor coal extraction. Pillars 15m x 15m were left for roof support. Since the hostel subsidence, the pillar size has been increased to 15m x 20m, with an offset to reduce roof spanning distances. A plan of the south headings, showing the numbering of panels and different room and pillar geometries is shown in Figure 1.3b.

The panel 1 workings under the N.Z.E.D. Hostel are 100 to 110 metres below the surface. Mining took place in this area between January and August 1982.

1.4.3 Urban Development

Stage 3 of the Kimihia subdivision, which includes the hostel was developed in the late 1970s by Murray North Partners Ltd. and the M.W.D. (Ministry of Works and Development). Development of the originally low lying poorly drained land involved (see Figure 1.4):

1) removal of peat and other unsuitable foundation materials to reserve areas.

2) filling with approximately 1.0-1.5m of sand to ground water level.
Figure 1.3a: Huntly
East Mine Workings

Figure 1.3b: Mine Workings of the Southern Headings, Panels 1 to 4, Huntly East Mine
FIGURE 1.4: KIMIHIA SUBDIVISION DEVELOPMENT
3) filling with 1.0-2.5m layer of recompacted coal measures mudstone from the adjacent Kimihia Opencast to the 'design flood level' of 11.5 m.a.s.l.

4) covering of mudstone with approximately 20cm of sand to aid drainage which was in turn covered by a veneer of top soil.

5) tapping of natural springs at ground level and drainage into the stormwater system (pers. comm. J. Kendrick, 1984 - M.W.D. supervisor involved in Kimihia Block development).

Most hostel buildings and residences affected by subsidence in the area are single storey brick veneer structures (Figure 1.5). Cracking of brick veneer through differential settlement in houses in Rosser Street was reported soon after construction but before mining commenced. This is interpreted by Williams (1983) as being caused by consolidation settlement due to fill emplacement.

Foundation works at the hostel consisted of minor site reggrading. Springs evident in pre-development aerial photographs (Section 3.2.1), on slopes below the hostel, have been piped into the stormwater system (pers. comm. J. Kendrick, 1984).

1.5 Regional Setting

1.5.1 Introduction

This review section is based on the 1:250,000 geological maps of Schofield (1967) and Kear (1960) plus New Zealand Geological Survey (D.S.I.R.) Bulletins of Kear and Schofield (1978) and Schofield (1972). Aspects of East Mine geology and hydrology are summarised from S.C.M. mine plans and assessments presented by B.M.C. (1984) and Todd (1982b). The primary objective of this review is to define aspects of regional geology and geomorphological development relevant to subsidence problems above the south headings.
Figure 1.5: Northeasterly view of Rosser Street housing with N.Z.E.D. Hostel on hill in background (Photograph J.R. Pettinga).
1.5.2 Regional Geology

1.5.2.1 Stratigraphy

The stratigraphy of the Waikato area consists of Triassic to Jurassic basement rocks of the Hokonui System, Tertiary strata of the Te Kuiti and Waitemata Groups, plus Pliocene to Holocene un lithified deposits of the Tauranga Group (Figures 1.6 and Table 1.1).

Hokonui rocks are part of the extensive North Island Mesozoic basement. Typical lithologies are moderate to highly indurated sandstone and siltstone. The Hakarimata Formation, the basal unit of the Hokonui system in the area, forms the base ment at the Huntly East Mine. Within the mine, the top of the basement is marked by a clay rich weathering zone typically 5 metres but ranging from 0.5 to 20 metres thick (pers. comm. R. Gregg, 1984 - S.C.M. geologist). The weathering zone grades into the underlying highly indurated siltstones and sandstones.

Unconformably overlying the basement, and common throughout the North Island, is the Tertiary sequence which includes basal coal measures, mudstones, sandstones and limestones. The basal Waikato Coal Measures comprise a c.60m sequence of mudstones, sub-bituminous coals and rare fine sandstones and siltstones (B.M.C. 1984). King (1978) describes the dominant lithologies as moderately indurated, dark grey to greyish brown slightly carbonaceous mudstones and brownish black carbonaceous mudstones. King notes that the fine to medium quartzose sandstones and coarse siltstones occur in isolated beds up to 3 metres thick. The presently mined Kupakupa seam is typically 7 to 8 metres thick with the overlying Renown seam 1 to 5 metres thick. Descriptions of lithologies from the remainder of the Tertiary sequence are presented in Table 1.1.

Common throughout the Waikato, Auckland and Bay of Plenty regions are the un lithified, Pliocene to Holocene, predominantly pumiceous deposits of the Tauranga Group. Within the Huntly region the Tauranga Group unconformably overlies the Tertiary strata, and consists of a sequence of fluvial, lacustrine and pyroclastic deposits (Kear and
<table>
<thead>
<tr>
<th>STRATIGRAPHY</th>
<th>AGE</th>
<th>APPROXIMATE THICKNESS</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Taupo Pumice Holocene</td>
<td></td>
<td>0 - 30m</td>
<td>- loose, creamy white, highly pumiceous silt, sand and sandy conglomerates commonly containing charcoal and bedded</td>
</tr>
<tr>
<td>Hinuera Formation</td>
<td></td>
<td>0 - 100m</td>
<td>- yellow brown silts, sands gravel and peat, highly pumiceous and commonly cross-stratified</td>
</tr>
<tr>
<td>Hamilton Ash</td>
<td></td>
<td>0 - 3m</td>
<td>- multilayered sequence of halloysitic yellow brown pinkish grey, firm,blocky, ashes (sandy silty clays) with prominent white veins</td>
</tr>
<tr>
<td>Kauroa Ash</td>
<td></td>
<td>0 - 4m</td>
<td>- multilayered sequence of halloysitic, brown, yellow and pink commonly mottled ashes (clay rich) with manganese nodules</td>
</tr>
<tr>
<td>Karapiro Formation</td>
<td>Pleistocene</td>
<td>± 30m</td>
<td>- brown and yellow brown, pumice-rock fragment (volcanic and greywacke) -quartzfeldspar sands and coarse gravelly sands, with some interbedded white/brown silty clays and peats</td>
</tr>
<tr>
<td>Waeranga Gravels</td>
<td>-</td>
<td></td>
<td>- highly weathered greywacke gravels</td>
</tr>
<tr>
<td>Puketoka Formation ¹</td>
<td>-</td>
<td></td>
<td>- pure pumiceous, sands and silts, breccias and distal deposits of ignimbrite flows interbedded with peats</td>
</tr>
<tr>
<td>Whangamarino Formation</td>
<td>Pliocene</td>
<td>± 30m</td>
<td>- green and brown fine silts and clays, with interbedded rare peat and common to abundant green, quartzfeldspathic rock fragment-pumice sands and coarse gravelly sands</td>
</tr>
<tr>
<td>Whaingaroa Siltstone</td>
<td>+ 80m</td>
<td></td>
<td>- massive grey, moderately calcareous siltstone</td>
</tr>
<tr>
<td>Glen Massey Formation</td>
<td>± 15m</td>
<td></td>
<td>- glauconitic fine to medium sandstone (Glen Massey Sandstone) with lenses of calcareous siltstone (Elgood Limestone) at base</td>
</tr>
<tr>
<td>Mangakotuku Siltstone</td>
<td>± 80m</td>
<td></td>
<td>- non calcareous, massive siltstone-silty claystone with minor calcareous beds</td>
</tr>
<tr>
<td>Pukemiro Sandstone</td>
<td>± 5m</td>
<td></td>
<td>- glauconitic sandstone</td>
</tr>
<tr>
<td>Glen Afton Claystone</td>
<td>± 30m</td>
<td></td>
<td>- non-calcaceous silty claystone</td>
</tr>
<tr>
<td>Waiato Coal Measures</td>
<td>± 60m</td>
<td></td>
<td>- kaolinitic, light brown mudstones to highly carbonaceous fissile mudstones with coal seams towards the base. Siderite concretions occur throughout</td>
</tr>
<tr>
<td>Hakarimata Formation</td>
<td>Rocene</td>
<td>-</td>
<td>- highly indurated siltstone and sandstone</td>
</tr>
</tbody>
</table>


¹The Puketoka Formation is absent from the East Mine Area (Todd, 1982).
Schofield, 1978; Todd, 1982a).

Todd (1982b), as part of a groundwater investigation recognised the Whangamarino Formation, Karapiro Formation, Waeranga Gravels, Hinuera Formation and Taupo Pumice Alluvium within the Tauranga Group above the East Mine. The Puketoka Formation is notably absent. Volcanic tephras mantling low hills surrounding Huntly (pers. obs.) are considered to be part of the Kauroa and Hamilton Ash deposits described within the Lower Waikato Basin by Ward (1967).

1.5.2.2 Structure

Normal faults within the basement and Tertiary sequences in the Huntly area are numerous and define coal field sector boundaries (Figure 1.6b).

The East Mine is bounded to the east and west by 2 normal subparallel N-S trending faults c.1km apart (Figure 1.7). Within the workings a second set of NE (050°-070°) striking normal faults with variable throws of less than 5 metres can be found. S.C.M. studies (B.M.C.,1984) show that regional face cleat orientation ranges from 040° to 070° with butt cleat from 120° to 150°. Cleat intensity is highest adjacent to the NE trending faults and crush zones.

Tertiary strata thickness is highly variable (Figure 1.7) due to the gentle basement relief, the 5° to 15° NW sequence dip and moderate relief on the Te Kuiti-Tauranga Group erosional contact. For the upper contact over the East Mine, cross-sections constructed from S.C.M. mine plans (based on a 200 metre borehole spacing) show c. 30 metres of relief over a distance of c.300 metres (Section B-B', Figure 1.7). In the Weavers Opencast area where the borehole spacing is c.40m. the Te Kuiti-Tauranga Group contact shows 20 to 40 metres of relief over a distance of 50 to 100 metres (Henderson, 1983).

S.C.M. cross-sections (Todd, 1982c) show that the Tauranga Group deposits are commonly channelised with lateral and vertical lithological variations over short distances.
Huntly East Mine Cross-sections

Legends:
- Unnamed Holocene alluvium (pa), Taupo Pumice Alluvium (pm), and Hinuera Formation (ph) (Formations not distinguished on cross-sections)
- Karapiri (ka), Puketea (tp) and Whanganui Formations (tf) (Formations not distinguished on cross-sections)
- Whangararoa Silstone (kw), Glen Massey Formation (kg), Mangatuku Silstone (km), Pukemiro Sandstone (kt), Glen Afton Claystone (ko) and Waikato Coal Measures (kh)
- Hokarimata Formation (nh) — NEWCASTLE GROUP

(for detailed lithological descriptions see Table 1-1)

Figure 1.7: Huntly East Mine Cross-sections

Fig 17
1.5.3 Regional Geomorphology

The present gross landscape features (Figure 1.8) are controlled by block faulting of Tertiary and basement strata with subsequent partial burial by the Tauranga Group sequence (Selby, 1982).

The most prominent landform features are the steep and rugged Hakarimata and Taupiri Ranges with up to 300 metres of relief. These ranges, comprised of basement rocks trend SW-NE and immediately south of Huntly are cut through by the Waikato River in the Taupiri Gorge.

By contrast, the Tertiary strata, form a moderately undulating landscape with broad stream valleys. Coarser and more calcareous units form bluffs up to 30 metres high. Hill sides of Tertiary mudstones are commonly hummocky due to slope instability.

The Huntly township is situated on the edge of the extensive Lower Waikato Lowland. The present geomorphic features of the lowland are the result of cyclic phases of aggradation and degradation (Kear and Schofield, 1978). The low rolling hills which rise above the lowland represent the oldest known phase of major aggradation with the deposition of the Whangamarino, Puketoka and Karapiro Formations plus the Waeranga Gravels (Pliocene to Pleistocene). This sequence was subsequently eroded before the deposition of the Kauroa and Hamilton Ash deposits (Selby, 1982) and Hinuera Formation (Pleistocene). The Waikato River and its tributaries have subsequently eroded into this succession. The erosion was temporarily interrupted when the Taupo Pumice Alluvium from the 186 A.D. Taupo Eruption (Wilson et al., 1980) filled the Waikato Valley forming the latest aggradation surface. Most of the present flat swampy topography surrounding Huntly represents the top of the Taupo Pumice Alluvium aggradation surface. Geomorphic development of the Lower Waikato Lowland is schematically illustrated by Figure 1.9.

1.5.4 Regional Hydrology

Climate of the area is mild with rainfall evenly
Figure 18: Major Regional Geomorphic Units of the Lower Waikato.
Diagram not to scale. Stratigraphic contacts approximate only. Construction is based on the regional study by Kear and Schofield (1978) and East Mine Tauranga Group interpretation by Todd (1982b).

Figure 1-9: Schematic geological cross-section showing the geomorphic development of the Huntly area as determined by cycles of aggradation and degradation within the Tauranga Group sequence.
distributed throughout the year and only slightly increasing in winter months. Data from the Huntly meteorological station gives a 1273mm mean annual rainfall with a range of 913-1592mm between 1940-1981. All surface drainage is either to the Waikato River or the numerous lakes on the Lower Waikato Lowland.

The Tauranga Group sequence contains extensive high yielding aquifer\(^1\) systems and over the East Mine at least two of these systems are present (Todd, 1982; B.M.C., 1984). The upper system is regionally extensive, generally 10 to 30 metres thick and comprises the Taupo Pumice Alluvium, Hinuera Formation, Waeranga Gravels, Karapiro Formation and part of the Whangamarino Formation. This system is unconfined and appears to be laterally continuous with the Waikato River. The lower aquifer system is approximately 5 to 10 metres thick and limited in extent to the channels on the Te Kuiti-Tauranga Group contact. The aquifer systems are separated by 10-25m of silts and clays which form a semi-confining layer or aquitard. The semi-confined lower aquifer system and the aquitard are both part of the Whangamarino Formation.

Te Kuiti and Newcastle Group deposits can be considered an aquiclade apart from areas of fractured ground or coal seams where permeability may be locally higher. A summary of the major hydrogeologic units is presented on Table 1.2.

1.5.5 Synthesis: Implications of Regional Factors to Hostel Site

Regional factors considered important to the site are that:

1) the subsurface geology of the site consists of Tauranga, Te Kuiti and Newcastle Group materials, each distinct in terms of lithology and structure.

1. See Appendix One for definitions of hydrologic terms.
<table>
<thead>
<tr>
<th>STRATIGRAPHY AND MATERIAL DESCRIPTION</th>
<th>THICKNESS (m)</th>
<th>WATER YIELDING CHARACTERISTICS</th>
<th>POROSITY (n)</th>
<th>HYDRAULIC CONDUCTIVITY (PERMEABILITY: k in m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>UPPER AQUIFER</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coarse pumice sand, gravel with minor silts, mud and peat layers. (Hinuera Formation, Taupo Pumice Alluvium and Karapiro Formation)</td>
<td>10 - 30</td>
<td>High yielding</td>
<td>35 - 67</td>
<td>$2 \times 10^{-5}$ to $4 \times 10^{-4}$</td>
</tr>
<tr>
<td><strong>AQUIFER</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silts, silty clay, muds, silty very fine sands; minor coarse sand and gravel layers (part of Karapiro and Whangamarino Formations)</td>
<td>10 - 25</td>
<td>Low yielding except for thin sand or gravel horizons</td>
<td>40 - 60</td>
<td>$10^{-4}$ to $10^{-5}$</td>
</tr>
<tr>
<td><strong>LOWER AQUIFER</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fine to coarse silty sands alternating with medium to coarse greywacke gravels, minor clays and silts (part of Whangamarino Formation)</td>
<td>5 - 10</td>
<td>Suspected high yielding</td>
<td>35 - 60</td>
<td>$1.2 \times 10^{-5}$ to $3.8 \times 10^{-5}$</td>
</tr>
<tr>
<td><strong>TE KUITI GROUP</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mangakotuku Siltstone</td>
<td></td>
<td>Virtually dry except near fractured ground or coal seams (AQUICLIDE)</td>
<td>less than 10</td>
<td>$10^{-10}$ to $10^{-8}$ with $10^{-7}$ + $10^{-3}$ for some coal seams and possibly the Pukeko Sandstone</td>
</tr>
<tr>
<td>Pukemiro Sandstone</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glen Afton Claystone</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Waikato Coal Measures</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Greywacke basement</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 For material descriptions and approximate thicknesses of Te Kuiti group strata see Table 1.1
2) the geometric constraints to the thickness of Tertiary overburden above mine workings are the 5-15° NW dip of the Te Kuiti Group sequence and the moderate relief on the Te Kuiti-Tauranga Group erosional contact (overburden stresses are also controlled by these two factors combined with surface relief determining Tauranga Group overburden thickness).

3) the Tauranga Group lithologies show complex field relationships with lateral and vertical variations over short distances.

4) high yielding aquifers are present within the Tauranga Group overburden at the site.

5) the only active process considered to be present at the N.Z.E.D. Hostel (apart from subsidence) during early 1983 is ground water movement.
CHAPTER II

PREVIOUS INVESTIGATIONS AND DEVELOPMENT OF SITE MODEL

2.1 Introduction

This chapter reviews and describes the development of the hostel subsidence and associated investigations. The chapter is based on reports, survey monitoring data, and personal observations from S.C.M., L.S. (Department of Lands and Survey) and M.W.D. These previous investigations are combined with the regional studies of Section 1.5 to develop a tentative engineering geological site model.

2.2 Survey Monitoring

Since 1978 a survey network has been established by L.S. over the East Mine to monitor mining related movement. Figure 2.1 shows the network over the south headings in relation to the mine geometry and the urban area. Average distance between marks is 100m. Bradley (1982), in his review of the monitoring, specifies the horizontal and vertical accuracies as ±10mm and ±3mm respectively. From observations of marks not affected by mining activity, Bradley reports that seasonal vertical movement from natural causes is generally less than 5mm.

Monitoring to July 1985 has identified 3 areas of subsidence greater than 100mm. Area location, subsidence timing and maximum measured settlements are shown in Figure 2.1. Subsidence above the south headings although irregular, is trough shaped and generally follows mine geometry. Limit angles or angle of influence (Figure 2.2) range between 38° and 71° for the 5mm subsidence contour.

The survey marks most affected by the hostel subsidence are E53 and E61. Settlement-time plots for these marks are presented in Figure 2.3. Features to note are that:

1) the subsidence is characterised by two phases of rapid movement with a maximum calculated settlement rate of 36mm/day.
Figure 2.2: Sectional view of an idealised subsidence trough showing the relationships between the various geometric parameters (Bell, 1975).
2) settlement rates gradually reduce after the rapid phases (additional data shows that the settlement rates became constant approximately 12 months after the rapid phases with constant rates evaluated at 0.7 and 0.6 mm/day for E61 and E53 respectively).

3) approximately 85 mm of settlement occurred for both marks before the first rapid phase commenced.

Incorporating settlement data from other survey marks subsidence profiles can be constructed across the hostel site (Figure 2.4). Profiles show the development of the trough with the easterly migration of subsidence associated with phase II rapid movement.

M.W.D. established a levelling network in the hostel grounds on the 12th January, 1983. Spacing of M.W.D. marks is approximately 20 metres, the marks being installed on concrete paths, kerbing and tarseal. Estimates of total settlement at these locations can be made using original building levels and site grading plans. Where possible this data is incorporated with L.S. data to construct subsidence contour plans (Figures 2.5 and 2.6).

Subsidence contours at the 14th March 1983, show the trough to be nearly circular with an approximate diameter of 300 m. Contours plotted for the 2nd August 1983 show similar features to the subsidence profiles (Figure 2.4) with the locus intensifying and a general northeasterly propagation of subsidence.

Vectors showing horizontal movement of L.S. survey marks for both phases of rapid movement are also shown on Figure 2.5 and 2.6. A circular 'dish' shaped subsidence produces horizontal movement vectors which intersect at the point of maximum subsidence. Due to the irregular nature of the subsidence trough hostel vectors for both phases of subsidence have areas of intersection. Vector data suggests that the area of maximum subsidence has migrated approximately 60 metres northwest. The apparent easterly migration of subsidence from levelling data is probably secondary to this northwesterly trend. Subsidence contours failed to reflect the northwesterly migration due to the
**SETTLEMENT-TIME PLOT OF E53 AND E61**

- **Phase I** (slow settlement where E53, E61 installed 1983)
- **Phase II** (phase of rapid subsidence)
- **E61**
  - 07 mm/day
- **E53**
  - 06 mm/day

**NZED Hostel Surface Effects**
- 5/1/83: minor cracks noticed
- 5/1/83: minor cracks
- 5/1/83: moderation of tension and compression features most prominent

**Mine Water Discharge in South Headings**
- Flow increased from 0.25 l/s to 5.0 l/s
- Water from 14 A.M. to 1 P.M.
- Note: no reports of dusted brine in water
- Flow measured 0.4 l/s, remaining figures visual estimates

**Underground Inspections**
- Roof falls mapped
- 1-8-93: 4-13-93: 14-23

**Miscellaneous**
- Mining under hostel
- Jan-1983
- Wastes accumulated near surface
- Statistical observation and monitoring of water level and features during settlement
- Some investigations required prior

**Figure 23: Subsidence at E53 and E61 with Related Events**
Figure 2.4: Subsidence profiles across hostel site

vertical exaggeration x 250
(subsidence profiles plotted by D. Depledge, SCM)
Figure 2.5: Subsidence contours and horizontal movement vectors at the 14th March, 1983 (post phase I-pre phase II rapid subsidence).
lack of levelling data in that area.

Maximum lateral movement detected during the hostel subsidence for monitoring to May 1984 is 296mm at E53. Maximum compression is between E39 and E53 with a shortening of 560mm in 134.5m (compressive strain is 4mm/m) and maximum tension between E39 and E39A where 60.5m is lengthened by 293mm (tensile strain is 5mm/m).

2.3 State Coal Mine Reports and Investigations

2.3.1 Mining conditions under hostel

Panel 1 below the hostel was the first area of the south headings to have pillars split and floor coal extracted. The southern and southeastern margins of the panel represent mine boundaries, in this case the result of the coal seam becoming too thin for extraction. Williams (1983) notes that the workings are in 'poor ground' with some intersections requiring propping and that the area was wet during mining with large inflows of water from roof fractures. Offord (1983), in his fortnightly mine manager's report described mining conditions as wet causing frequent bogging down of shuttle cars.

2.3.2 Mine water discharge

On the 5th October, 1982 (prephase I rapid subsidence, see Figure 2.3) a large inrush of water from panel 1 was reported (Offord, 1983) at a partially constructed concrete stopping at the end of No. 2 return (see Figure 2.3 for location). The water was reported to have a sulphury odour with an estimated flow of 10 l/s subsequently reducing to 2 l/s (Figure 2.3). At the same location on the 5th November, 1982 flow increased to an estimated 15-20 l/s subsequently reducing to 6 l/s (Williams, 1983). During mid-April 1983 discharge from panel I was noticed at another locality in the belt haulage roadway (McInally, 1983b). All reports of panel I discharge state that the water was clear and contained no suspended solids. It is important to note that the only measurement
of flow from panel I is 7 l/s, recorded on the 6th May, 1983 (Depledge, 1983c). All previously stated flow rates are approximate only and based on visual observation.

S.C.M., investigating possible dewatering consolidation, carried out tests to determine the hydraulic continuity between mine discharge and overlying Tauranga Group aquifers. Water tracing procedures using fluorescein dye and salt combined with chemical and tritium analyses on mine, aquifer and surface waters were completed.

Fluorescein dye, mixed with drilling mud was pumped into sandy gravels at the bottom of M.W.D. borehole 6599 (Section 2.4) on the 20th January, 1983. Final depth of BH 6599 is 52m, approximately 17m above the Te Kuiti-Tauranga Group contact. No tracer dye in the south headings water has been recorded (Depledge 1983a). Salt (NaCl) was placed in piezometers established by S.C.M. (Section 2.3.4) into the upper aquifer later in 1983. Sensors detected no increase in salt concentration of mine water from panel I (pers. comm. J. Gumbley, 1984 - S.C.M. geologist).

Six chemical analyses giving pH, Ca, Mg, K, Na, Fe, Zn and B contents of mine and surface water were completed for S.C.M. by the Chemistry Department, University of Waikato (Brenner, 1983). Analyses failed to provide any indication of hydraulic continuity because of limited sampling combined with a lack of regional groundwater chemistry data.

Tritium is a radioactive isotope found in water and is primarily used to determine water age (Stewart and Taylor, 1981). Tritium ratios (3H/H) determined for S.C.M. by the Institute of Nuclear Sciences, D.S.I.R., are presented on Table 2.1. Conclusions drawn from this data by Taylor (1983) are;

1) the tritium ratios from surface water match ratios of recent precipitation.
2) the tritium ratios for water collected out of piezometers 6660 and 6664 (upper aquifer piezometers) are typical of springs and groundwater from shallow depths in pumiceous deposits of the Central North Island volcanic zone. The ratios suggest a mixture of water of various ages.
Table 2.1: Tritium Ratios of East Mine Water (Analyses determined for S.C.M. by Institute of Nuclear Sciences, D.S.I.R.).

<table>
<thead>
<tr>
<th>Location</th>
<th>Sampling Dates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>12 May 1983</td>
</tr>
<tr>
<td>Crosscut No. 1</td>
<td>0.00±0.12</td>
</tr>
<tr>
<td>South Section settling tank(^1)</td>
<td></td>
</tr>
<tr>
<td>East Mine Discharge</td>
<td>0.26±0.11</td>
</tr>
<tr>
<td>Piezometer 6660(^2)</td>
<td>1.89±0.15</td>
</tr>
<tr>
<td>Surface Water</td>
<td>3.35±0.13</td>
</tr>
<tr>
<td>Piezometer 6664(^2)</td>
<td></td>
</tr>
<tr>
<td>North Headings Cross-cut No. 3</td>
<td></td>
</tr>
<tr>
<td>Borehole 71(^2)</td>
<td></td>
</tr>
</tbody>
</table>

1. The settling tank collects all mine water from the south headings, including panel 1.

2. For location of piezometers and borehole 71 see Figure 3.11. Borehole 71 is an incompletely sealed borehole encountered during mining and out of which inflow occurs.
3) the data from south headings discharge suggest entry of younger water into workings with the September 1983 value, and subsequent total absence of younger water with the April 1984 value.

Thus the tritium analyses are inconclusive with respect to the possible hydraulic connection between Tauranga Group aquifers and the mine workings.

2.3.3 Underground Inspections and Mapping

After the hostel subsidence mines rescue personnel inspected the panel 1 workings on three occasions. On the 20th January 1983, they discovered roof falls 1 to 8 (Figure 2.3) which prevented further inspection of the panel. After excavating an exploratory drive down the northwest panel margin, entry was made again in early February, 1983. During this inspection photographs were taken and are reproduced here as Figures 2.7 to 2.10. Full inspection of the panel was again prevented by roof falls 9 to 13.

On the basis of observations from the January and February underground inspections, McInally (1983a) drew the following conclusions:

1) The pillars in the areas examined are all intact and show no signs of 'weighting' or any significant deterioration.

2) The roadways and crosscuts are standing well and no major roof, rib or floor stability problems are apparent in any open roadway. Some minor heave and rib spall is present but this is not sufficiently developed to cause roof or pillar stability problems.

3) Access into one half of the panel is blocked by a series of roof falls. These falls are all at or very near junctions and there is no sign of stressing of roof or ribs right to the lip of the falls. The indications are that the falls could be failures at junctions with no extensive failures along roadways.

4) Most of the falls are showing no sign of water inflow and much of the material in the falls is
Figure 2.7: View northeast in cross-cut 22. Note the water on floor and minor rib spall. Cross-cut geometry is approximately 5 metres wide x 6 metres high. Rock bolts at c.1 metre centres extend 1.8 metres into roof.
(Note: Photographs 2.7 to 2.10 by S.C.M. Rescue Team, February, 1983).

Figure 2.8: Northeasterly view of roof fall No 9. Ladder on left was used for access from exploratory drive. Note the blocky nature of fallen debris.
Figure 2.9: View of pillar corner between roof falls 13B (left) and 13A (right).

Figure 2.10: View into caving chimney above roof fall 13A. Approximate distance to chimney roof is estimated at 3.0m. Note the 'ellipsoidal' caving geometry.
blocky and compact.

On the 10th April, 1983 an airblast resulting from sudden roof collapse blew out the stoppings which separate panel 1 from the belt haulage drive and the No.1 return (Figure 2.3). A subsequent underground inspection discovered roof falls 14 to 23.

Underground mapping of mine dimensions, geologic structure and pillar rib condition of the northeastern section of panel 1 by geomechanics staff (S.C.M.), is presented on Figures 2.11 to 2.13. Collection of this data was prompted by subsidence above panel 1, and was not part of normal mine working procedure.

Figure 2.11 shows estimated roof heights to vary between 3.5m for 'first-cut' drives to 6m where floor coal is extracted. Half of the pillars in the area are below the design dimensions of 15m x 15m (60m circumference).

Geologic structures mapped are shears and face cleat (Figure 2.12). The face cleats are near vertical with trends of 069° to 090°. These orientations are consistent with the average trends of 072° to 088° for the East Mine (Field, 1980). Shears trend 080° to 095° and 010° (minor). The minor 010° shear zones are of similar orientation to the N trending major faults of the area (Section 1.5.2.2). Shears with orientations 080° to 095° have similar trends to cleat direction.

Observations of ribs at the time of mapping (Figure 2.13) show that rib 'erosion' is most commonly concentrated in the upper and middle portions of the rib, extending up to 1.5m past the original cut face.

2.3.4 Subsidence fracture mapping and piezometer installation

Surface investigations by S.C.M. include mapping of compressive and tensile subsidence features around the hostel in January 1983 (see Section 3.2.2) and installation of 10 piezometers during May 1983. S.C.M. piezometers monitor the upper aquifer in the Tauranga Group (see Section 3.5.3).
2.4 Ministry of Works and Development reports and investigations

From the 13th to 24th January 1983, M.W.D. drilled a cored borehole (BH 6599) at the hostel site (Figure 3.11) to help identify the cause of ground instability. The hole was confined to the Tauranga Group, its finishing depth estimated as 17m above the Te Kuiti-Tauranga Group contact. Core recovered from the top 10 to 15m showed no signs of shearing which would be consistent with slope failure causing the ground instability (Stewart, 1983). Core from BH 6599 has been relogged (see Appendix Two) and included in geological cross-sections (Figure 3.17 and 3.18) as part of this study.

M.W.D. laboratory investigations on BH 6599 core include grain size analyses, and determinations of bulk density, solid density, water content and porosity. These results are discussed in Chapter 4.

M.W.D. have provided advice to N.Z.E.D. regarding the hostel subsidence. Subsidence description, discussion of possible modes of failure and recommendations regarding site investigations and landuse are presented by Williams (1983, 1985).

2.5 Synthesis: Development of a tentative engineering geological site model.

From existing information a number of failure modes have been suggested to explain ground instability at the N.Z.E.D. Hostel (Depledge, 1983a; McInally, 1983a and Williams, 1983). Possible modes of failure can be divided into:

a) Those independent of mining activity;
   i) slope instability,
   ii) consolidation of upper Tauranga Group materials due to placement of landfill and drainage associated with residential development.

b) Those related to mining;
   i) consolidation of lower Tauranga Group materials due to mining induced dewatering,
ii) mine floor failure (pillar punching),  
iii) mine pillar failure,  
iv) mine roof failure consisting of,  
   - roof collapse migrating to the ground surface  
   - or roof collapse causing material erosion of Tauranga Group silts, sands and gravels.  

A model explaining the cause of the subsidence could also include any number of the above.  

If slope failure were to have caused the instability, the area of subsidence would be typically located on the upper slope with a zone of uplift (accumulation) on the lower slope. All horizontal movement would be downslope. Survey monitoring (Figures 2.5 and 2.6) has shown the maximum area of subsidence to be on the lower slope with no evidence of a zone of uplift. L.S. marks E53 and E54 on the lower slope have moved towards the slope during the subsidence. Monitoring data is supported by M.W.D. drilling (Section 2.4) where no shear surface or zone of disturbance was identified in core.  

Consolidation related to residential development is possible through either placement of landfill or establishment of stormwater drainage. Placement of landfill increasing surface loads as a failure mode for the hostel subsidence can be dismissed since the areas of maximum subsidence are located in natural ground (Figure 1.4). Stormwater drainage associated with residential development has reduced infiltration and recharge to the upper aquifer system (Section 1.5.4). Effects of reduced infiltration is considered negligible due to the regional extent of the upper aquifer system.  

Roof collapse migrating directly to the surface produces 'crown holes' ¹ and not the subsidence trough defined by survey monitoring. Based on bulking factors for coal measure rocks, St. George (1983) estimates the upper limit of void migration from roof collapse (discussed in Chapter Five) as approximately 10t (where t = mined thickness, 3.5 to 6.0m for panel 1). Panel 1 is 100 to 110m below the hostel and thus significantly greater than 10t.  

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¹. See Appendix One.
The absence of Tauranga Group materials, both in the mine workings and in mine water discharge, provides further evidence of the restriction of caving to the Te Kuiti Group sequence.

Field evidence against mine floor and pillar failure are the underground inspection of the 20th January 1983, and the location of maximum subsidence related to phase I rapid movement. Figure 2.3 shows the workings inspected by S.C.M. which included part of the area of maximum subsidence as defined by survey data. As section 2.3.3 discusses more fully, no significant pillar or floor failure was observed. However, since the panel inspection was limited, pillar or floor failure cannot be completely dismissed as contributing to the observed settlement.

The engineering geological model considered most likely on the basis of field evidence alone is mine roof collapse allowing drainage and depressurising of aquifers at the base of the Tauranga Group (Figure 2.14). Aquifer depressurisation would cause consolidation of materials associated with the lower aquifer system. This model is suggested by:

1) the close proximity of the lower Tauranga Group aquifers to mine workings (Figure 1.7).
2) the underground observation of mine roof falls under the subsided area (Section 2.3.3).
3) the amount of water encountered during mining which suggests the presence of drainage paths from overlying Tauranga Group aquifers (Section 2.3.1).
4) the general timing of mine water discharge from panel 1 in relation to the observed subsidence.
FIGURE 2.14: TENTATIVE ENGINEERING GEOLOGICAL SITE MODEL
3.1 Site Investigation Objectives and Approach

Objectives for the hostel site investigation programme are:

a) to define subsurface geology in the lower part of the Tauranga Group in the area of subsidence.

b) to define relief on the Te Kuiti-Tauranga Group contact under the area of subsidence.

c) to recover core from Tauranga Group lithologies for laboratory testing.

d) to install piezometer monitoring in the lower aquifer system over adjacent, more recent workings where subsidence may occur in the future.

e) to measure in-situ compressibility of Tauranga Group materials using a static cone penetrometer.

The site investigation programme was carried out in two stages:

a) an initial reconnaissance stage which involved mapping of site geomorphology and surface strain features associated with the subsidence; and

b) a later more extensive phase of subsurface investigation by core and wash drilling, geophysical borehole logging, dutch cone static penetrometer soundings and piezometer installation.

The approximate cost of site investigation programme was $30,000.

3.2 Engineering Geological Mapping

The absence of surface outcrop in the hostel area restricted mapping to description of site geomorphology plus the location and description of surface strain features.
3.2.1 Site Geomorphology

Mapping of morphological features is based on a site 'walk-over' and interpretation from aerial photographs.

The site can be divided into two main landforms (Figure 3.1). The eastern part of the study area, where the hostel is located, exhibits relief up to 46 metres (a.s.l.) and is part of an extensive modified aggradation surface (Section 1.5.3). Modification of this surface is illustrated 150 metres north of the hostel with stream incision, and by the irregular nature of the northern and western slopes below the hostel, interpreted as being caused by degradation from a past meandering river system.

The western low lying area is generally flat and is part of an extensive younger fluvial terrace formed by overbank deposits from the Waikato River (Taupo Pumice Alluvium - Kear and Schofield, 1978). The western area has been significantly modified by urban development (See Section 1.4.3).

3.2.2 Surface Strain Features

3.2.2.1 Description

The distribution of surface strain features associated with the hostel subsidence is shown on Figures 3.2 and 3.3. Strain features were initially mapped by Stewart (1983) after phase I rapid subsidence then mapped as part of this study following phase II rapid subsidence.

Surface strain features are either tensile or compressive. Tensile features are most commonly distributed around the outer margins of the subsidence trough. During subsidence, fractures occurred in concrete paths (Figure 3.4), kerbing, tarseal (Figure 3.5), brick veneer (Figure 3.6) and natural ground (Figure 3.7). Fracture offsets generally range from a few to 100 millimetres. Orientations of fractures are commonly concentric and rarely radial to the locus of maximum subsidence (Figures 3.2 and 3.3).

Compressive features generally occur on the inner parts of the subsidence trough. Compressive strain is
FIGURE 3.1: SITE GEOMORPHOLOGY
Figure 3.2: Distribution of surface strain features with subsidence contours - Post phase I rapid subsidence. Strain features mapped by D. Stewart (SCM), late January, 1983.
Figure 3.4: Tension fracture in concrete path and kerbing, Burke Place, January 1984. Fracture displacement ranges between 5 and 20 millimetres. Path fractures are common around the hostel and surrounding areas.

Figure 3.5: Tension fracture in kerbing and tarseal, Burke Place, January 1984. The most prominent fracture on Burke Place with an absolute displacement of 180mm measured between two originally adjacent points. Fracture is at kerb join.
Figure 3.6: Tension fracture in brick veneer, Block E, N.Z.E.D. Hostel, January 1984. Fracture displacement is a few millimetres.
Figure 3.7: Tension fracture in natural ground below the Hostel Manager's residence, January 1984. Fracture displacement ranges between 20 and 50 millimetres. Note the extension of the fracture through the foundation wall in background.
Figure 3.8: Compression zone in concrete curbing and lawn between Blocks E and B, N.Z.E.D. Hostel, January 1984. Curbing on right overthrusts that on the left. Compression roll in foreground is marked by a bare area where lawn has been removed with mowing.

Figure 3.9: Compression zone in concrete curbing and tarseal, southern garage area, N.Z.E.D. Hostel, January 1984. Kerbing on left has been thrust to the right, causing a compression roll in tarseal.
expressed in brittle materials as upward buckling of footpaths and kerbing (Figure 3.8) or in more ductile materials as compression rolls in both tarseal (Figure 3.9) and natural ground. Vertical amplitudes of compression rolls generally range between 20 and 50 millimetres with a maximum measured amplitude of 160 millimetres. Orientations of the compressive features are generally either concentric or radial to the locus of maximum subsidence (Figures 3.2, 3.3).

3.2.2.2 Interpretation

Distribution of strain features with an outer tensile zone and an inner compressive zone is consistent with observations from other subsidence troughs in both North America (Gray et al., 1977) and the United Kingdom (N.C.B., 1975). An idealised subsidence trough showing this relationship is presented in Figure 2.2.

The northeasterly propagation of tensile fractures and extension of the central compression zone between January and December 1983 is indicated by survey data (Section 2.2) and marks the development of phase II rapid subsidence. Maximum zones of compression and tension defined by survey data are between E39-E53 and E39A-E39 respectively (Section 2.2). Both zones exhibit a concentration of surface strain features.

Surface strain features are most common in the eastern half of the subsidence trough. This apparent concentration is interpreted as being due to a combination of the effect of ground slope, which exaggerates the surface expression of strain, and a lack of good strain indicators in the northern and southwestern margins of the trough.

The predominant concentric orientation of strain features suggests that the principal horizontal strain direction is radial to the subsidence locus. The subordinate radial orientation of strain features, most common within the compression zone suggests a component of concentric horizontal strain during the development of the sub-circular subsidence trough.
3.3 Wash and Core Drilling

Brown Brothers Ltd were contracted by S.C.M. for the drilling programme. Rigs used included a tractor mounted Gardner Denver 200T and a truck mounted Failing Model 1250, the larger rig being required for coring (Figure 3.10).

The location of all boreholes including those from previous S.C.M. and M.W.D. investigations are shown in Figure 3.11. Borehole 6651, the main investigation hole, is fully cored and central to the subsidence area. Holes 6652, 6653 and 6657 are partially cored and wash drilled with 6654, 6655 and 6656 all wash drilled holes. All holes apart from 6657 extended through the Tauranga Group to the top of the Te Kuiti Group sequence. Sample dimensions for consolidation testing required 150mm diameter core to be recovered.

All core was logged as recovered. Samples from wash drilled holes were bagged every metre and logged later with the aid of geophysical data. Circulation loss and responses of rig to penetration are noted on the borehole logs. Detailed lithology logs and summary logs for each hole are presented in Appendix Two. Summary logs are used for cross-section construction in Section 3.8.

3.4 Geophysical Borehole Logging

3.4.1 Introduction

Borehole geophysics was included in the site investigation programme to aid geological and geotechnical correlation and interpretation of the Tauranga Group sequence.

In the proceeding section each logging tool used is briefly described according to its response. Following these descriptions, lithologic interpretations and geotechnical implications are summarised. A more comprehensive discussion of theoretical aspects, typical responses, uses, limitations, tool design and units of measurement is presented by Hoffman et al. (1982).
Figure 3.10: Brown Brothers Model 1250 'Failing' rig, recovering 150mm core from BH6651. Once core is extracted from triple tube barrel (about to be pulled away from mast), it is rolled into split p.v.c. (sitting on drill rod rack in foreground), logged, wrapped in plastic then stored under refrigeration.
Figure 3.11: Locations of Boreholes and Penetrometer Soundings for Site Investigation Programme
A summary field data sheet showing log responses for BH6651 is presented on Figure 3.12. Log responses for other boreholes are presented in Appendix Three.

3.4.2 Log Responses

3.4.2.1 Natural Gamma Log

Gamma ray logging tools measure natural gamma radiation emanating from decaying isotopes (B.P.B., 1981). Common isotopes include potassium-40 (K\(^{40}\)), uranium-238 (U\(^{238}\)), uranium-235 (U\(^{235}\)) and thorium-232 (Th\(^{232}\)). Due to the very low concentrations of uranium and thorium in most areas (Levinson, 1974), the most important source of radiation is considered to be K\(^{40}\).

On the basis of field observations, potential sources of K\(^{40}\) within the Tauranga Group sequence are acid volcanic glass, potassic feldspars (K,Na) [AlSi\(_3\)O\(_8\)], illite clays K\(_{1-1.5}\)Al\(_{4}\) [Si\(_7\)-6.5Al\(_1\)-1.5O\(_{20}\)] (OH)\(_4\) and jarosite KFe(SO\(_4\))\(_2\)(OH)\(_6\).

The natural gamma response is relatively unaffected by borehole caving or casing but does show amplification out of water (Figure 3.12). Natural gamma radiation is measured in A.P.I. units (American Petroleum Institute), which are empirical, and based on calibration test pits.

3.4.2.2 Gamma-Gamma Log

A gamma-gamma or density logging tool bombards the borehole walls with gamma radiation which is backscattered to a counter. The count rate reflects electron density which can be related to the material density (Hoffman et. al., 1982). Gamma-gamma logs are presented in terms of material density on a logarithmic scale.

Important considerations when interpreting gamma-gamma logs include the effects of borehole caving and high natural gamma radiation. In caved zones the logging tool incorporates drilling fluid density which results in an underestimate of material density (Figure 3.12). The logging tool counter is also sensitive to natural gamma
Summary log for Borehole 6651

- Ignimbritic breccia and tuffaceous sandstone
- Sandy silt
- Upper ignimbritic silt unit

Karapeti Formation
- Sandy silt
- Fine sand to sandy silt
- Laterally layered silt to fine sand
- Ignimbritic breccia

Karanjino Formation
- Fine sand to sandy silt
- Sandy silt to fine sand
- Laterally layered sands and fine sand

Wahmararingo Formation
- Sandy silt to fine sand
- Fine sand to sandy silt
- Laterally layered silt

Wahmararingo Formation (Lower ignimbritic silt unit)
- Sandy silt to fine sand
- Sandy silt to fine sand
- Laterally layered silt

Fig 3.12: Field Data Logs for Borehole 6651

- Gamma-gamma log
- Neutron-neutron log
- Collar log
- Static cone penetrometer soundings

Limit of soundings
Penetrometer Soundings: M.K.D. Hamilton District Laboratory
Original Depth Scale: 1:200

Low resistivity indicates high natural porosity
Higher gamma radiation values for possible rich sands in comparison to possible poor sands (refer to detailed lithology logs)

Fig. 3.12
radiation and when these levels are high, the material density response is exaggerated. Out of water a low density response is typical, reflecting air density (Figure 3.12).

### 3.4.2.3 Neutron-Neutron Log

The neutron-neutron tool emits high energy neutrons which bombard nuclei in the strata and borehole fluid. The tool's detector is only sensitive to low energy neutrons, therefore the response is determined by the effectiveness of the nuclei to dissipate neutron energy. Most efficient in reducing neutron energy is the hydrogen nucleus H⁺, so the log is used as a direct measure of hydrogen content within its range of influence (Hoffman et al., 1982). Most hydrogen is present as water which includes H₂O in pore spaces and chemically bound water associated with clay minerals. Neutron-neutron logs are thus used as an indirect measure of water content and porosity.

Units of measurement are c.p.s. (counts per second) with high count rates indicating low H⁺ content and low count rates high H⁺ content. Casing reduces the count rate and the tool only operates below the borehole water level. A short spaced neutron logging tool was used for the hostel investigations.

### 3.4.2.4 Caliper Log

The caliper logging tool uses a mechanical arm to measure borehole diameter. The resulting caliper log identifies zones of caving and swelling which in turn are used for interpreting gamma-gamma logs. Since loose or poorly cohesive materials are prone to caving or swelling, the log can also be used as an indirect indicator of material strength.

### 3.4.3 Lithological Interpretation

Comparing log responses to detailed material descriptions from cored portions of boreholes, geophysical signatures unique to particular units can be determined.
These signatures are particularly useful for geological interpretation in zones of core loss and of washings from wash drilling. Determining material type from geophysical logs is an 'indirect' procedure based on an intuitive knowledge of material composition for the natural gamma log, density for the gamma-gamma log and porosity for the neutron-neutron log.

General observations based on geophysical data from cored portions of boreholes are:

- for fine grained materials (see Section 3.8.2 for material classification;

  a) fluviatile sandy silts and clayey silts are characterised by high density and low porosity responses (BH6651, 34.6 to 35.0, 36.3 to 38.0 and 61.4 to 62.2, see Figure 3.12)

  b) ignimbritic clayey silts and sandy silts are characterised by a higher porosity and lower density log responses (BH6651, 48.0 to 58.0, Figure 3.12).

- for coarse grained materials

  a) caving is a common feature particularly with sands lacking fines (BH6651, 25.0 to 27.5) and at unit contacts (BH6651, 46.4 to 48.0, Figure 3.12).

  b) higher natural gamma radiation levels are recorded for pumice rich sands in comparison to pumice poor sands (BH6651 - 28.7 to 31.4, Figure 3.12).

  c) gravels are characterised by low porosity and high natural gamma log responses (BH6651, 35.4 to 36.2 and 62.8 to 64.0, Figure 3.12).

- for peat;

  a) peat horizons are characterised by low density and high porosity log responses (BH6652, 14.1 to 14.6 and BH6657, 1.0 to 3.0, Appendix 3).

  Log responses to lithological boundaries are

1. For zone locations in boreholes, BH6651, 34.6 to 35.0 refers to material between depths 34.6 and 35.0 in borehole 6651.
generally dependent on the source-detector spacing, length of detector crystal and speed that the logging tool travels up the borehole (Hoffman et al., 1982). For this investigation positions of lithological contacts are determined on the basis of 'rules of thumb' as defined by Haines (1984). On the natural gamma log, the contact occurs one third of the distance along the slope from the high side to the low side. Contacts for the gamma-gamma log occurs at two thirds of the distance from the high side and for other logs the mid-point of the amplitude change mark lithological changes.

3.4.4 Geotechnical Implications

Determinations of in-situ semi-quantitative geotechnical data from geophysical borehole logging include material bulk density (gamma-gamma log) and porosity (neutron-neutron log).

Laboratory determined bulk densities, porosities and water contents (Section 4.3) are compared with gamma-gamma and neutron-neutron log traces for BH6651 on Figure 3.13. Conclusions drawn from these comparisons are:

a) that in general a good correlation exists between laboratory and geophysically determined material bulk density.

b) that the neutron-neutron log in general reflects changes in actual porosity and water content. Low count rates are associated with highly porous materials (BH6651, 48.0 to 58.0) and high count rates with materials of low porosity (BH6651 - 35.0). The inverse relationship between porosity and neutron-neutron count rate is explained in Section 3.4.2.3.

3.5 Piezometers

3.5.1 Design and Installation

Selection of piezometer type was based on hydrostatic response time\(^1\), cost, and simplicity of installation,  

1. See Appendix One for definition of hydrostatic response time.
Figure 3-13: Comparison of Geophysical and Laboratory Determinations of Geotechnical Data

**Boresite 6651 Summary Log**

- igneous porphyritic SANGER BNT with some clay
  - (UPPER IGNIMETRIC SILT UNIT)

- igneous porphyritic SALT with some fine gravel
  - (IGNIMETIC SALT UNIT)

- fine to medium SINNERG with some fine gravel
  - (MERGARVING FORMATION)

- fine to medium SINTER with some fine gravel
  - (MERGARVING FORMATION)

- fine to medium SINSIL with some fine gravel
  - (MERGARVING FORMATION)

- fine to medium SINSIL with some fine gravel
  - (LOWER IGNIMETRIC SALT UNIT)

- fine to medium SINSIL with some clay
  - (LOWER IGNIMETRIC SALT UNIT)

- fine to medium SINSIL with some clay
  - (LOWER IGNIMETRIC SALT UNIT)

- fine to medium SINSIL with some clay
  - (LOWER IGNIMETRIC SALT UNIT)

- fine to medium SINSIL with some clay
  - (LOWER IGNIMETRIC SALT UNIT)

- fine to medium SINSIL with some clay
  - (LOWER IGNIMETRIC SALT UNIT)

**Geophysical Borehole Logs**

- Gamma-gamma log with laboratory density data
- Caliper log
- Neutron-neutron log

**Laboratory Data**

- Void ratio (e) and porosity (n)
- Natural moisture content (w %)

*Note: The neutron-neutron log generally reflects porosity and water content. Low count rates are observed for materials with high porosities and water contents. High count rates are typical for materials with low porosities and water contents.*

*Figure 3-13: Comparison of Geophysical and Laboratory Determinations of Geotechnical Data*
operation and maintenance. Terzaghi and Peck (1967 - page 672) graph the relationship between response time and soil permeability for different piezometers. Using this graph with a knowledge of expected aquifer permeabilities (10^{-4} to 10^{-5} m/s, Table 2.2), a Casagrande open standpipe piezometer (Casagrande, 1949) gave a 90% response time to increase in pore pressure of less than 3 hours. This piezometer is economical and simple to install and operate. The design of the installed piezometers are presented in Appendix 3.

Piezometers were placed at different levels in boreholes 6652, 6653 and 6656 for field permeability tests and monitoring of aquifers over adjacent mine workings away from the central area of hostel subsidence. Aquifer monitoring in areas where subsidence may occur in the future will allow field testing of the dewatering consolidation model (Figure 2.14) with observations of ground settlement and piezometer water levels.

3.5.2 Measurement of Field Permeability

3.5.2.1 Test Method

Falling head permeability tests were carried out on piezometers in boreholes 6652 and 6653. Tests required a tractor water wagon, stop watch and a water level indicator. Initially water was poured into the piezometer inducing an excess head. Horizontal permeability of the soil mass is determined from dissipation of excess head with time (Figure 3.14). Head fall was measured over a 20 minute period. A comprehensive description of the test method is presented by Sharp et al. (1977).

3.5.2.2 Results

Test results are plotted on field sheets, an example is shown on Figure 3.14. (Remaining field sheets are presented in Appendix Four). From plotted data horizontal permeability is calculated. Field permeabilities determined
Figure 3.14: Principle of field falling head permeability test and calculation sheet (adapted from Sharpe et al., 1977).
as part of this project and S.C.M. investigations are presented on Table 3.1 Field and laboratory permeability data are combined in Section 3.8.4 for site hydrology.

3.5.3 Piezometer monitoring

All piezometers installed by this and previous investigations over the hostel site have been monitored by S.C.M. To date no significant lowering in water levels have been recorded.

3.6 Dutch Cone Penetrometer Soundings

3.6.1 Test Procedure

The sounding device of a dutch cone static penetrometer consists of two parts, a conical point and a friction sleeve (Figure 3.15). The cone and sleeve are successively advanced into the soil by a static load. Resistance to penetration is measured in newtons (force) which is then divided by the areas of the cone and sleeve to produce cone bearing capacity ($q_c$) and sleeve friction ($f_s$) usually in units of MPa.

Penetrometer soundings are included in field investigations in an effort to measure in-situ soil compressibility within the Tauranga Group sequence. Correlation between lithological variation and penetrometer data is not attempted.

Tests were carried out by the M.W.D. Hamilton District Laboratory truck mounted penetrometer adjacent to boreholes 6651, 6652 and 6653 (Figure 3.11). Penetrometer data from these localities is presented on Figure 3.12 and in Appendix Three. Due to depth limitations of the penetrometer (maximum depth reached was 30 metres adjacent to BH6653), the data is restricted to the upper half of the Tauranga Group succession.
Table 3.1: Field permeability data for this and S.C.M. investigations from tests on N.Z.E.D. Hostel piezometers.

<table>
<thead>
<tr>
<th>Piezometer</th>
<th>Filter Depth (metres below ground surface)</th>
<th>Material Description and Formation</th>
<th>Horizontal Permeability ($k_h \text{ \text{- m s}^{-1}}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6652(^1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Blue</td>
<td>16.2 - 18.9</td>
<td>silty fine to coarse sand</td>
<td>$4.81 \times 10^{-7}$</td>
</tr>
<tr>
<td>- Red</td>
<td>27.9 - 30.7</td>
<td>medium to coarse sand</td>
<td>$1.77 \times 10^{-6}$</td>
</tr>
<tr>
<td>6653(^1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- White</td>
<td>49.6 - 51.9</td>
<td>silty medium sand</td>
<td>$1.11 \times 10^{-6}$</td>
</tr>
<tr>
<td>6658(^2)</td>
<td>c.12.5 - 15.5(^4)</td>
<td>silty coarse sand</td>
<td>$c.3.6 \times 10^{-6}$</td>
</tr>
<tr>
<td>6661(^2)</td>
<td>c.18.6 - 22.6</td>
<td>fine to coarse sand</td>
<td>$c.3.6 \times 10^{-7}$</td>
</tr>
<tr>
<td>6663(^2)</td>
<td>c.29.5 - 33.1</td>
<td>fine gravel and coarse sand</td>
<td>$c.3.8 \times 10^{-6}$</td>
</tr>
<tr>
<td>6664(^2)</td>
<td>c.24.6 - 26.8</td>
<td>fine gravel and coarse sand</td>
<td>$c.6.8 \times 10^{-6}$</td>
</tr>
<tr>
<td>6666(^2)</td>
<td>c.31.5 - 34.5</td>
<td>fine gravel and sand</td>
<td>$c.4.1 \times 10^{-6}$</td>
</tr>
</tbody>
</table>

1. This investigation.
2. S.C.M. investigation tests (Heu, 1984).
3. Formation abbreviations Kp - Karapiro Formation
   Wg - Whangamarino Formation.
4. S.C.M. filter depths based on incomplete information.
Figure 3.15: Conical point and friction sleeve for Dutch Cone Static Penetrometer (from Schuster and Krizek, 1978).
3.6.2 Penetrometer Evaluation of In-Situ Compressibility

3.6.2.1 Theoretical Consideration

For non-cohesive soils, compressibility has been related to cone resistance by:

\[ C = \frac{1.5q_c}{\sigma_0} \]  \hspace{1cm} \text{(Buisman, 1940)} \hspace{1cm} \text{[Eqn. 3.1]} \ni

where \( C \) = constant of compressibility

\( q_c \) = cone resistance

\( \sigma_0 \) = effective overburden stress at depth

where \( q_c \) is measured.

The compressibility constant can be then used to approximate settlement from a semi-empirical formula derived by Terzaghi-Buisman (Sanglerat, 1972):

\[ \Delta h = \frac{h}{C} \ln \left( 1 + \frac{\Delta \sigma}{\sigma_0} \right) \] \hspace{1cm} \text{[Eqn. 3.2]} \ni

where \( \Delta h \) = settlement of an elemental layer of thickness \( h \)

\( \Delta \sigma \) = increase in effective stress at the same level of \( \sigma_0 \)

Equation 3.2 shows the inverse relationship between the compressibility constant and predicted settlement.

Sanglerat et al. (1969) developed equation 3.1 for application to cohesive soils replacing the constant 1.5 with \( \alpha \), a variable dependent on the nature of the soil. For overconsolidated soils (only considering overconsolidated soils on the basis of laboratory investigations described in Chapter 4), \( \alpha \) can be determined by:

\[ \alpha = \frac{2.3 \left( 1 + e_c \right) \sigma_c}{C_{cc} \frac{q_c}{qc}} \] \hspace{1cm} \text{[Eqn. 3.3]} \ni
where \( C_{cc} = \frac{e_c - e'}{\log_{10}(100 + \frac{1}{\sigma_c})} \)

and \( \sigma_c \) = preconsolidation pressure

\( e_c \) = void ratio corresponding to \( \sigma_c \)

\( e' \) = void ratio at \( (\sigma_c + 100 \text{ kPa}) \)

By using \( e' \) to calculate \( C_{cc} \), Sanglerat intends that values of \( \alpha \) only be used to estimate settlements associated with increases in effective stress of about 100 kPa.

3.6.2.2 Discussion of Results

Values of \( \alpha \) and \( C \) calculated for the overconsolidated ignimbritic\(^1\) sandy silts (Kauroa - Hamilton Ash) and ignimbritic sands (Karapiro Formation) from BH6651 are presented on Table 3.2.

Laboratory testing data (Chapter 4) as presented on Table 3.2 identify two units of contrasting compressibility. The ignimbritic sandy silts and silty sands (samples from BH6651, 6.08 to 10.50) are characterised by high \( C_c \) values and low \( \sigma_c \) values in comparison to the underlying ignimbrite fine to medium sands (samples from BH6651, 12.85 to 19.30) with values of lower \( C_c \) and higher \( \sigma_c \). Cone resistance directly reflects this contrast with low \( q_c \) values through the upper highly compressible unit and high \( q_c \) values through the underlying sands. Consistently low \( q_c \) values for the upper horizon (see Figure 3.12, \( q_c \) values over 3.5 to 12.5 metres depth) indicates that the unit is highly compressible throughout its thickness.

Table 3.3 shows values of \( \alpha \) calculated for a wide range of soil materials from a French study based on 600 samples (Sanglerat, 1972). Values of \( \alpha \) evaluated for samples of BH 6651-608 to 1050m average at 1.7, below the range determined by the French study for

\(^1\) See Section 3.8.2 for material classification.
Table 3.2: Dutch cone compressibility α values based on one-dimensional consolidation testing from BH 6651.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Material Description</th>
<th>$C_c$ (kPa)</th>
<th>$\sigma_c$ (kPa)</th>
<th>$\sigma_0^1$ (kPa)</th>
<th>$q_c$ (kPa)</th>
<th>α</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.08</td>
<td>ignimbritic sandy silt (MH)</td>
<td>1.236</td>
<td>245</td>
<td>91</td>
<td>1014</td>
<td>1.5</td>
<td>16.7</td>
</tr>
<tr>
<td>6.45</td>
<td>ignimbritic sandy silt (MH)</td>
<td>0.967</td>
<td>365</td>
<td>96</td>
<td>919</td>
<td>2.7</td>
<td>25.9</td>
</tr>
<tr>
<td>7.21</td>
<td>ignimbritic sandy silt (MH)</td>
<td>1.012</td>
<td>330</td>
<td>108</td>
<td>1297</td>
<td>1.8</td>
<td>21.6</td>
</tr>
<tr>
<td>9.83</td>
<td>ignimbritic silty fine-medium sand (MH-SM)</td>
<td>1.396</td>
<td>210</td>
<td>147</td>
<td>1149</td>
<td>1.0</td>
<td>7.8</td>
</tr>
<tr>
<td>10.50</td>
<td>ignimbritic silty fine-medium sand (MH-SM)</td>
<td>0.876</td>
<td>180</td>
<td>157</td>
<td>973</td>
<td>1.4</td>
<td>8.7</td>
</tr>
<tr>
<td>12.85</td>
<td>ignimbritic fine-medium sand (SM-SW)</td>
<td>0.213</td>
<td>670</td>
<td>192</td>
<td>3041</td>
<td>4.8</td>
<td>76.0</td>
</tr>
<tr>
<td>13.59</td>
<td>ignimbritic fine-medium sand (SM-SW)</td>
<td>0.156</td>
<td>630</td>
<td>197</td>
<td>15,000</td>
<td>1.7</td>
<td>129.4</td>
</tr>
<tr>
<td>19.30</td>
<td>ignimbritic fine-medium sand (SM-SW)</td>
<td>0.133</td>
<td>650</td>
<td>233</td>
<td>30,270</td>
<td>0.8</td>
<td>103.9</td>
</tr>
</tbody>
</table>

$^1$ Effective overburden stresses calculated from boundary conditions described in Appendix 6

$C_c$ = Compression Index  
$\sigma_0$ = Effective overburden stress  
$\sigma_c$ = Preconsolidation pressure  
$q_c$ = Cone resistance  
C = Constant of compressibility
Table 3.3: Values of the $\alpha$ coefficient for clayey and silty soils (Sanglerat, 1972).

<table>
<thead>
<tr>
<th></th>
<th>$q_c$ &lt; 7 bar</th>
<th>7 &lt; $q_c$ &lt; 20 bar</th>
<th>$q_c$ &gt; 20 bar</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CL – low-plasticity clay:</strong></td>
<td>3 &lt; $\alpha$ &lt; 8</td>
<td>2 &lt; $\alpha$ &lt; 5</td>
<td>1 &lt; $\alpha$ &lt; 2.5</td>
</tr>
<tr>
<td><strong>ML – low-plasticity loam:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$q_c$ &lt; 20 bar</td>
<td>3 &lt; $\alpha$ &lt; 6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$q_c$ &gt; 20 bar</td>
<td>1 &lt; $\alpha$ &lt; 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>OH – very plastic clay</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>MH–OH – very plastic loam:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$q_c$ &lt; 20 bar</td>
<td>2 &lt; $\alpha$ &lt; 6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$q_c$ &gt; 20 bar</td>
<td>1 &lt; $\alpha$ &lt; 2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
MH-OH soils with qc <2MPa. The low α average is interpreted as being due to the highly compressible nature of the material. Sand α values for samples BH6651, 12.85m to 19.30m, although limited, extend over a wide range (0.8 to 4.8), a feature also observed with the French data. Also presented on Table 3.2 are C values evaluated using Sanglerat's modified version of equation 3.1. For the upper highly compressible unit the low C values when substituted into equation 3.2 produce the expected high values of estimated settlement.

Penetrometer data is not incorporated into the settlement analysis (Chapter 5) because of critical importance to the dewatering model (Figure 2.14), is the consolidation behaviour of the lower Tauranga Group, out of the range of penetrometer soundings.
3.7 **Mine Roof Fall Bulking Assessment**

Since panel 1 under the hostel was sealed at the time of this investigation, underground observations were restricted to assessing the bulking factor (volumetric expansion) of coal measure materials associated with roof falls.

Bulking is determined by the expression:

\[ B = \frac{(V_c - V_o)}{V_o} \quad (\text{Dunrud, 1984}) \]

where

- \( B \) = Bulking factor
- \( V_c \) = Volume of rock before caving
- \( V_o \) = Volume of rock after caving

A roof fall from crosscut 13, panel 4 was chosen for the study. The fall, accessible from all sides was measured with a tape and compass. A plan and sections through the collapsed area are presented on Figure 3.16.

Volumes are approximated using a modified version of Simpson's Rule (Berkman, 1982-p.187). The irregular section areas required for Simpson's Rule are estimated using a polar planimeter.

Bulking at this site is calculated at 28.7% (c.30%). This figure is in agreement with McInally (1983a) who uses a 30% bulking value for calculating maximum caving heights above panel 1. Discussion of bulking factors with reference to void migration are presented in Chapter 5.

Factors to note from mapping of the roof fall in crosscut 13 are that:

a) the fall occurs at the intersection of two mine drives (also observed for roof falls in panel 1 below hostel – Figure 2.3).

b) chimney development is partially defect controlled as suggested by
   - the intersection of two vertical shears in the collapse area
   - the common occurrence of slickensided blocks in the debris pile
   - bedding planes exposed in the southwest section of the chimney
Note that roof fall has occurred at the intersection of both mine roadways and mapped shears.

- Bedding on resin bands in coal
- Minor (up to ~1m) 'fretting' at mid-height of pillar corners
- Blocks in debris pile tabular and commonly slickensided - average block size 10m x 10m x 0.5m
- 45° slope on rock pile
- 40° slope on rock pile
- 33° slope on rock pile
- Roof failure on bedding planes
- Debris pile of adjacent roof fall
- Roof failure in part on bedding planes

LEGEND:
- Cleat orientation (subvertical)
- BC - butt cleat
- FC - face cleat
- Orientation of shears (subvertical unless specified)
- Bedding
- 10° (dip of coal measures)
- Carbonaceous mudstone
- Debris pile
- Coal
- Debris pile contours (metres)
- Fallen debris
- Debris pile contours (metres)
- Mine pillar

Figure 3.16: Plan and sections of roof fall area, cross-cut 13, panel 4, South headings, East Mine (area mapped to estimate bulking factor of fallen coal measure materials)
3.8 Synthesis: Hostel Site Geology and Groundwater Hydrology

3.8.1 Introduction

This synthesis presents an engineering geological description of the Tauranga and Te Kuiti Group sequences encountered during drilling. The description is based on borehole logs presented in Appendix Two from which large scale cross-sections (Figures 3.17 and 3.18)\(^1\) are constructed through the hostel area. Site hydrology is described in terms of the large scale cross-sections, field and laboratory permeability determinations and piezometric data.

3.8.2 Material Classification

Rock and soil material field descriptions are based on a classification scheme proposed by Bell and Pettinga (1983). This classification is preferred to others (N.Z.G.S., 1985; I.A.E.G., 1981) because of its conciseness and 'graphical' presentation, allowing rapid field logging of samples from core and wash drilling.

Laboratory investigations (see Chapter 4) of Tauranga Group materials demonstrates a relationship between soil compressibility and mode of deposition. Due to the importance of soil compressibility to the tentative engineering geological model (Figure 2.14), modification of the Bell and Pettinga scheme for soil materials in terms of their origin is required.

Geological evidence indicates that the Tauranga Group sequence consists of both ignimbritic and fluviatile deposits (Section 3.8.3.3). When these deposits are distinguished within the sequence, the soil material descriptions are preceded by either 'ignimbritic' or 'fluviatile'. Where the depositional mode cannot be determined, for example some sand units sampled by wash drilling, the preceding adjective is omitted from the description.

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\(^1\) Figures 3.17 and 3.18 are in map pocket at back of thesis.
The term 'ignimbritic' as used for description refers to deposits that are formed by large scale, generally rhyolitic, pyroclastic volcanism (Wilson and Walker, 1982). 'Fluvialite' when used in descriptions refers to deposits that are formed by river action.

The advantages of using genetic qualifiers to textural descriptions over introducing classification schemes for each type of deposit (for example adopting the engineering geological classification of ignimbritic deposits by Prebble, 1983) are simplicity through lack of double nomenclature and ease of detailed textural descriptions. Preceding a soil description with a genetic qualifier was adopted by Paterson (1977) who described "ignimbritic silty sands" in the Poutu Tunnel, Tongariro Power Development.

3.8.3 Site Geology

3.8.3.1 Te Kuiti Group Materials

All boreholes of this investigation, apart from BH6657, extend through the Tauranga Group sequence into the top of the Te Kuiti Group succession. The top of the Te Kuiti Group sequence is commonly marked by a 0 to 2.0 metre thick highly weathered silty clay (CH)\textsuperscript{1}. The clay contains rare rootlets and shows a gradational contact with the underlying unweathered, strong to very strong, dark olive and greyish brown, finely layered carbonaceous mudstone.

On the basis of colour and carbonaceous content the mudstone is interpreted as being part of the Waikato Coal Measures. Levelling data from the mine floor of panel 1 indicate that the coal measures underlying the hostels generally dip 8° NW. In the southern section of panel 1 this orientation changes to 8° N. Details of geologic structure mapped in the northeastern section of panel 1 by S.C.M. is summarised in Section 2.3.3.

The absence of other Te Kuiti Group formations over panel 1 indicate that erosion extended into the Waikato Coal

\textsuperscript{1}. Abbreviations according to the Unified Soil Classification System.
Measures prior to the deposition of the Tauranga Group sequence (Figure 1.7).

3.8.3.2 **Te Kuiti Group-Tauranga Group Contact**

One of the principal objectives of this study is to determine relief on the Te Kuiti-Tauranga Group contact by means of drilling. This contact provides a geometric constraint to thickness of Tertiary overburden over mine workings (Section 1.5.5) which is important in terms of void migration resulting from roof falls (Figure 2.14). Structure contours for the Te Kuiti-Tauranga Group contact and isopachs showing coal measures material thicknesses above the mine workings are shown in Figure 3.19.

Paleorelief on the contoured surface is dominated by a buried hill (-10m. R.L.) to the east gently sloping (c. 8°) to a buried valley in the west (-45m. R.L.). On this surface drilling has delineated a east-west striking minor buried valley. Constructed isopachs show that coal measure material thicknesses range from 35 to 50 metres above panel 1. The significance of these thicknesses in terms of void migration resulting from roof falls is discussed in Chapter 5.

3.8.3.3 **Tauranga Group Materials**

Soil materials of the Tauranga Group consist of a 50 to 70 metre interlayered sequence of sands, silts and gravels with minor clay and peat. The sequence is highly pumiceous and typically well graded (poorly sorted). Lithological field relationships are generally complex with lateral and vertical variations over short distances (Figures 3.17 and 3.18).

A. **Ignimbritic deposits**

Ignimbritic deposits are generally either silts or sands. Engineering geological descriptions of these materials are presented in Appendix Two. Ignimbritic sands are identified on the basis of
Figure 3.19: Isopach of coal measures material cover above Panel I and structure contours of Te Kuiti-Tauranga Group contact

Structure contours constructed from borehole data, isopach constructed from structure contours and S.C.M. levelling data of mine floor

buried valley on Te Kuiti-Tauranga Group contact trending approx. E-W

buried hill on Te Kuiti-Tauranga Group contact

LEGEND

isopach of coal measures material cover above panel I in metres (Tertiary overburden)

Structure contours (metres below sea level) on Te Kuiti-Tauranga Group contact

Mine workings

Land title boundaries

Position of cross-sections (Figures 3.17 and 3.18)

6656 Site investigation boreholes

6654 S.C.M. coal exploration boreholes
field texture and rhyolitic 'composition' (pumice rich with lesser amounts of quartz, feldspar and rhyolite rock fragments). These sandy deposits are generally well graded (poorly sorted), massive, lack internal structure and contain charcoal and flattened pumice fragments (Figure 3.20). Two laterally extensive units are identified within the Tauranga Group sequence (Figures 3.17, 3.18). Upper and lower contacts of these units are generally subhorizontal. The upper unit is within the Karapiro Formation and the lower unit in the Whangamarino Formation.

Ignimbritic silts are generally identified on the basis of their halloysitic composition, textural uniformity (Figure 3.21) and extensive nature (pers. comm. C. Nelson, 1985). Two major layers of ignimbritic silts are recognised within the Tauranga Group sequence. The upper layer which generally follows relief over the hostel site is up to c.10m thick comprising the Kauroa-Hamilton Ashes. The lower layer is within the Whangamarino Formation, and is generally flat lying, ranging in thickness from 4 to 10 metres.

Three 'distal ignimbrite' deposits, including both sands and silts have been described in the Whangamarino Formation at Ohinewai (6km north of hostel site) by Todd (1982a). It is possible that the ignimbritic deposits within the Whangamarino Formation at the hostel site are lateral equivalents to those described by Todd. No ignimbritic deposits have been previously described from the Karapiro Formation. The upper most ignimbritic silt unit interpreted as comprising the Kauroa-Hamilton Ashes are regionally extensive over the Lower Waikato Basin (Ward, 1967).

B) Fluviatile deposits

Fluviatile deposits are identified on the basis of fine to coarse layering (bedding and lamination), the greywacke component of material composition and association with peat and gravel (Figure 3.22). Texturally these deposits range from clayey silts to medium gravels and comprise the non-ignimbritic component of the Whangamarino, Karapiro and Hinuera Formations.

1. "Greywacke" is used here as a field term and refers to the highly inurated sandstones and siltstones of the
Figure 3.20: Upper part of a c.4m thick ignimbritic SILTY fine to medium SAND (SC), BH6651, Whangamarino Formation. Note the massive nature of the deposit, flattened pumice fragments (fp) and minor charcoal (c).

Figure 3.21: Part of a c.10m thick ignimbritic fine to medium SANDY SILT with some clay (ML), lower ignimbritic silt unit, BH6651, Whangamarino Formation. The outside of the cone has oxidized from the original light greyish green (seen in part) to the whitish yellow. Note the materials massive nature.
Figure 3.22: Fluvial interlayered sequence of fine to medium GRAVELS (GM), composed of reworked Tauranga Group and greywacke clasts, SILTY CLAY, SILTY medium SAND (SM) and wood fragments (at 44.20m), BH6651, Whangamarino Formation. Note the fine and coarse layering (bedding) controlling material variability.
Engineering geological descriptions of fluviatile materials are presented in Appendix Two. Bedding attitudes are variable within these materials and range from subhorizontal up to 10°.

The fluviatile nature of materials within the Whangamarino, Karapiro and Hinuera Formations has been well documented regionally by Kear and Schofield (1978) and at Ohinewai by Todd (1982a).

C. Stratigraphic Correlations

The major stratigraphic units recognised within the succession are the Whangamarino Formation, Karapiro Formation and the Kauron-Hamilton Ashes (Figures 3.17 and 3.18). The Waeranga Gravels (of greywacke composition) although recognised, are not described separately as they are interlayered and partially transitional with both the Whangamarino and Karapiro Formations.

1. Whangamarino Formation

Unconformably overlying the coal measures is a 20 to 35 metre thick sequence of light bluish-green, olive-white and olive-brown, interlayered ignimbritic sands and silts plus fluviatile silts, sands and gravels with rare peat. Intersected in boreholes 6651 to 6656 at the top of this sequence is a c. 0.2m thick highly to completely weathered, organic rich, dark brown clayey silt.

On the basis of colour and stratigraphic position this sequence is interpreted as being part of the Whangamarino Formation. The weathered silt which marks the upper boundary is interpreted as representing a paleosol.

This formation is extensive being described regionally by Kear and Schofield (1978), at Ohinewai by Todd (1982a) and over the north headings of Huntly East Mine by Todd (1982b).

2. Karapiro Formation

Overlying the paleosol at the top of the Whangamarino Formation is a 10 to 40 metre thick sequence of light and dark, grey and brownish yellow interlayered ignimbritic
sands and fluviatile silts, sands, gravels with rare peat.

This succession is interpreted as being part of the Karapiro Formation on the basis of colour and stratigraphic position. As with the Whangamarino Formation this unit is extensive being described regionally by Kear and Schofield (1978), at Ohinemui by Todd (1982a), and over the north headings of Huntly East Mine by Todd (1982b).

3. **Kauroa-Hamilton Ashes**

Mantling the Karapiro Formation over the hostel site is a c.4 to 10 metre thick interlayered sequence of commonly mottled, light reddish-yellow and yellowish-brown ignimbritic silts with minor silty sands.

On the basis of colour, composition and stratigraphic position this sequence is interpreted as being part of the Kauroa-Hamilton Ashes described by Ward (1967). Ward reports 4-5 metres of Kauroa and Hamilton Ash at Huntly as part of his extensive study over the Lower Waikato Basin. Distinguishing between the ash formations is beyond the scope of this study.

4. **Hinuera Formation**

Overlying the Karapiro Formation and Kauroa-Hamilton Ashes and extensive under low lying areas north and west of the hostel area is an interlayered sequence of light yellowish brown and greyish green fluviatile silts, sands, gravels and peat.

On the basis of stratigraphic position and colour, this sequence is interpreted as being part of the Hinuera Formation. Due to compositional similarity with the underlying Karapiro Formation, the base of the Hinuera Formation is not easily recognised.

Kear and Schofield (1978) map the extensive flat area west of the hostel as an aggradation surface related to deposition of the Taupo Pumice Alluvium. The alluvium is not present in boreholes and was possibly largely removed with residential development.
3.8.4 **Groundwater Hydrology**

Combining the large scale cross-sections (Figures 3.17 and 3.18) with field (Section 3.5.2) and laboratory (Section 4.6) permeability data, the groundwater hydrology of the site can be described.

Three aquifers\(^1\) are recognised within the Tauranga Group sequence. Described as the upper, middle and lower, aquifer permeabilities are estimated as ranging from \(10^{-2} - 10^{-3}\) m/s for gravels (estimate from Terzaghi and Peck, 1967, p.55) to \(10^{-5} - 10^{-7}\) m/s for sands. Aquifers are separated by two silt aquitards with estimated permeabilities of \(10^{-7} - 10^{-10}\) m/s. The Waikato Coal Measures at the base of the Tauranga Group sequence with estimated permeabilities of \(10^{-8} - 10^{-10}\) m/s (Table 1.2) represent an aquiclude. Localised zones within the coal measures have higher permeabilities due to the presence of joints and faults (B.M.C., 1984) or possibly mining induced fractures as suggested by the tentative engineering geological model in Figure 2.14.

Cross-section analysis (Figures 3.17 & 3.18) indicates that the lower aquifer is confined with the middle and upper aquifers laterally extensive and possibly unconfined. The lower and middle aquifers are within the Whangamarino Formation with the upper aquifer consisting of the Karapiro and Hinuera Formations.

Todd (1982b) in his groundwater study over the northern headings of the East Mine reports a similar sequence with two aquifers in the Whangamarino Formation and a third being represented by the Karapiro and Hinuera Formations. In two areas over the north headings Todd notes that the middle and upper aquifers are in direct contact. B.M.C. (1984), because of hydraulic continuity between the upper and middle aquifers describe the Tauranga Group in terms of two aquifer systems (Figure 3.23). The two aquifer systems are discussed in Section 1.5.4.

The phreatic surface of the upper aquifer at the hostel site based on S.C.M. monitoring data is present in Figure 3.24. The phreatic surface slopes to the north west over the hostel site and is horizontal (c. 2m deep) below

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1. See Appendix One for definitions of hydrological terms.
Figure 3.23: Schematic cross-section showing the general relationship between aquifers and aquitards in the Tauranga Group sequence over the northern and southern headings, Huntly East Mine.
Figure 3.24: Phreatic surface of upper aquifer over hostel site. Constructed from average S.C.M. piezometer water levels over September, 1983.
the flat area of Rosser Street. Springs (Section 3.2.1) at the base of the slope below the hostel represent areas where the phreatic surface is in contact with the ground surface.
CHAPTER IV

LABORATORY INVESTIGATIONS

4.1 Laboratory Investigation Objectives and Approach

To augment field investigations, laboratory testing was conducted to further quantify the tentative engineering geological model (Figure 2.14) for geotechnical analysis (Chapter Five). Objectives of the laboratory investigations are:

1) to characterise materials in terms of their grain size distribution, clay mineralogy, density, void ratio and natural water content.

2) to determine material consolidation parameters.

3) to determine material permeabilities.

4) to study material fabric as it relates to soil compressibility.

The laboratory testing programme was generally restricted to samples from the fully cored borehole 6651, located in the maximum area of subsidence. Limited consolidation and material characterisation tests were completed on samples from borehole 6652 and 6653 located on the outer margins of the hostel subsidence trough. This approach allows the identification of any possible compressible units within the Tauranga Group sequence with respect to this subsidence area. Laboratory data is used in the back analysis procedures of Chapter Five to investigate the likely dewatering consolidation component of the hostel subsidence.

Laboratory testing has been undertaken by a number of investigators. All permeability, the majority of the consolidation and material characterisation tests were completed as part of this study. The remainder of the
consolidation tests were carried out by the Hamilton District Laboratory of the M.W.D. (1985 a,b), with material characterisation testing being completed by Wezenberg (1985) under supervision of the author. All other data, including laboratory investigations (M.W.D., 1983) on core from M.W.D. borehole 6599 (see Section 2.4) is reviewed in this chapter.

4.2 Sample History

To minimise sample disturbance and changes of moisture content, core (150mm diameter) retrieved during drilling (Section 3.3) was initially placed in split p.v.c. garnite tubing, wrapped in plastic and stored under refrigeration. During transport from Huntly to Christchurch the wrapped core was supported by wet wood shavings and on arrival stored in a humidity room until testing. All materials apart from loose gravels and poorly graded fine sands arrived in Christchurch in an undisturbed condition. Selected samples of core were transported from Huntly to Christchurch. The remaining material from the drilling programme is stored at Huntly by S.C.M.

4.3 Material Characterisation

4.3.1 Test procedures and results

Laboratory procedures for grain size analyses, natural moisture contents and Atterberg Limits are in accordance with N.Z. Standards 4402, Part 1 (1980). Due to the suspected presence of allophane, samples used for Atterberg Limit determinations were maintained at or above their natural moisture contents. Determinations of void ratios, dry and bulk densities were made in conjunction with one-dimensional tests following procedures described in N.Z.S. 4402, Part 2P, Test 21 (1981). Clay mineralogy was determined using an X-ray Diffractometer (X.R.D.) and the sodium fluoride reactivity test (N.Z.S. 4402, Part 1, Test 13, 1980) for allophane content. Solid densities were determined using a technique especially designed for
pumiceous materials, and originally developed by the M.O.W. (1958).

Classification tests completed on ignimbritic and fluviatile materials from boreholes 6651 and 6599 are presented in Tables 4.1 and 4.2. Envelopes representing ranges of grading curves for both material types are presented in Figures 4.1, a to c. Saturation ratios are presented with consolidation data in Tables 4.3 and 4.4.

4.3.2 Interpretation and Discussion

4.3.2.1 Void ratios, densities and moisture contents

The Tauranga Group sequence is generally characterised by high void ratios (average \(e\) is 1.37), low dry densities (average \(\rho_d\) is 1.01 t/m\(^3\)) and high moisture contents (average \(w\) is 53%) which are indicative of 'open' structured materials with a large component of void space.

The 'open' structured nature of deposits is interpreted as being due to:

1) the pumice component of the sand to fine gravel fraction within ignimbritic and fluviatile materials.

2) the 'loose' packing of the silt fraction (Section 4.5) within ignimbritic materials.

3) the presence of allophane in the clay fraction (Section 4.3.2.4) within ignimbritic and fluviatile materials.

Notable in terms of their 'open' texture are the ignimbritic sandy silts and silty sands with clay-silt contents of greater than 53% (BH6651, at 7.21m, 9.80m, 55.28m and 55.50m - Table 4.1) and whose void ratios range between 2.23 and 2.77. These values are contrasted by void ratios of the fluviatile clayey and sandy silts (BH6651, at 34.9m and 61.9m) which are measured at 0.84 and 0.77 respectively.

High void ratios combined with low dry densities and
TABLE 4.1: CLASSIFICATION TEST RESULTS FOR IGNEOUS AND FLUVIATE MATERIALS FROM BOREHOLE 6651

<table>
<thead>
<tr>
<th>MATERIAL FIELD DESCRIPTION</th>
<th>Depth (m)</th>
<th>Moisture Content (w %)</th>
<th>Dry Density (t/m³)</th>
<th>Solid Density (t/m³)</th>
<th>Void Ratio</th>
<th>Particle size analysis (%)</th>
<th>Atterberg Limits (%)</th>
<th>CLAY MINERALOGY</th>
<th>X-ray Diffraction Analysis</th>
<th>ALLOPHANE TEST</th>
</tr>
</thead>
<tbody>
<tr>
<td>Igneous materials:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pumiceous SANDY SILT with some clay (K-8) (3)</td>
<td>7.21</td>
<td>91.0</td>
<td>0.83</td>
<td>2.66</td>
<td>2.23</td>
<td>7 48 45</td>
<td>83 51 32</td>
<td>Halloysite (2)</td>
<td>Halloysite (1) and unidentified mixed layer clay</td>
<td>c. 5-7%</td>
</tr>
<tr>
<td>Pumiceous SILT fine to medium SAND with some clay (K-8)</td>
<td>9.80</td>
<td>88.4</td>
<td>0.80</td>
<td>2.65</td>
<td>2.33</td>
<td>12 41 47</td>
<td>51 38 13</td>
<td></td>
<td></td>
<td>N.T.</td>
</tr>
<tr>
<td>Pumiceous SILT fine to medium SAND (Kp)</td>
<td>13.0</td>
<td>50.0</td>
<td>1.08</td>
<td>2.47</td>
<td>1.27</td>
<td>2 35 63</td>
<td>N.O. (4)</td>
<td>N.T.</td>
<td></td>
<td>N.T.</td>
</tr>
<tr>
<td>Pumiceous fine to medium SAND with some silt (Kp)</td>
<td>19.30</td>
<td>46.0</td>
<td>1.12</td>
<td>2.48</td>
<td>1.22</td>
<td>- 15 85</td>
<td>N.O. (4)</td>
<td>N.T.</td>
<td></td>
<td>N.T.</td>
</tr>
<tr>
<td>Pumiceous fine to medium SAND with some fine gravel and silt (Kp)</td>
<td>22.60</td>
<td>47.4</td>
<td>1.13</td>
<td>2.44</td>
<td>1.16</td>
<td>2 11 82 5</td>
<td>N.O. (4)</td>
<td>N.T.</td>
<td></td>
<td>N.T.</td>
</tr>
<tr>
<td>Pumiceous SILT fine to medium SAND with some clay (Ug)</td>
<td>40.20</td>
<td>61.3</td>
<td>0.97</td>
<td>2.52</td>
<td>1.59</td>
<td>6 29 63 2</td>
<td>N.O. (4)</td>
<td>N.T.</td>
<td></td>
<td>N.T.</td>
</tr>
<tr>
<td>Pumiceous SILT fine to coarse SAND with some fine gravel (Ug)</td>
<td>43.70</td>
<td>65.9</td>
<td>0.92</td>
<td>2.39</td>
<td>1.60</td>
<td>- 17 75 8</td>
<td>N.O. (4)</td>
<td>N.T.</td>
<td></td>
<td>N.T.</td>
</tr>
<tr>
<td>Pumiceous SANDY SILT with some clay (Ug)</td>
<td>55.28</td>
<td>98.0</td>
<td>0.74</td>
<td>2.69</td>
<td>2.66</td>
<td>5 60 35</td>
<td>N.T.</td>
<td>Halloysite and kaolin</td>
<td></td>
<td>c. 5-7%</td>
</tr>
<tr>
<td>Pumiceous SANDY SILT with some clay (Ug)</td>
<td>55.50</td>
<td>103.5</td>
<td>0.71</td>
<td>2.69</td>
<td>2.77</td>
<td>10 55 35</td>
<td>67 55 12</td>
<td>Halloysite and kaolin</td>
<td></td>
<td>c. 5-7%</td>
</tr>
<tr>
<td>Pumiceous SILT fine to medium sand (Ug)</td>
<td>59.00</td>
<td>70.5</td>
<td>0.89</td>
<td>2.55</td>
<td>1.85</td>
<td>2 35 63</td>
<td>N.O. (4)</td>
<td>N.T.</td>
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<td>N.T.</td>
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<td>Fluvialite materials:</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Organic CLAY Silt with some sand (Ug)</td>
<td>34.9</td>
<td>32.0</td>
<td>1.42</td>
<td>2.62</td>
<td>0.84</td>
<td>35 39 26</td>
<td>43 21 22</td>
<td>Unidentified mixed layer clay (plus tridymite)</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>SANDY Silt with some clay</td>
<td>34.9</td>
<td>32.0</td>
<td>1.42</td>
<td>2.62</td>
<td>0.84</td>
<td>35 39 26</td>
<td>43 21 22</td>
<td>Unidentified mixed layer clay</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>fine gravel and wood fragments (Ug)</td>
<td>61.9</td>
<td>29.3</td>
<td>1.49</td>
<td>2.63</td>
<td>0.77</td>
<td>8 52 35 5</td>
<td>52 23 29</td>
<td>Unidentified mixed layer clay</td>
<td>5</td>
<td></td>
</tr>
</tbody>
</table>

1 Moisture contents, dry densities and initial void ratios determined in conjunction with one-dimensional consolidation testing.

2 Grain size analyses, Atterberg limits and Clay Mineralogy determinations as part of this study. Evaluations below the first row are by Wienen (1985). Note that Wienen's Atterberg limits are determined on whole sample while values determined as part of this study are from the silt-clay fraction.

3 Abbreviations for stratigraphy: (K-8) = Kaurun-Hamilton Ashes (Kp) = Karapiro Formation (Ug) = Whangam我们的 Formation

4 Abbreviations: N.O. = not available N.T. = not tested.
<table>
<thead>
<tr>
<th>Material Field Description</th>
<th>Depth (m)</th>
<th>Moisture Content (w %)</th>
<th>Solid Density (t/m³)</th>
<th>Void Ratio e</th>
<th>Particle Size Analysis (%)</th>
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</thead>
<tbody>
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<td>Ignimbritic materials:</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pumiceous SILTY fine to coarse SAND with some clay and gravel (K-H)</td>
<td>1.24-3.04</td>
<td>42.7</td>
<td>2.71</td>
<td>1.16</td>
<td>20 75 5</td>
</tr>
<tr>
<td>Pumiceous SILTY fine to coarse SAND (K-H)</td>
<td>4.10-5.70</td>
<td>33.3</td>
<td>2.69</td>
<td>0.90</td>
<td>28 70 2</td>
</tr>
<tr>
<td>Pumiceous SILTY fine to medium SAND (Kp)</td>
<td>13.00-16.54</td>
<td>49.0</td>
<td>2.44</td>
<td>1.20</td>
<td>40 60 -</td>
</tr>
<tr>
<td>Pumiceous SILTY fine to medium SAND (Kp)</td>
<td>16.54-26.25</td>
<td>39.2</td>
<td>2.46</td>
<td>0.96</td>
<td>18 79 3</td>
</tr>
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<td>Fluvialite materials:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium to coarse SANDY fine to medium GRAVEL with some silt (Kp)</td>
<td>5.70-6.58</td>
<td>36.4</td>
<td>2.70</td>
<td>0.98</td>
<td>11 34 55</td>
</tr>
<tr>
<td>Pumiceous SILTY medium to coarse SAND with some silt (Kp)</td>
<td>6.58-13.00</td>
<td>35.0</td>
<td>2.67</td>
<td>0.93</td>
<td>14 82 4</td>
</tr>
<tr>
<td>Medium to coarse SANDY fine to medium GRAVELS (Kp)</td>
<td>38.19-40.25</td>
<td>21.0</td>
<td>2.59</td>
<td>0.54</td>
<td>2  29 69</td>
</tr>
<tr>
<td>Pumiceous fine GRAVELLY medium to coarse SAND (Kp)</td>
<td>40.35-41.71</td>
<td>88.9</td>
<td>2.34</td>
<td>2.08</td>
<td>5  80 15</td>
</tr>
<tr>
<td>Fine to medium GRAVELLY medium to coarse SAND (Wg)</td>
<td>45.25-48.55</td>
<td>31.3</td>
<td>2.66</td>
<td>0.83</td>
<td>8  70 22</td>
</tr>
<tr>
<td>Pumiceous SILTY fine to medium SAND (Wg)</td>
<td>48.55-50.45</td>
<td>35.7</td>
<td>2.67</td>
<td>0.95</td>
<td>14 86 -</td>
</tr>
<tr>
<td>Medium to coarse SANDY fine to medium GRAVELS (Wg)</td>
<td>50.45-52.00</td>
<td>26.9</td>
<td>2.62</td>
<td>0.70</td>
<td>2  43 55</td>
</tr>
</tbody>
</table>
Figure 4.1a: Range of particle grading curves for ignimbritic materials.

Figure 4.1b: Range of particle grading curves for fluvialite materials.
Figure 4.1c: Comparison of particle grading curves from ignimbritic and fluviatile materials.
high water contents are typical of ignimbritic soil materials (Prebble, 1983; Norrthey, 1966). Parton and Olsen (1980) in their study of Bay of Plenty volcanic soils measure void ratios of up to 5 with natural moisture contents in excess of 150%.

4.3.2.2 Particle size distributions

Particle size distribution curves (Figure 4.1 a,b) show the well graded nature of the ignimbritic and fluvial deposits, and that fines within the whole sequence are generally dominated by silt size particles. Grain size envelopes for both material categories (Figure 4.1 c) have a large 'area' of overlap which, together with their pumiceous content (Table 4.2) indicates that the fluviatile materials are in part the reworked (by river action) equivalents of the ignimbritic deposits.

4.3.2.3 Atterberg Limits

Igminbritic sandy silts and silty sands are MH (Unified Soil Classification System) type materials with high plasticity, and fluviatile clayey and sandy silts CL and CH type soils with low to high plasticity.

It is generally accepted that soil materials with liquid limits greater than 50% exhibit high compressibility (Terzaghi and Peck, 1967; White, 1982). This relationship suggests that the ignimbritic silts (LL = 83% and 67% for BH 6651, at 7.21m and 55.50m) are highly compressible in comparison to the fluviatile silts (LL = 43% and 52% for BH 6651, at 34.9m and 61.9m). High liquid limits are commonly associated with allotphanic soils (see Section 4.3.2.4) as described by Gradwell and Birrell (1954) in their study on the physical properties of volcanic clays.

4.3.2.4 Clay mineralogy

Allophane is common to all fine grained materials tested in the Tauranga Group sequence. Within all ignimbritic silt rich materials halloysite is identified and
is associated with kaolin at depth (BH6651, 55.28m and 55.50m - Table 4.1). Within the fluviatile fine grained materials the allophane is present with an unidentified mixed layer clay. The possible mineralogies of the mixed layer clay as suggested by Wezenberg (1985) are either kaolin-smectite, or halloysite-vermiculite, or smectite-illite.

Studies of allophane (Henmi and Wada, 1976) have shown that the mineral consists of irregular aggregates of hollow, spherical shaped particles. The walls of the irregular spheres are radially partitioned forming 'tunnel-like' pores which allow the passage of water (Lowe and Nelson, 1983). As a result of this 'open' and 'bulky' structure, allophanic soils commonly have high natural water contents and low dry densities (Oborn et al., 1982). These properties are observed particularly within the ignimbritic silt rich units of the Tauranga Group succession (Section 4.3.2.1).

Allophane, halloysite and kaolin are common components of weathered volcanic materials in New Zealand (Gradwell and Birrell, 1954; Prebble, 1983).

4.4 Consolidation Testing

4.4.1 Consolidation parameters

When a load is applied to a saturated sample, it is initially supported by the incompressible pore fluid. The resulting increase in pore pressure is called hydrostatic excess pressure. With time the excess pore pressure dissipates as the water drains from the sample, with the load being gradually transferred to the compressible soil skeleton. Drainage and accompanying dissipation of excess pore pressure is controlled by the soils permeability. The transfer of load accompanied by a reduction in the soil mass volume (equal to the volume of water drained) is known in soil mechanics as consolidation (Terzaghi, 1925).

The conventional approach of characterising the rate of consolidation from the oedometer tests (Section 4.4.2) is shown in Figure 4.2. This figure plots the relationship
**Figure 4.2**: Definition of Primary and Secondary Compression Stages (adapted from Mesri, 1985).

**Figure 4.3**: Estimation of $C_V$ for the 1600 kPa load increment on a sample from the upper ignimbritic silt unit (BH6651, 6.50 m) using the empirical logarithm of time fitting method (Taylor, 1948).
between reduction in soil mass volume (expressed as a decrease in void ratio) with time for one load increment. Compression can be divided into two stages (Crawford, 1965):

1) primary compression that occurs while the excess pore pressure dissipates \( \frac{d\bar{\sigma}}{dt} \neq 0 \)

2) secondary compression that occurs after excess pore pressure has dissipated \( \frac{d\bar{\sigma}}{dt} = 0 \)

The coefficient of consolidation \( (c_v) \), commonly used to calculate settlement rates is evaluated for the same curve using the logarithm of time fitting method of Taylor (1948) shown in Figure 4.3. An equation is then used to evaluate \( c_v \). Where secondary compression is highly developed \( c_v \) can be determined (for example Figure 4.3) but cannot be used for accurate predictions of the rate or final magnitude of settlement (Lambe and Whitman, 1979).

Secondary compression has a characteristic linear e-log \( t \) relationship with a slope represented by \( C_\alpha \) (rate of secondary compression).

The conventional approach of relating volumetric changes of the sample with increasing load is by plotting an e-log \( \bar{\sigma} \) curve as shown in Figure 4.4. Estimations of the preconsolidation pressure \( (\bar{\sigma}_c) \), which indicates the past maximum effective stress, and compression index \( (C_c) \), which measures material compressibility can be made from the e-log \( \bar{\sigma} \) plot. The coefficient of compressibility \( (a_v) \) is evaluated from plots of void ratio versus effective stress.

4.4.2 Test procedure and results

One-dimensional consolidation tests were performed in Casagrande oedometers on selected samples from BH6651, 6652 and 6653. Testing priority was given to the ignimbritic materials with high void ratios (Section 4.3.2.1). Test apparatus is shown in Figures 4.5 and 4.6. Procedures for the consolidation testing are in accordance with N.Z.S. 4402, Part 2P, Test 21 (1981). All consolidation testing apart from that carried out by M.W.D. (1985a,b), was
Figure 4.4: Estimation of Compression Index ($C_c$) and Preconsolidation Pressure ($\sigma_c$) from $e$ - $\log \sigma$ plot (after Casagrande, 1936; Lambe and Whitman, 1979)
Figure 4.5: One-dimensional consolidation test apparatus. The load is applied by calibrated weights (painted green) through a lever arm to the oedometer cell (c) which contains the sample. Axial strain is measured by a dial gauge (accurate to 0.001mm).

After BS1377 [5.2]

Figure 4.6: A section through a typical oedometer cell (from Scott [1980]).
completed in the Geomechanics Laboratory, Department of Civil Engineering, University of Canterbury.

Samples (76mm diameter x 19mm thick) were trimmed into the oedometer rings at their 'natural' moisture content from the 150mm diameter core. No difficulties were experienced during sample preparation. For the majority of tests, loads were applied every 24 hours, each test taking 8 days to complete. To investigate the duration of secondary compression, loads on 3 ignimbritic silt samples were applied every 7 days, the tests taking 50 days to complete. Loading for shallow samples in the Taupunga Group sequence ranged from 25 to 1600 kPa and for deeper samples 25 to 3200 kPa. All six oedometers used were calibrated for machine strain prior to operation.

Test results in terms of the consolidation parameters described in Section 4.4.1 plus the initial and final soil properties are presented in Table 4.3 and 4.4. Consolidation work sheets and plots of the e-log $\bar{\sigma}$ relationship for each sample are presented in Appendix 5.

The standard graphical procedures of estimating $c_v$ (Section 4.4.1) within the ignimbritic silt rich materials were unsuccessful due to high material permeability causing primary compression to be complete before the initial 6 second dial gauge reading. Using a L.V.D.T. (Linearised Voltage Displacement Transducer) and a chart recorder which provided a continuous record of axial strain with time, $c_v$ was able to be evaluated. A graph of axial deflection versus the logarithm of time using this procedure is shown in Figure 4.3. This figure shows that for the 1600 kPa load increment, primary compression is complete after 0.3 seconds and that secondary compression is highly developed. A high degree of secondary compression is observed for all ignimbritic silt rich materials during the oedometer tests.

4.4.3 Interpretation and discussion

The test results show that the ignimbritic silts are highly compressible ($C_c = 0.76$ to 1.63 - Table 4.5) once their preconsolidation pressures ($\tau_C$) are exceeded. In terms of virgin compression indexes, least compressible within the
<table>
<thead>
<tr>
<th>MATERIAL DESCRIPTION</th>
<th>Depth (m)</th>
<th>Water Content (w)</th>
<th>Dry Density (t/m³)</th>
<th>Void Ratio</th>
<th>Degree of Saturation Dr (%)</th>
<th>Compression Index Cc</th>
<th>Preconsolidation Pressure P0 (kPa)</th>
<th>Coefficient of Compressibility αv (kPa⁻¹)</th>
<th>Water Content (w)</th>
<th>Dry Density (t/m³)</th>
<th>Void Ratio</th>
<th>Degree of Saturation Dr (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Igneous breccia SHAIRE SILT with minor clay (B-8)</td>
<td>h</td>
<td>6.08</td>
<td>66.9</td>
<td>0.22</td>
<td>2.39</td>
<td>92.3</td>
<td>1.16</td>
<td>245</td>
<td>56.2</td>
<td>1.11</td>
<td>1.47</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>”</td>
<td>6.5</td>
<td>79.3</td>
<td>0.05</td>
<td>2.26</td>
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<td>0.97</td>
<td>365</td>
<td>55.6</td>
<td>1.12</td>
<td>1.64</td>
<td>100</td>
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<tr>
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<td>6.88</td>
<td>78.9</td>
<td>0.07</td>
<td>2.15</td>
<td>100</td>
<td>0.91+</td>
<td>360+</td>
<td>56.8</td>
<td>1.10</td>
<td>1.49</td>
<td>100</td>
</tr>
<tr>
<td></td>
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<td>7.21</td>
<td>91.0</td>
<td>0.03</td>
<td>2.13</td>
<td>100</td>
<td>1.01</td>
<td>330</td>
<td>37.0</td>
<td>1.11</td>
<td>1.39</td>
<td>100</td>
</tr>
<tr>
<td>Igneous breccia SILTY fill to medium SAND with some clay (K-8)</td>
<td>h</td>
<td>5.03+</td>
<td>88.4</td>
<td>0.795</td>
<td>2.33</td>
<td>100</td>
<td>1.40+</td>
<td>210+</td>
<td>51.0</td>
<td>1.18</td>
<td>1.25</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>”</td>
<td>12.50+</td>
<td>77.0</td>
<td>0.085</td>
<td>2.03</td>
<td>98.3</td>
<td>0.87+</td>
<td>180+</td>
<td>47.3</td>
<td>1.21</td>
<td>1.14</td>
<td>100</td>
</tr>
<tr>
<td>Igneous breccia SILTY fill to medium SAND (Kg)</td>
<td>h</td>
<td>12.85</td>
<td>50.0</td>
<td>1.09</td>
<td>1.27</td>
<td>97.0</td>
<td>0.21</td>
<td>670</td>
<td>48.0</td>
<td>1.34</td>
<td>1.17</td>
<td>100</td>
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<td>”</td>
<td>13.59</td>
<td>49.4</td>
<td>1.09</td>
<td>1.22</td>
<td>98.0</td>
<td>0.16</td>
<td>680</td>
<td>47.7</td>
<td>1.14</td>
<td>1.13</td>
<td>100</td>
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<tr>
<td>Igneous breccia Silt (Kg)</td>
<td>h</td>
<td>19.30</td>
<td>46.0</td>
<td>1.12</td>
<td>1.22</td>
<td>94.0</td>
<td>0.13</td>
<td>660</td>
<td>46.0</td>
<td>1.17</td>
<td>1.11</td>
<td>100</td>
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<td>47.4</td>
<td>1.13</td>
<td>1.16</td>
<td>99.4</td>
<td>0.36</td>
<td>3120</td>
<td>46.0</td>
<td>1.20</td>
<td>1.03</td>
<td>100</td>
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<tr>
<td>Igneous breccia SANDY SILT</td>
<td>h</td>
<td>27.85</td>
<td>89.5</td>
<td>0.75</td>
<td>2.23</td>
<td>97.5</td>
<td>1.15</td>
<td>975</td>
<td>73.6</td>
<td>0.88</td>
<td>1.77</td>
<td>100</td>
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<tr>
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<td>”</td>
<td>28.65</td>
<td>75.8</td>
<td>0.81</td>
<td>2.03</td>
<td>91.7</td>
<td>0.83</td>
<td>990</td>
<td>61.8</td>
<td>1.00</td>
<td>1.66</td>
<td>100</td>
</tr>
<tr>
<td>Fluvialite CLAY</td>
<td>Silt with some sand (Kg) (Palesol)</td>
<td>h</td>
<td>34.90</td>
<td>32.0</td>
<td>1.42</td>
<td>0.84</td>
<td>99.6</td>
<td>0.35</td>
<td>1290</td>
<td>28.9</td>
<td>1.53</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td>”</td>
<td>60.10</td>
<td>61.3</td>
<td>0.97</td>
<td>1.59</td>
<td>97.2</td>
<td>1.04</td>
<td>1370</td>
<td>50.6</td>
<td>1.18</td>
<td>1.14</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>”</td>
<td>63.70</td>
<td>65.9</td>
<td>0.92</td>
<td>1.60</td>
<td>98.2</td>
<td>0.76</td>
<td>2000</td>
<td>65.7</td>
<td>0.99</td>
<td>1.40</td>
<td>100</td>
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<tr>
<td>Igneous breccia SANDY SILT with some clay (Kg)</td>
<td>h</td>
<td>51.05</td>
<td>74.4</td>
<td>0.885</td>
<td>2.07+</td>
<td>100+</td>
<td>1.14+</td>
<td>1380+</td>
<td>665</td>
<td>1.02+</td>
<td>1.46+</td>
<td>100</td>
</tr>
<tr>
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<td>”</td>
<td>55.28</td>
<td>98.0</td>
<td>0.74</td>
<td>2.66</td>
<td>99.0</td>
<td>1.63</td>
<td>605</td>
<td>64</td>
<td>1.93</td>
<td>1.62</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>”</td>
<td>55.40</td>
<td>103.5</td>
<td>0.71</td>
<td>2.77</td>
<td>100</td>
<td>1.60</td>
<td>930</td>
<td>78.4</td>
<td>0.89</td>
<td>2.01</td>
<td>100</td>
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<tr>
<td></td>
<td>”</td>
<td>55.92</td>
<td>101.4</td>
<td>0.72</td>
<td>2.75</td>
<td>99.0</td>
<td>1.62</td>
<td>500</td>
<td>66.2</td>
<td>1.04</td>
<td>1.58</td>
<td>100</td>
</tr>
<tr>
<td>Igneous breccia SILTY fill to medium SAND (Kg)</td>
<td>h</td>
<td>58.89</td>
<td>70.5</td>
<td>0.89</td>
<td>1.85</td>
<td>97</td>
<td>0.97</td>
<td>795</td>
<td>48.6</td>
<td>1.15</td>
<td>1.20</td>
<td>100</td>
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<td>”</td>
<td>58.95</td>
<td>62.7</td>
<td>0.96</td>
<td>1.66</td>
<td>96.5</td>
<td>0.76</td>
<td>935</td>
<td>49.1</td>
<td>1.16</td>
<td>1.21</td>
<td>100</td>
</tr>
<tr>
<td>Fluvialite CLAY</td>
<td>Silt with some sand (Kg)</td>
<td>h</td>
<td>60.95</td>
<td>77.0</td>
<td>0.85</td>
<td>1.91</td>
<td>99.1</td>
<td>1.59</td>
<td>1920</td>
<td>64.0</td>
<td>0.97</td>
<td>1.53</td>
</tr>
<tr>
<td></td>
<td>”</td>
<td>61.85</td>
<td>29.3</td>
<td>1.49</td>
<td>0.77</td>
<td>100</td>
<td>0.44</td>
<td>1390</td>
<td>28.7</td>
<td>1.59</td>
<td>0.66</td>
<td>100</td>
</tr>
<tr>
<td>SILT CLAY (weathered Volcano Calcareous Mudstone)</td>
<td>h</td>
<td>66.20</td>
<td>24.0</td>
<td>1.66</td>
<td>0.58</td>
<td>100</td>
<td>0.18</td>
<td>1130</td>
<td>21.0</td>
<td>1.75</td>
<td>0.50</td>
<td>100</td>
</tr>
</tbody>
</table>

* Horizontal and vertical orientations of the flat circular surfaces of the cylindrical samples with respect to the Tauarunga Group sequence.

+ Tests carried out with one week loading cycles.

# Stratigraphic abbreviations: (B-8) = Beaconsfield-Bayshane (Kg) = Kerepore formation (Kg) = Wanganuiteri formation

# Consolidation tests by the Hamilton District Laboratories, W.N.D.
### Table 4.4: Consolidation Test Results for Boreholes 6652 and 6653

<table>
<thead>
<tr>
<th>MATERIAL DESCRIPTION</th>
<th>INITIAL PROPERTIES</th>
<th>TEST RESULTS</th>
<th>FINAL PROPERTIES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Depth (m)</td>
<td>Water Content (w %)</td>
<td>Dry Density (t/m³)</td>
</tr>
<tr>
<td>BH 6652</td>
<td>39.50</td>
<td>26.1</td>
<td>1.55</td>
</tr>
<tr>
<td>RH6652</td>
<td>16.05</td>
<td>51.9</td>
<td>1.06</td>
</tr>
<tr>
<td>RH6652 - NGD (1985a,b)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fluvialite medium to coarse SAND with some fine gravel (Kp)</td>
<td>h 94</td>
<td>0.67</td>
<td>2.67</td>
</tr>
<tr>
<td></td>
<td>h 18.31-19.31</td>
<td>43</td>
<td>1.13</td>
</tr>
<tr>
<td></td>
<td>h 42</td>
<td>0.392</td>
<td>1.10</td>
</tr>
<tr>
<td>Ignimbritic CLAYEY SILT with some sand (Wg)</td>
<td>h 47.00</td>
<td>93</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>h 48.00</td>
<td>95</td>
<td>0.76</td>
</tr>
</tbody>
</table>

* Horizontal and vertical orientations of the flat circular surfaces of the cylindrical samples with respect to the Tauranga Group sequence.

# Stratigraphic abbreviations:
- (K-H) = Kauroa-Hamilton Ashes
- (Kp) = Karapiro Formation
- (Wg) = Whangamarino Formation
<table>
<thead>
<tr>
<th>Material Type</th>
<th>Range of $C_C$</th>
<th>Average $C_C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ignimbritic silts (silt and clay fraction $&gt;53%$)</td>
<td>0.76-1.63</td>
<td>1.18 (15)*</td>
</tr>
<tr>
<td>Ignimbritic sands</td>
<td>0.13-1.04</td>
<td>0.50 (7)</td>
</tr>
<tr>
<td>Fluviatile silts</td>
<td>0.10-0.44</td>
<td>0.30 (3)</td>
</tr>
<tr>
<td>Fluviatile sands</td>
<td>0.39-1.59</td>
<td>0.89 (4)</td>
</tr>
</tbody>
</table>

* number of one-dimensional consolidation tests completed for each material type.
sequence are the fluviatile silts ($C_c = 0.10$ to $0.44$) followed by the ignimbritic ($C_c = 0.13$ to $1.04$) and fluviatile ($C_c = 0.39$ to $1.59$) sands (Table 4.5). The large ranges of $C_c$ values observed for the ignimbritic and fluviatile sands are interpreted as being due to variations in silt content, interparticle packing and the percentage of pumice within the sample. Pumiceous sands are considered more prone to granular breakdown during consolidation testing than quartz-feldspar rich sands.

The preconsolidation pressure ($\bar{\sigma}_c$) for each sample is estimated from the $e$-$\log\bar{\sigma}$ plots (Figure 4.4; Appendix 5) using the graphical procedure of Casagrande (1936). For samples from the fully cored borehole 6651, estimated values of $\bar{\sigma}_c$ are plotted with calculated overburden effective stresses in Figure 4.7. As this figure shows materials within the sequence range from normally consolidated to highly overconsolidated.

Significant in terms of determining the cause of overconsolidation within the sequence is the reduction in measured $\sigma_c$ with depth between $6.45m$ and $9.83m$, $43.70m$ and $55.10m$, plus $60.95m$ and $61.85m$. These trends combined with the general slightly overconsolidated nature of the ignimbritic silts from $48.00m$ to $58.00m$ suggest that the majority of the overconsolidation is 'apparent' rather than due to past greater overburden stress. Possible causes of the 'apparent' overconsolidation are:

1) dessication within the upper 10 metres of the sequence where negative pore pressure in the capillary zone increases the effective stress causing the soil to consolidate.

2) partial welding within the ignimbritic sand deposits during emplacement.

3) iron cementation from groundwater (observed in core at $43.70m$ where $\sigma_c = 2300$ kPa).

No significant differences in preconsolidation pressures from the lower ignimbritic silt unit are observed
Figure 4.7: Consolidation, void ratio and natural moisture content data for Borehole 6651.
between samples central to the subsidence area (BH6651, samples at 51.0m to 58.95m) and those from the outer margins of the subsidence trough (BH 6653, samples at 47.00m and 48.00m).

Vertically and horizontally orientated samples (orientation of the upper and lower surfaces of the cylindrical sample used in the oedometer with respect to the Tauranga Group sequence) of ignimbritic materials generally have similar values of $C_C$ (Table 4.3) which suggests isotropic compressibility within the sequence. This isotropy is consistent with the massive nature of the deposits observed during field description and S.E.M. studies (Section 4.5). Values of $\overline{c}_C$ are also similar apart from samples at 19.30m (horz.) and 22.55m (vert.) from BH 6651 where a difference of 1,460 kPa is recorded. Similar values of $\overline{c}_C$ suggest that the effects of 'apparent' overconsolidation are generally equal in both horizontal and vertical directions. Observations of Fe-oxide banding in core (Appendix 2) is interpreted as causing the very high $\overline{c}_C$ value at 22.55m.

Estimations of $c_v$ were not made from the oedometer test data because of the well developed secondary compression observed within the highly compressible ignimbritic silts (discussion of the effects of secondary compression on $c_v$ is presented in Section 4.4.1). For a typical 24 hour load increment primary and secondary compression comprised 60% and 40% respectively of the overall axial strain. Three oedometer tests were completed on ignimbritic silt materials (BH6651, samples at 6.68m, 9.83m, and 10.50m) with 7 day load increments. At the end of each load cycle for all three samples, secondary compression with its characteristic linear e-log t relationship (Figure 4.2) was observed. The alternative procedure for calculating settlement rates uses $a_v$ with measured values of $k$ which is adopted for analysis in Chapter Five. The rapid completion of the primary compression phase after loading (Figure 4.3) for the ignimbritic silts is interpreted as being due to relatively high material permeability (c. $10^{-7}$ m/s, Section 4.6).

To date the highly compressible ignimbritic silts
have not been identified from investigations in the Huntly area by S.C.M. (pers. comm. D. Depledge, Senior Mining Engineer, 1985), at Ohinewai by R.W.L. Mining Consultants (R.W.L., 1984) or in the Waikato area by M.W.D. (pers. comm. R.L. Williams, M.W.D. Senior Civil Engineer, 1985). However, investigators of the Taranaki ash deposits (Gradwell and Birrell, 1954; White, 1982; Fullarton, 1978) describe the volcanic clayey silts to silty sands as highly compressible with typical void ratios of 3 to 6 and $C_c$ values ranging from 1 to 2.

4.5 Fabric Study

4.5.1 Introduction

Fabric studies are undertaken to investigate the ignimbritic silt materials in terms of their high void ratios (Section 4.3.2.1) and response to consolidation testing. The two samples investigated (BH6651, 9.84m and 55.28m) are from the main upper and lower highly compressible ignimbritic silt units described within the Tauranga Group sequence.

4.5.2 Method and Results

Soil fabric observations were made using the D.S.I.R. Scanning Electron Microscope (S.E.M.) operated in the Plant and Microbiological Sciences Department, University of Canterbury.

Sample preparation involved oven drying (oven temperatures $105^\circ - 110^\circ$C), mounting on S.E.M. stubs and application of a thin coating (200Å - 500 Å) of gold-palladium. Samples were stored in a dessicator before use in the S.E.M.

The S.E.M. produces an enlarged, three-dimensional view of the sample surface. Selected S.E.M. micrographs from both samples before and after consolidation testing are shown on Figures 4.8 to 4.12. The micrographs were taken in stereoscopic pairs for the fabric study.
Figure 4.8: S.E.M. micrograph of an ignimbritic fine to medium SANDY SILT with some clay (BH 6651, 9.84m - upper ignimbritic silt unit) before consolidation testing. Note the 'open' fabric and the platy, nodular and tabular forms of halloysite. Void ratio = 2.33.

Figure 4.9: S.E.M. micrograph of the ignimbritic SANDY SILT shown in Figure 4.8, but after consolidation testing. The micrograph is at the same scale as Figure 4.8 and exhibits a more compact arrangement and general fragmentation of the matrix. Note the fractured pumice sand grain (p) and the glass shard(s). Void ratio = 1.25.
Figure 4.10: S.E.M. micrograph of an ignimbritic fine to medium SANDY SILT (BH6651, 55.28m - lower ignimbritic silt unit), before consolidation testing. The micrograph shows a fine sand sized quartz grain (identified by conchoidal fracture) supported by a 'flaky' matrix of irregular halloysite sheets. Void ratio = 2.66.

Figure 4.11: S.E.M. micrograph of the ignimbritic SANDY SILT shown in Figure 4.10, but after consolidation testing. As observed in Figure 4.9, a more compact arrangement and general breakdown of the matrix has occurred. Void ratio = 1.62.
Figure 4.12: S.E.M. micrograph of an ignimbritic SANDY SILT (BH6651, 9.84m) at high magnification showing loosely packed aggregates of allophane (a - tentative identification) on curved halloysite sheets (h).
4.5.3 Interpretation and discussion

'Bulky' material structure with random 'loose' packing of particles can be seen in all micrographs of samples prior to consolidation testing. Void space is typically distributed through the materials as irregular 'cavities' and rarely as oval 'ducts' within the solids.

Solids identified within the sand fraction (60 to 200 μm) are quartz (Figure 4.10), pumice and glass shards (Figure 4.9) which are generally matrix supported. The matrix, of predominantly silt size (2 to 60 μm) fragments exhibit flat and curved platy, nodular and tabular forms. These morphologies are consistent with a halloysite-kaolin mineralogy (Dixon, 1977; Lowe and Nelson, 1983) as identified by X.R.D. analyses (Section 4.3.2.4). Very fine (c. 1 μm in diameter) loosely packed aggregates characteristic of allophane (Lowe, 1981) are tentatively identified from the upper ignimbritic silt unit (Figure 4.12).

No preferred particle orientation is observed within the ignimbritic silts which is consistent with the 'massive' field description of the materials, and similar values of permeability and compressibility in both horizontal and vertical directions (Sections 4.4.3 and 4.6.3).

Loading during one-dimensional consolidation testing of the ignimbritic silts typically extended to c. 6 times their preconsolidation pressure. Both consolidated samples exhibit (Figures 4.9 and 4.11) a more compact arrangement and fragmentation of the silt matrix. Fractures within sand size pumice and quartz grains are also observed. The overall particle fragmentation is illustrated by Figure 4.13 where grain size distribution curves, before and after consolidation testing are plotted.

The 'open' texture illustrated in the ignimbritic silts has also been described by Carr (1981) in his S.E.M. study of the Matahina Ignimbrite, Bay of Plenty.

4.6 Permeability Testing

4.6.1 Introduction
Figure 4.13: Particle grading curves for an ignimbritic fine to medium SANDY SILT (BH6651, 6.45m) showing material fragmentation with consolidation testing.
Permeability tests were undertaken for all materials within the Tauranga Group sequence from BH6651 (apart from gravels and gravelly sands) to provide detailed data for the settlement-time analysis of Chapter Five. Samples were orientated from core in both vertical and horizontal directions to investigate permeability anisotropy.

4.6.2 Test Procedure and Results

A falling head apparatus was used for all tests in preference to the constant head equipment because of low expected material permeabilities as indicated by field tests (Section 3.5.2).

Test procedure follows that described by Vickers (1978) and Scott (1980). The undisturbed samples are initially trimmed by the permeability mould (to ensure no excess leakage at sample circumference) which is then filled with water and attached to a stand-pipe. A hydraulic head is then applied to the sample and its drop with time recorded. Test apparatus and permeability calculations are presented in Figure 4.14.

Samples were left in apparatus for up to 12 hours under flow conditions to ensure complete saturation of materials before permeabilities were estimated. Permeability determinations were made with varying changes of head, the average of these values being adopted as the result.

Results of all permeability tests are presented in Table 4.6. Due to sample trimming difficulties, the permeabilities of fluviatile silts could not be measured using the falling head apparatus. Permeabilities for these materials are calculated from one-dimensional consolidation test data and are presented in Table 4.7. Ranges of measured permeabilities are compared with S.C.M. Ohinemutu data (Heu, 1983) in Table 4.8.

4.6.3 Interpretation and Discussion

The test results show that k values of the Tauranga
Figure 4.14: Apparatus and calculations for falling head permeability testing.
<table>
<thead>
<tr>
<th>Sample Depth (m)</th>
<th>Material Description and Formation¹</th>
<th>Permeability² (k_v, k_h in m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.22</td>
<td>Ignimbritic fine to medium SANDY SILT (K-H)</td>
<td>k_v = 4.76 x 10^-7</td>
</tr>
<tr>
<td>9.74</td>
<td>Ignimbritic SILTY fine to medium SAND (K-H)</td>
<td>k_v = 4.55 x 10^-7</td>
</tr>
<tr>
<td>10.00</td>
<td>Ignimbritic SILTY fine to medium SAND (K-H)</td>
<td>k_v = 5.57 x 10^-7</td>
</tr>
<tr>
<td>10.15</td>
<td>Ignimbritic SILTY fine to medium SAND (K-H)</td>
<td>k_h = 6.34 x 10^-7</td>
</tr>
<tr>
<td>10.40</td>
<td>Ignimbritic SILTY fine to medium SAND (K-H)</td>
<td>k_v = 5.22 x 10^-7</td>
</tr>
<tr>
<td>10.25</td>
<td>Ignimbritic SILTY fine to medium SAND (K-H)</td>
<td>k_h = 4.49 x 10^-7</td>
</tr>
<tr>
<td>13.00</td>
<td>Ignimbritic SILTY fine to medium SAND (Kp)</td>
<td>k_v = 1.39 x 10^-6</td>
</tr>
<tr>
<td>13.43</td>
<td>Ignimbritic SILTY fine to medium SAND (Kp)</td>
<td>k_h = 1.40 x 10^-6</td>
</tr>
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<td>k_v = 1.44 x 10^-5</td>
</tr>
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<td>22.60</td>
<td>Ignimbritic fine to medium SAND (Kp)</td>
<td>k_h = 1.98 x 10^-5</td>
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<td>22.90</td>
<td>Ignimbritic fine to medium SAND (Kp)</td>
<td>k_v = 1.73 x 10^-5</td>
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<td>39.75</td>
<td>Ignimbritic SILTY fine to medium SAND (Wg)</td>
<td>k_h = 5.21 x 10^-7</td>
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<td>43.75</td>
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<td>k_v = 2.55 x 10^-6</td>
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<td>Ignimbritic fine to medium SANDY SILT (Wg)</td>
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</tr>
<tr>
<td>55.50</td>
<td>Ignimbritic fine to medium SANDY SILT (Wg)</td>
<td>k_v = 1.17 x 10^-7</td>
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<td>59.00</td>
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<td>k_v = 1.60 x 10^-7</td>
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<td>59.20</td>
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<td>k_h = 1.89 x 10^-6</td>
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<tr>
<td>60.95</td>
<td>Fluviatile SILTY fine to medium SAND (Wg)</td>
<td>k_v = 7.48 x 10^-7</td>
</tr>
<tr>
<td>66.40</td>
<td>SILTY CLAY (weathered Waikato Coal Measures Mudstone)</td>
<td>k_v = 1.23 x 10^-10</td>
</tr>
</tbody>
</table>

¹ Formation abbreviations (K-H) = Kauroa-Hamilton Ashes (Kp) = Karapiro Formation (Wg) = Whangamarino Formation

² k_v = vertical permeability; k_h = horizontal permeability
<table>
<thead>
<tr>
<th>Material Description</th>
<th>Sample Depth (m)</th>
<th>Permeability $k_v$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fluviatile CLAYEY SILT with some sand (Wg)*</td>
<td>34.90</td>
<td>2.70 x 10^{-11}</td>
</tr>
<tr>
<td>Fluviatile SANDY SILT with some clay (Wg)</td>
<td>61.85</td>
<td>1.55 x 10^{-10}</td>
</tr>
</tbody>
</table>

* (Wg) = Whangamarino Formation
<table>
<thead>
<tr>
<th>Texture</th>
<th>Laboratory permeabilities(^1) ((k_h\ \text{and} \ k_v\ \text{in m/s}))</th>
<th>Average values of S.C.M. (Ohinewai) data(^2) ((\text{m/s}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAND</td>
<td>((\text{ignimbritic})) (1.31 \times 10^{-5}\ \text{to} \ 1.98 \times 10^{-5}) ((4))</td>
<td>(1.72 \times 10^{-5}) ((1))</td>
</tr>
<tr>
<td></td>
<td>((\text{fluviatile})) (1.89 \times 10^{-6}\ \text{to} \ 7.48 \times 10^{-7}) ((3))</td>
<td></td>
</tr>
<tr>
<td>SILTY SAND</td>
<td>((\text{ignimbritic})) (2.55 \times 10^{-6}\ \text{to} \ 4.55 \times 10^{-7}) ((9))</td>
<td>(3.05 \times 10^{-7}) ((4))</td>
</tr>
<tr>
<td></td>
<td>((\text{fluviatile})) (1.89 \times 10^{-6}\ \text{to} \ 7.48 \times 10^{-7}) ((3))</td>
<td></td>
</tr>
<tr>
<td>SANDY SILT</td>
<td>((\text{ignimbritic})) (1.06 \times 10^{-7}\ \text{to} \ 4.76 \times 10^{-7}) ((2))</td>
<td>(1.80 \times 10^{-7}) ((2))</td>
</tr>
<tr>
<td></td>
<td>((\text{fluviatile})) (1.55 \times 10^{-10}) ((1))</td>
<td></td>
</tr>
<tr>
<td>CLAYEY SILT</td>
<td>((\text{fluviatile})) (2.70 \times 10^{-11}) ((1))</td>
<td>(1.0 \times 10^{-8}) ((1))</td>
</tr>
</tbody>
</table>

1. Laboratory data from both falling head permeability one-dimensional consolidation testing.

2. Average measured \(k\) values from Heu (1983).

3. Number of permeability determinations.
Group sequence range from medium to very low ($10^{-5}$ to $10^{-11}$ m/s) values of permeability. This range can probably be extended to $10^{-2}$ to $10^{-3}$ m/s for the highly permeable gravel lenses within the succession (Section 3.8.4).

Consistent permeabilities ($c.10^{-7}$ m/s) are evaluated for the upper and lower ignimbritic silt rich units (BH 6651, at 6.22m to 10.25m and 55.40m to 55.50m). These values are contrasted by the relatively impermeable fluviatile silts where $k$ ranges from $10^{-10}$ to $10^{-11}$ m/s. The ignimbritic and fluviatile sands have similar $k$ values, and are generally more permeable than the ignimbritic silts.

Vertical and horizontal permeabilities, apart from the samples of BH6651, 39.75m and 43.75m (Table 4.6) are approximately equal which is consistent with the massive nature of the materials observed from core (Appendix 2) and S.E.M. investigations (Section 4.5.3). The order of magnitude difference between $k_h$ at 39.75 and $k_v$ at 43.75 is interpreted as being due to the effects of subhorizontal ‘flattened’ pumice clasts (see BH6651 log, Appendix 2) which cause the higher horizontal permeability.

Variations in $k$ values within the ignimbritic and fluviatile sands, as indicated by field material descriptions (Appendix 2) are due to variations in silt content, material compaction and iron cementation.

The S.C.M. permeability data (Table 4.8) is in general agreement with $k$ values of this investigation. The large differences between the laboratory and S.C.M. determinations for the (fluviatile) sandy silts and clayey silts is considered to be due to varying degrees of material compaction.

4.7 Synthesis: Implications of Laboratory Investigation to the Engineering Geological Model

In terms of the tentative engineering geological model shown in Figure 2.14, laboratory investigations have identified a highly compressible silt aquitard within the lower part of the Tauranga Group succession. The highly compressible nature of the ignimbritic silt is primarily due to its bulky, loose packing of particles as described by the
S.E.M. fabric study and inferred by high void ratios and natural water contents. As shown by the large scale cross-sections (Figures 3.17 and 3.18, in map pocket), the ignimbritic sandy and clayey silt ranges in thickness from c.4 to 10 metres and is extensive over the hostel site. No significant differences in consolidation parameters or material properties are observed between samples from the lower ignimbritic silt unit centrally to, and on the outer margins of the subsidence area.

The highly compressible ignimbritic silts are contrasted by the incompressible fluvialite silts which are distinctive in terms of their low void ratios and water contents.

The ignimbritic and fluvialite sands within the lower part of the Tauranga Group sequence exhibit moderate to low compressibility values.

Directional isotropy in terms of compressibility and permeability is illustrated for most materials within the Tauranga Group succession.
CHAPTER V

ANALYSIS OF ENGINEERING GEOLOGICAL MODEL

5.1 Introduction

The purpose of this chapter is to analyse the data collected during the field and laboratory investigations with respect to the tentative engineering geological model shown in Figure 2.14. The analysis aims to determine whether:

1) void migration from roof falls is able to extend close enough to the Tauranga Group aquifer to cause drainage.
2) dewatering consolidation of the lower Tauranga Group materials contributed to the subsidence observed at the hostel site.

5.2 Roof Failure

5.2.1 General Considerations

Redistribution of forces in the roof after mining, as modelled for an ideal situation is shown in Figure 5.1. The two basic modes of failure recognised are shear failure resulting from shear forces immediately adjacent to the pillars, and flexural failure due to bending forces central to the roof-span area (Morgan, 1973).

Morgan, in his study of roof problems sites conditions conducive to shear failure as; the presence of discontinuities such as joints and faults orientated to permit instability; high vertical stress either from a large overburden thickness or transferred from adjacent mine workings; high horizontal stress; plus pillars and floor which are 'stiff' in comparison to the roof.

Morgan summarises conditions conducive to flexural failure as; low ratios of horizontal to vertical stress; thinly bedded layers or layers that are separated along
Figure 5.1: Sectional view of an idealised distribution of roof forces between two pillars (from Roberts, 1977). Possible failure modes are indicated by shear forces immediately adjacent to pillars (shear failure) and bending forces central to the roof-span area (flexural failure).
bedding planes; wide roof spans; jointing in the roof; and roof materials of low relative 'stiffness'.

Once a rock fall has initiated, void migration will propagate upwards until equilibrium is achieved compatible with the existing gravitational and regional stresses, plus the physical properties of the overburden. Void migration is inhibited by; 'competent' stratum in the overburden bridging the void; the formation of a 'stable' pressure arch restricting the development of the de-stressed strata zone (Figure 5.1); and bulking of the collapsed material.

The effectiveness of void bridging within the overburden is dependent on the flexural strength of the spanning material. The height to which the pressure arch can extend is a function of room geometry and relative pillar stiffness (Alder and Sur, 1968). Ackenheil and Dougherty (1970) describe pressure arches attaining their maximum height at approximately twice the distance between the supporting pillars. Conditions of void bridging or stable arch development should always be considered temporary in terms of long term stability (St. George, 1982).

Determining the volumetric increase or bulking (Section 3.7) of the collapsed roof materials for a particular caving geometry, the height to which void migration can extend can be estimated. A graph, developed by Dunrud (1984), showing the relationship between maximum height of void migration and bulking for different caving geometries is shown in Figure 5.2.

Bulking factors for coal measure materials typically range from 0.3 to 0.5 (Piggot and Eynon, 1978) although values as low as 0.2 have been measured for soft claystones and shales (Dunrud and Osterwald, 1980). The most common caving geometry for room and pillar workings approximates an ellipsoid (consistent with 'pressure arch' theory shown in Figure 5.1), although prism, wedge and cone geometries have also been recorded (Dunrud, 1984).

Using the typical bulking factor of 0.3 to 0.5, Figure 5.2 shows that the maximum height of void migration
\[ B = \frac{V_c - V_o}{V_o}, \quad V_o = \text{Volume of rock before caving}\]
\[ V_c = \text{Volume of rock after caving}\]

**SAMPLE CALCULATION:**

**ELLIPSE:**

\[
B = \frac{(4/3)(1/2) \pi H b^2 + \pi b^2 t - (4/3)(1/2) \pi H b^2}{(4/3)(1/2) \pi H b^2}
\]

\[
B = \frac{\pi b^2 t}{2/3 \pi H b^2} = \frac{t}{2/3H}, \quad H = 3/2 t/B = 1.5t/B
\]

---

**Figure 5.2:** Maximum height of void migration for various caving geometries (from Dunrud, 1984, page 160, Figure 6). This graph shows the relationship between height of caving \( (H) \), in terms of thickness of coal mined \( (t) \), and the bulking factor \( (B) \) for various caving geometries. Sample calculation for one-half the volume of an ellipsoid of revolution is shown.
ranges from c.2t to 10t (where t is the thickness of coal mined). The estimated upper bound of 10t is consistent with field observations above room and pillar mines in New Zealand (Kelsey, 1980) and the United Kingdom (Taylor, 1975; St. George, 1982).

5.2.2 Roof Failures in the South Headings

On the basis of field evidence (Sections 2.3.3 and 3.7) roof instability is interpreted as being primarily flexural type failure with minor shear failure occurring as caving develops. Flexural failure is indicated by the location of virtually all roof falls at mine roadway intersections (Figures 2.3 and 3.16) where spanning distances are greatest. The exposure of bedding planes in the caving chimneys (Figure 3.16) is also consistent with this failure mode. The secondary shear failure mode was observed underground with blocks intermittently falling off the chimney walls along joints and shears.

The low regional values of in-situ horizontal stress as measured in the Huntly area (Mills, 1985) is listed by Morgan (1973) as one of the conditions conducive to flexural roof failure.

The geometry of south headings roof fall chimneys is approximately ellipsoidal as shown by Figures 3.16 and 2.10. This geometry, combined with the measured bulking factor of 0.3 (Section 3.7) when plotted in Figure 5.2 indicates that the maximum height of caving will be c.5t. This upper limit for panel 1 is estimated at 30m since the mine drive heights (t) range between 3.5m and 6.0m (Section 1.4.2).

In terms of the Tertiary overburden above panel 1 (Figure 3.19), the calculations indicate that the maximum caving height ranges between 5 and 25 metres below the Te Kuiti-Tauranga Group contact. The extension of caving to near the top and not through the Te Kuiti Group sequence is consistent with field evidence as there has been no record of Tauranga Group materials either in mine workings or as suspended solids in mine water discharge (Section 2.3.2).
5.3 Dewatering Consolidation

5.3.1 Conceptual Aspects

The effective (intergranular) stress ($\bar{\sigma}$) within saturated soil materials is equal to the total overburden stress ($\sigma$) minus the pore fluid pressure or neutral stress ($\mu$).

$$\bar{\sigma} = \sigma - \mu$$

(Terzaghi, 1925)

Terzaghi's equation shows that increased $\bar{\sigma}$ can result from load application ($\sigma$) or reduction in pore water pressure ($\mu$).

In terms of the engineering geological model (Figure 2.14), drainage at the base of the Tauranga Group sequence will cause dissipation of pore pressure ($\mu$) which results in increased effective stress ($\bar{\sigma}$). The effects of drainage are schematically shown in Figure 5.3 for a confined aquifer overlain by an aquitard, a simplistic approximation of the lower Tauranga Group succession. The materials consolidate as a result of the increased effective stress, with the rate and magnitude of consolidation dependent on the soil's permeability and compressibility. These two factors are discussed with other consolidation parameters in Section 4.4.1.

Land subsidence due to groundwater withdrawal is a well documented phenomenon and is described world-wide by Poland (1972). Subsidence has also been related to the withdrawal of oil and gas (Poland and Davis, 1969), and geothermal fluid (Narasimhar and Goyal, 1984).

5.3.2 Back-analysis of Ultimate Settlement and Piezometric Head Drop

5.3.2.1 Analysis Procedure

The analytical approach taken assumes that the lower part of the Tauranga Group was normally consolidated prior to the hostel subsidence. The overconsolidation of these
Figure 5.3: Ground water withdrawal from a confined aquifer overlain by an aquitard. Consolidation results from increase in effective stress ($\bar{\sigma}$) (adapted from De Simone and Viggiani, 1978).

$\Delta h_p$ = pressure head drop with drainage
materials (as measured by the one-dimensional consolidation tests - Section 4.4) is assumed to be attributable to increases in effective stress associated with dewatering. The analysis is restricted to two layers, the lower highly compressible ignimbritic silt and underlying less compressible silty sand (Figure A6.1) in Appendix Six.

The pre-subsidence ground level is calculated from estimates of the original thickness of the two layers. The piezometric head drop \( \Delta h_p \) is evaluated from the increase in effective stress (decrease in \( \mu \)) inferred by the preconsolidation pressures.

The boundary conditions, a full list of assumptions, and calculations for the analysis are presented in Appendix Six.

5.3.2.2 Results and Interpretation

The relationship between assumed initial effective stress and measured preconsolidation pressure suggests an ultimate settlement of 894mm for the two consolidating layers. Measured settlement at E53, a precise levelling mark 20 metres away from borehole 6651 is 829mm. Thus, the observed and calculated magnitudes of ultimate settlement are in general agreement.

Estimated piezometric head drops at the mid-heights of each layer are 16.61m and 32.65m for the ignimbritic silt and underlying sand respectively. The maximum possible mid height piezometric head drops are c.41.0m for the ignimbritic silt and c.48.0m for the underlying sand. The lower calculated values of 16.61m and 32.65m indicate that only partial drainage of the consolidating layers has occurred. The difference in piezometric head drop between the two layers suggests that recharge and re-establishment of higher pore pressures took place in the sand before complete drainage of the ignimbritic silt.

These results and interpretations are tentative since it cannot be shown that the lower part of the sequence was originally normally consolidated, nor can the effects of 'apparent' overconsolidation (Section 4.4.3) be established with any certainty.
5.3.3 Back Analysis of Settlement Rates

5.3.3.1 Analysis Procedure

The multilayered nature of the Tauranga Group sequence does not allow evaluation of settlement rates using conventional Terzaghi one-dimensional consolidation theory. The approach adopted for the calculations uses an explicit one-dimensional finite difference analysis which calculates settlement for the layered sequence undergoing drainage from its bottom boundary.

The effects of aquifer recharge are not incorporated into the procedure, thus making the assessment of settlement rates conservative. However, since the lower aquifer in the Tauranga Group sequence (Section 3.8.4) at the hostel site is confined, the effects of recharge within this critical zone is considered negligible.

The theoretical basis for the finite difference analysis was developed by Drs R.O. Davis and B.W. Hunt of the Civil Engineering Department, University of Canterbury. This study used the theoretical basis to develop DECONS1 (dewatering consolidation), a Basic computer program for the finite difference solution. The theoretical basis, and a listing of DECONS1 is presented in Appendix Seven.

Values for the coefficients of compressibility \(k\) and permeability \(a_v\) are assumed to be constant throughout the analysis. Groundwater flow is assumed to be taking place under saturated conditions.

The input data and boundary conditions for DECONS1 are presented in Figure 5.4. The boundary conditions are interpreted from the summary log of Borehole 6651 (Appendix Two) and the \(a_v\) and \(k\) parameters used for input data are representative values from laboratory testing (Sections 4.4 and 4.6). The \(a_v\) values for layers 1, 3 and 4 are adjusted for the purposes of back analysis to represent initial normally consolidated conditions. The method of adjustment is shown in Appendix Five (BH6651, samples from 55.28m and 58.89m).

Gravels within the sequence, due to their high permeabilities are omitted because of numerical stability
Figure 5.4: Input parameters and calculated changes of pressure head and vertical compression using DECONS I.
requirements (Appendix Seven, Section 3.2). This omission is not considered significant in terms of the overall analysis since these materials are relatively incompressible (Lambe and Whitman, 1979), of limited thickness, and are laterally discontinuous.

DECONSI evaluates settlement for any specified time period (target time) after drainage from the bottom boundary has commenced. By running the program for different target times, a settlement-time curve can be constructed.

The input data considered least reliable for DECONSI are the permeability values of layers 2 and 6 (fluvialite silts). The permeabilities of these layers were estimated indirectly from the one-dimensional consolidation test data (Table 4.7). During the analysis, a number of settlement-time curves were constructed, adjusting k in layers 2 and 6 to investigate whether a close fit of the field curve could be made. The input data shown in Figure 5.4 lists the adjusted k values for the 'closest-fit' curve.

5.3.3.1 Results and Interpretation

The 'closest-fit' theoretical and observed field settlement-time curves are shown in Figure 5.5. As shown by Figure 5.4 the curve for E53 can be matched using realistic permeability values for layers 2 and 6.

The theoretical maximum and minimum rates of settlement are 271mm/day and 0.2mm/day respectively. Measured settlement rates on the basis of limited survey data for E53 range between 10mm/day and 0.6mm/day. The deviation of the E53 field curve from the theoretical curve during April 1983, is attributed to the effects of Phase II rapid subsidence (Section 2.2) where drainage possibly occurred at another location, from the base of the sequence.

As part of the estimation of settlement, DECONSI calculates changes in void ratio, total water head, superficial water velocity and water table position. Output data from the program lists all these parameters for individual zones within layers present below the water table. Representative output data for the 'closest-fit' settlement time curve are presented in Appendix Eight.
Figure 5.5: Comparison of observed and calculated rates of settlement for the hostel subsidence.
One-dimensional drainage of the sequence can be expressed in terms of reduction of pressure head \( h_p \) with time. The pressure head distribution is plotted for the sequence at the target times of 30 minutes, 24 hours and 500 days in Figure 5.4. Vertical compression for each layer at the target time of 500 days is also plotted on Figure 5.4.

Features to note from the pressure head and compression data presented in Figure 5.3 are:

1) the large changes \( \Delta h_p = 52.5 \text{m}, \Delta \mu = 515 \text{kPa} \) of pressure head at the base of the sequence when drainage takes place.

2) the significant effect of the low permeability of layer 2 controlling pressure head dissipation.

3) the development of negative pressure head (up to c. \(-3.5 \text{m}\)) in layer 1.

4) that the greatest compression occurs in the ignimbritic silt (layer 4) and the lower silty sand (layer 1).

One of the major implications of this analysis is that the fine grained materials of very low permeability limit the magnitude of pressure head dissipation within the sequence. For example if layer 2 was absent from the succession, much larger settlements would result.

5.4 Elastic Response of Mine Pillars

A minimum of 50mm of subsidence has occurred over all workings of the south headings (Figure 2.1). The subsidence takes place during and immediately after mining (Figure 2.3) and in interpreted by Depleedge (1983a) as representing the elastic response of coal to pillar. This 'immediate' subsidence has taken place prior to the rapid subsidence phases monitored above panels 1 and 3, as shown in Figure 2.1. The magnitudes of 'immediate' subsidence in these areas is measured as c.85mm for E53 and
90mm for E91 (Figure 2.1).

To account for subsidence related to the elastic response of coal to pillar cutting, the datum for the theoretical settlement-time curve is adjusted to 80mm of observed settlement (Figure 5.5).
CHAPTER VI

SUMMARY AND CONCLUSIONS

6.1 Causes and Mechanisms of the Hostel Subsidence

The proposed engineering geological model, supported by field and laboratory investigations, and considered most likely in explaining the hostel subsidence is presented in Figure 6.1. The causes of ground movement defined by this model can be categorised into engineering geological, geotechnical and mining related factors.

6.1.1 Engineering Geological Factors

Engineering geological factors considered to have contributed to the hostel subsidence are:

1) the relief on the Te Kuiti-Tauranga Group contact and the structural position of the mined Kupakupa seam. Relief on the Te Kuiti-Tauranga Group contact (as defined by drilling), is dominated by a buried hill (-10m R.L.) to the east, gently sloping (c. 8°) to a buried valley in the west (-45m R.L.). On this surface an east-west striking minor buried valley has also been delineated; and

2) panel 1, below the hostel, represents workings at the highest structural position in the southern headings. Structure contours for the mine floor in panel 1 range from -75m to -85m metres below sea level.

These two engineering geological factors provide the geometric constraints to the thickness of Tertiary overburden above mine workings. Over panel 1, coal measures material cover ranges from 35 to 50 metres.

6.1.2 Geotechnical factors

Geotechnical factors considered to have
Figure 6.1: Engineering Geological Site Model
contributed to the hostel subsidence are:

1) the moderate to highly compressible materials identified at the base of the Tauranga Group sequence; and

2) the presence of an aquifer, associated with these materials, under high peizometric head.

Near the base of the highly pumiceous, typically well graded, interlayered Tauranga Group sequence, two compressible units are present. The compressible materials consist of an ignimbritic silt unit, c.4 to 10 metres thick which is transitional to an underlying ignimbritic silty sand c.1 to 2 metres thick. Both units are extensive over the hostel area as defined by drilling (Figures 3.17 and 3.18). The ignimbritic silt is highly compressible \( (C_c = 1.14 \text{ to } 1.63) \) and is characterised by high void ratios \( (e = 2.07 \text{ to } 2.77) \), high water contents \( (w = 70 \text{ to } 134\%) \) and low dry densities \( (p_d = 0.71 \text{ to } 0.88 \text{ t/m}^3) \). The underlying ignimbritic silty sand is moderately compressible \( (C_c = 0.76 \text{ to } 0.97) \). Material permeabilities for the ignimbritic materials are typically low \( (k_v = 10^{-7} \text{ m/s}) \), with both units comprising an aquitard above the lower aquifer.

The lower aquifer is interpreted as being confined and has a restricted distribution over the hostel site area. The piezometric head at the base of the aquifer, prior to any mining activity is considered to be c.53m.

6.1.3 Mining Related Factors

Factors related to mining activity considered to have contributed to the hostel subsidence are:

1) mine roof instability causing caving to near the top of the Te Kuiti Group sequence; and

2) possible elastic response of coal pillars during mining.
Roof failure and resulting caving chimneys have been observed during the underground inspections by State Coal Mines in panel 1, below the hostel. Roof collapse in terms of void migration through the Te Kuiti Group sequence is summarised as part of the model analysis in Section 6.1.4.

Subsidence resulting from possible elastic response of coal pillars to mining is interpreted on the basis of surface monitoring data, where at least 50mm of settlement is measured above all mine workings of the south headings. This subsidence mechanism is speculative and further investigations are required to determine its nature and magnitude (Section 6.4).

6.1.4 Subsidence Mechanisms and Model Analysis

The primary mechanism of the hostel subsidence is identified as consolidation of materials within the lower part of the Tauranga Group sequence in response to mining induced dewatering. Considered to be of secondary importance is the possible initial subsidence response of the coal pillars to mining.

The bulking factor (volumetric expansion) of the coal measure materials is calculated at 0.3 from the underground mapping of a roof fall area. Using this factor and observations of caving chimney geometry, the estimated maximum height of caving is 5t (where t = the mined seam thickness) which is in accordance with comparative overseas studies. In terms of the Tertiary overburden above panel 1, the maximum caving height extends to between 5 and 25 metres below the Te Kuiti-Tauranga Group contact. It is considered likely that drainage (and resulting depressurisation) of the lower confined aquifer within the Tauranga Group took place along geological discontinuities or caving induced shear fractures.

Two one-dimensional back analysis procedures were adopted to compare calculated and observed magnitudes and rates of settlement. Both procedures use boundary conditions and geotechnical data from borehole 6651, in the area affected by maximum subsidence.
For the determination of ultimate settlement, the two compressible ignimbritic units are assumed to be normally consolidated prior to subsidence. The overconsolidation of these materials (as measured by oedometer testing) is assumed to be attributable to increases in effective stress associated with dewatering. Calculated ultimate settlement for the two consolidating layers is 894mm which compares well with the measured settlement of 829mm at E53, a levelling mark 20 metres away from borehole 6651. The calculations of ultimate settlement are considered tentative, since it cannot be shown that the lower part of the sequence was originally normally consolidated, nor can the effects of 'apparent' overconsolidation be established with any certainty.

The analysis of settlement rates uses a one-dimensional explicit finite difference numerical procedure, developed as part of this study. The analysis calculates settlement for the layered sequence undergoing drainage from its bottom boundary. The consolidation parameters for the compressible ignimbritic materials are adjusted for the purposes of back analysis to represent initial normally consolidated conditions. Using measured and estimated permeability values for the layered sequence, a close fit between observed and calculated settlement-time curves is attained. The theoretical maximum and minimum rates of settlement are 271mm/day and 0.2mm/day respectively. Measured settlement rates on the basis of limited survey data at E53 range between 10mm/day and 0.6mm/day.

One of the major implications of the finite difference analysis is that fine grained materials of very low permeability limit the magnitude of pressure head dissipation and resulting consolidation within the sequence. If these materials were absent from the lower part of the Tauranga Group sequence, significantly larger settlements would result.

The effect of aquifer recharge is not incorporated into the finite difference analysis, thus making the assessment of settlement rates conservative. However, since the lower aquifer in the Tauranga Group sequence at the
hostel site is confined, the effect of recharge within the critical zone of drainage is considered negligible.

The proposed model, although considered most probable in explaining the hostel subsidence, still requires further field verification, particularly observations related to mining induced drainage of the lower Tauranga Group materials. Suggested procedures for such field verifications are discussed in Section 6.4.

6.2 Future Subsidence over the South Headings

In terms of the model presented in Figure 6.1, the potential exists for continued subsidence over the south headings of the Huntly East Mine.

A minimum of c.50mm (based on existing Lands and Survey Department surface monitoring) of subsidence can be expected above all future panels of the south headings. This component is interpreted as being due to the elastic response of coal with pillar splitting and is likely to occur during and immediately after mining in a particular area.

Greater magnitudes and rapid phases of subsidence, related to dewatering consolidation, and interpreted as causing the greater part of observed settlements above panels 1 and 3 (maximum observed settlement over these panels to the 24th June, 1985 is 1134mm at E91) is considered possible over existing and future workings of the south headings. Factors important in determining areas likely to be subject to this type of subsidence are:

1) whether the Tertiary overburden thickness is adequate to prevent caving chimneys (as a result of mine roof failure) from draining the lower part of the Tauranga Group sequence; and

2) whether the compressible materials exist within zones likely to be subject to aquifer drainage (and depressurisation).

A recommended zonation scheme over the south headings
in terms of the minimum allowable Tertiary overburden thickness is discussed in Section 6.4.

The ignimbritic materials, because of their mode of origin were likely to have been extensive over the south headings area. However, subsequent erosion may mean a restricted present day distribution. Thus, although these materials are extensive beneath the hostel site, their presence elsewhere over the south headings area cannot be predicted without detailed subsurface information.

Panel 4, adjacent to the workings under the hostel is considered to be an area of potential dewatering consolidation subsidence since:

1) the highly compressible lower ignimbritic silt is described at the base of the Tauranga Group sequence above these workings in BH6652 (48.3m to 52.3m, Appendix 2).

2) the Tertiary overburden thickness, on the basis of extrapolated data (see Figure 3.19) is of a similar thickness to that described beneath the hostel site.

3) roof falls are observed (Figure 3.16) within the panel.

The geometry of the drained aquifer is important in determining the extent of the subsidence trough. It is thus possible that subsidence troughs related to dewatering consolidation can extend significantly beyond the area directly above the mine workings. This feature is observed with the area affected by the hostel subsidence (Figure 2.5 and 2.6) where limit angles (Section 2.2) extend up to 71°.

The timing of subsidence related to dewatering consolidation after the mining of a panel is considered to be unpredictable and related to the rate at which caving extends through the Tertiary overburden. The two rapid phases of the hostel subsidence took place 5 and 8 months after mining in panel 1 was completed.

The likelihood of repeated rapid phases of subsidence over panels 1 and 3, according to the model, will depend on
the extent to which the Lower Tauranga Group aquifers have depressurised with mining induced drainage. Model analysis (Section 5.3) indicates that the highly compressible ignimbritic silts have only been partially drained with the 1983 hostel subsidence. Thus, the potential exists for further rapid subsidence phases depending on the extent to which mining induced drainage extends into the base of the Tauranga Group sequence.

In part due to the difficulties of mine excess, mining subsidence related to either mine pillar or floor failure has not been investigated as part of this study. As a consequence, the preceding discussion has not incorporated these two failure modes. The limited subsurface data available from panel 1 does indicate that it is unlikely either pillar or floor failure have played a significant role in causing the subsidence observed.

6.3 Implications for Future Coal Mining in the Huntly Area

This study has identified highly compressible materials within the Tauranga Group sequence. Where mining activity is likely to dewater these materials, the highly compressible units should be defined, and where present incorporated into settlement predictions during the mine design stages.

Planned mining developments likely to involve dewatering of the Tauranga Group sequence include the longwall mines of Huntly East and West and the proposed Weavers extension and Ohinewai opencast mines.

6.4 Future Work

6.4.1 Further Verification of the Engineering Geological Model

Future investigations considered to be of greatest importance are those associated with the further verification and confirmation of the proposed engineering geological model. The suggested methods of verification
1) piezometric water level monitoring of the piezometers installed (as part of this study) into the lowest Tauranga Group aquifer immediately above panel 4.

2) underground inspection of mine workings under the areas of maximum subsidence in panels 1 or 3.

During site investigations, piezometers were installed into the middle aquifer above panel 4 (the lower aquifer is absent). Monitoring of piezometric water levels with possible future settlement will allow a direct observation of aquifer depressurisation. Ideally instrumentation should be installed for these piezometers to provide a continuous record of water level fluctuations.

Until further investigations are completed of workings under areas of maximum subsidence, the assessment of any pillar or floor instability which may have contributed to observed settlement cannot be made with confidence.

6.6.2 Zoning of Potential Dewatering Consolidation Subsidence Areas

A suggested empirical approach for delineating areas likely to be affected by dewatering consolidation above room and pillar workings is to combine an upper limit of chimney caving with the mine Tertiary overburden thickness. Such a zonation scheme would define areas where caving, resulting from roof falls is likely to induce drainage of the lower Tauranga Group sequence.

The estimation of an upper limit of caving requires further investigation of coal measures bulking factors and observations of caving chimney geometries as discussed in Sections 3.7 and 5.2. As a general rule, based on conservative estimates of coal measures bulking factors, the upper limit of caving is 10 times the seam thickness (St. George, 1982; Babcock and Hooker, 1977).

An isopach map, similar to the one constructed as
part of this study and shown in Figure 3.19, of Tertiary overburden can be used to define areas less than the upper limit of caving. Within these areas the potential for dewatering consolidation subsidence exists.

6.4.3 Other Investigations

The common occurrence of roof falls observed in panels 1 and 4 requires a further reassessment of room and pillar design. A modified room and pillar design reducing roof spanning distances at intersections was adopted after the hostel subsidence and has been used for panels 3 and 4. This modified design has failed to control roof instability.

The lack of geological data from the room and pillar workings has hindered the identification of geologically related factors to the hostel subsidence. It is recommended that geological mapping combined with room and pillar design for roof stability.

An underground investigation programme determining the magnitude of immediate subsidence related to the elastic response of coal to pillarings is also recommended. Such a programme would involve the monitoring of both pillars (with borehole extensometer measurements) and roadways (with convergence measurements) during and after mining.
REFERENCES


TERZAGHI, K., (1925) Erdbaumechanik, Vienna, F. Deuticke.


APPENDIX 1: GLOSSARY OF TERMS

Aquifer: an aquifer is a formation, group of formations, or part of a formation that contains sufficient saturated permeable material to yield significant quantities of water to bores and springs.

Confining Bed: a confining bed is a body of impermeable material stratigraphically adjacent to one or more aquifers. In nature its permeability may vary from zero (aquiclude) to some value distinctly lower than that of the aquifer (aquitard).

Crown Holes: holes at the ground surface that result from void migration associated with mine roof failure. Those features commonly approximate a circular outline in plan and in section resemble an upright truncated cone.

Hydrostatic Response Time: hydrostatic time lag between increase in pore pressure of monitored material to increase in piezometric head as measured in a piezometer.

Settlement: measured or calculated component of downward ground movement.

Subsidence: downward ground movement.
APPENDIX 2: Detailed Lithology and Summary Logs
## LOG OF DRILL HOLE

### DESCRIPTION OF CORE

<table>
<thead>
<tr>
<th>DEPTH</th>
<th>ROCK OR SOIL TYPE</th>
<th>DESCRIPTION OF CORE/soil type</th>
<th>ROCK STRUCTURES (Defects)</th>
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<th>WATER PRESSURE TESTS</th>
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### WEATHERING

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<th>Moderately weathered</th>
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### HOLE NO: 6651

### LOCATION: Hectile Grounds

### PROJECT: Huntly Subsidence Feature N2E0, Hectile

### GRID REF: 623907/10, 381497/20

### MWD CO-ORD.

### DIRECTION: ""R""""R"

### ANGLE FROM HORIZONTAL: 0.0°

### DATUM: GEORETIC

### H.A.D. GROUND: 21:33

### WATER LEVEL: 0.0

### LOSS: 0.0

### PERMEABILITY: 0.0

### PEAKING: 0.0

### MUD TYPE: 0.0

### WATER PRESSURE: 0.0

### LOGGED: 0.0

### PROJECT: 0.0

### DATE: 0.0

### LENGTH: 0.0

### CHECKED: 0.0

### CORE BOXES: 0.0

### VERTICAL SCALE: 0.0

### SHEET: 0.0

---

**EXPLANATION:**

- Rock: light grayish brown, in parts clayey sandstone, thin to medium bedded, locally fossiliferous. Carboniferous age. 
- Fractures: few, minor, mainly fracturing of the shale.<

---

**FINISHED:**

- Date: 3/15/88
- Depth: 0.32 m
- Type of core: coring
- Core length: 1250 mm

---

**NOTES OF CORE BOXES:**

- Core is interpreted from geophysical logs.
## LOG OF DRILL HOLE

**HOLE NO.** 6651

**LOCATION** Hostel Grounds

**H.A.D. GROUND** 21.73

### DESCRIPTION OF CORE

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<th>DIRECTION</th>
<th>ROUGHNESS</th>
<th>ROCK STRUCTURES (Defects)</th>
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<td>Little</td>
<td>15.0 - 16.0</td>
<td>North</td>
<td>Rough</td>
<td>Joint Vein, Seams, Shatter, Shear &amp; Crush Zones, Fracture, Kinking</td>
<td>-</td>
<td>-</td>
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<td>-</td>
<td>-</td>
<td>Loss with core loss interpreted from geophysical logs</td>
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**DRILLER:** T. Hayman

**STARTED:** 8.1.85

**FINISHED:** 15.1.85

**DRILL:** Elgin 1250

**DATE:** 8/15/85

**LENGTH:** 67.25

**CHECKED:** P. Relph

**CORE BOXES:**

**SCALE:** 1:50

**SHEET:** 4...

**ORIG NO.:**
### Log of Drill Hole

**Project:** Huntly, Subsidence, Feature NZER Hostel  
**Location:** Hostel Grounds  
**Grid Ref:** 625 957 19 335 432 70  
**M.W.D. Co-Ord.:**  
**Datum:** Geodetic  
**Hole No.:** 6651  
**H.A.D. Ground:** 2173

#### Description of Core

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#### Core Description

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<td>63</td>
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<tr>
<td>Dark grey or black coal</td>
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#### Rock Structures (Defects)

- **White Vein:** 61-62 ft
- **Shear Zone:** 62-63 ft
- **Foliation Schistosity:** 63-64 ft
- **Contact alteration:** 64-65 ft

#### Water Pressure Tests

- **Pore Pressure:** 61-62 ft
- **Fracture Pressure:** 62-63 ft

---

**Driller:** J. Gordon  
**Drill:** 1250  
**Start:** 1.8.53  
**Finish:** 14.1.53

**Explanations:**

- Core bore hole depth is recorded (and) off with some clay (Pluvialite).
- Light green sandstone with zone of clay (Pluvialite).
- Light grey sandstone with zone of clay (Pluvialite).
- Dark grey sandstone with zone of clay (Pluvialite).

---

**Log Sheet:** A, OR, R

---

**Date:** 8.1.53  
**Hole No.:** 6651  
**Length:** 6725 ft

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**Acknowledgements:**

- [Names of acknowledgments]

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**References:**

- [References and sources]
## LOG OF DRILL HOLE

### PROJECT
Huntey Subsurface Feature

### GRID REFERENCE
NG 35 35

### MWD CO-ORD

### ANGLE FROM HORIZONTAL
030°

### DESCRIPTION OF CORE

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</tr>
<tr>
<td>Light Brownish Gray Shale</td>
<td>Light brownish grey, fine gravel, smooth friable</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>Light Brownish Gray Shale</td>
<td>Light brownish grey, fine gravel, smooth friable</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>Light Brownish Gray Shale</td>
<td>Light brownish grey, fine gravel, smooth friable</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>Light Brownish Gray Shale</td>
<td>Light brownish grey, fine gravel, smooth friable</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
</tbody>
</table>

### WEATHERING
- Unweathered
- Very weak
- Moderately strong
- Weak

### HARDNESS
- Very hard
- Hard
- Moderately hard
- Soft
- Very soft

### FRACTURE LOG
- Spacing of natural fractures
- Fractures in shatter

### WATER PRESSURE
- Low
- Medium
- High

### DRILLER
- Name: [Name]
- Started: [Date]
- Finished: [Date]

### EXPLANATION
- Description: [Description]
- Notes: [Notes]

### LOGGED
- By: [Name]
- Date: [Date]

### TRAILED
- By: [Name]
- Length: [Length]

### CHECKED
- By: [Name]

### VERTICAL SCALE
- 1:50

### SHEET
- 1 of 5

### PHOTO NO.
- HAD COLLAR

---

**Sheet 1 of 5, DRG No. [Number]**
**LOG OF DRILL HOLE**

**PROJECT:** [Hypothetical Subsidence]  
**FEATURE:** NZED Hotel  
**LOCATION:** Adder Street  
**M.W.D. CO-ORD.:** [Data not visible]  
**ANGLE FROM HORIZONTAL:** [Data not visible]  
**DIRECTION:** [Data not visible]  
**HOLE NO.:** 6652  
**HAD GROUND:** 12 ft

### DESCRIPTION OF CORE

<table>
<thead>
<tr>
<th>Depth</th>
<th>Rock Type</th>
<th>Color</th>
<th>Texture</th>
<th>Fracture Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-1 ft</td>
<td>Light yellowish grey phosphite</td>
<td>Light fine sand</td>
<td>Loose to clastic</td>
<td>Slight cleavage</td>
</tr>
<tr>
<td>1-2 ft</td>
<td>Dark grey, crumbly cement</td>
<td>Dark fine sand</td>
<td>Loose to clastic</td>
<td>Slight cleavage</td>
</tr>
<tr>
<td>2-3 ft</td>
<td>Dark green, clayey cut</td>
<td>Dark fine sand</td>
<td>Loose to clastic</td>
<td>Slight cleavage</td>
</tr>
</tbody>
</table>

---

**LOG OF DRILL HOLE**

**DESCRIPTION OF CORE:** [Data not visible]  
**HEATNG:** [Data not visible]  
**MOISTNESS:** [Data not visible]  
**FRACTURE LOG:** [Data not visible]  
**WATER DRILL:** [Data not visible]  
**WATER PRESSURE:** [Data not visible]

---

**DRILLER:** [Data not visible]  
**HEADING:** [Data not visible]  
**MAGNITUDE:** [Data not visible]  
**FRACTURE LOG:** [Data not visible]  
**LOADED PRESSURE:** [Data not visible]  
**DATE:** [Data not visible]  
**TRACED:** [Data not visible]  
**LENGTH:** 55 ft  
**CHECKED:** [Data not visible]  
**CORE BOXES:** [Data not visible]  
**VERTICAL SCALE:** 1:50  
**SHEET:** A OR S  
**DRAW NO.:** [Data not visible]
**LOG OF DRILL HOLE**

**PROJECT:** Hankey Sandstone, Feature N20. Hostel  
**GRID REF:** 515 210.26 511 337.26  
**MWD CO-ORD.:**  
**LOCATION:** Rosita Street  
**DATE:**  
**HOLE NO.:** 6452  
**DAM. GROUND:** 1704  
**PHOTO NO.:**  
**H.D. COLLAR:**  

<table>
<thead>
<tr>
<th>CONSTRUCTION LOSS</th>
<th>HAD</th>
<th>ROCK STRUCTURES (DEFORMED)</th>
<th>JOINTS VER.K. SEAMS, SHATTER, SHEAR &amp; CRUSH ZONES</th>
<th>FISSILE, SCHISTOSITY</th>
<th>(OR SOIL DESCRIPTION)</th>
<th>GRAPHIC LOG</th>
<th>WATER LEVEL</th>
<th>WATER LEVEL</th>
<th>WATER PRESSURE</th>
<th>TESTS</th>
<th>TRENCHING</th>
<th>LOCATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>5</td>
<td>SHEAR ZONE</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>SHEAR ZONE</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DESCRIPTION OF CORE**  
- Light greenish brown potashitic  
- Light yellowish brown potashitic  
- Light greenish brown clayey silt  
- Light blue clay silt, medium to coarse sand  
- Olive white potashitic clayey silt  
- Dark greenish clayey silt  
- Yellowish grey silt clay  
- Dark olive brown carbonaceous silt clay  

**WEATHERING**  
- UW - Unweathered  
- SW - Slightly weathered  
- HW - Highly weathered  
- CH - Completely weathered

**HARDNESS**  
- VH - Very hard  
- NH - Moderately hard  
- MS - Moderately soft  
- S - Soft  
- VS - Very soft

**FRACTURE LOG**  
- Spacing of natural fractures  
- Fractured

**EXPLANATION**  
- Two or three

**DRILLER:**  
- Started: 11/18  
- Finished: 12/18  
- Diameter: 5500-5501 ft  
- 5502-5503 m  

**LOGGED BY:**  
- Project:  
- Date:  
- Hole No.: 6452  
- Traced:  
- Length: 55.75  
- Checked:  
- Vertical Scale: 1:50  
- Sheet:  
- Core Boxes:  
- ORG. NO.: 
**LOG OF DRILL HOLE**

**PROJECT:** Boulders Sandstone Feature, N260, Hole 6

**GRID REF:** 888.85 m, 358.638 83 m W.D. CO-ORD.

**LOCATION:** Boulders, Bluff

**HOLE NO.** 6

**H.A.D. GROUND** 6/5

**DESCRIPTION OF CORE**
- **FORMATION NAME:**
- **ROCK OR SOIL TYPE:**
- **DESCRIPTION OF CORE:**
  - Light tan-brown silt with some sand
  - Yellow brown sand
  - Yellowish brown sand
  - Yellowish brown silt
  - Yellowish brown sand
  - Yellowish brown sand
  - Yellowish brown sand

**WEATHERING**
- **WEATHERED:**
  - SW - Slight weathered
  - NW - Moderately weathered
  - HW - Highly weathered
  - DW - Very weathered

**HARDNESS**
- **HARDNESS:**
  - VH - Very hard
  - MM - Moderately hard
  - MS - Moderately soft
  - VS - Very soft

**WEATHERING & HARDNESS**
- **WEATHERING:**
  - SW - Slight weathered
  - NW - Moderately weathered
  - HW - Highly weathered
  - DW - Very weathered

**FRACTURE LOG**
- **SPACING:**
  - 0 - 2
  - 2 - 5
  - 5 - 10

**WATER LEVEL LOSS**
- **LOSS:**
  - 0 - 1
  - 1 - 2

**WATER PRESSURE TESTS**
- **PERMEABILITY:**
  - PC -Permeability

**TRANSLATIONAL BOUNDARY**
- **BOUNDARY:**
  - tmb

**EXPLANATION**
- **EXPLANATION:**
  - Transitional boundary
  - Gypsum, feldspar, gypsite, and other minerals

**LOGGED, PROJECT**
- **LOGGED:** 6/5/86
- **PROJECT:** Boulders Sandstone Feature

**DATE:** 6/7/86
- **HOLE NO.:** 6

**LENGTH:** 67.95 m

**SHEET: 1 OF 4**

**DRAWN:**
- **DRAWN BY:**
  - L. R. K.

**SCALE:**
- **SCALE:** 1:50

**DRAWN ON:**
- **DRAWN ON:** 12/20/88
LOG OF DRILL HOLE

PROJECT: Huntly Subdivision  FEATUERE 1200 Hotel
GRID REF: 5233.982,5343.6184  MWD. CO-ORD.
ANGLE FROM HORIZONTAL: 0.90°  DIRECTION:

LOCATION:  Burke Place  CATUM:  Geotechnical
H.A.D. GROUND: 4157

HOLE NO. 6654  PHOTO NO.

H.A.D. COLLAR

DEEP LOG OF DRILL HOLE

SECTION

DESCRIPTION OF CORE
FORMATION NAME: ROCK OR SOIL TYPE:

DESCRIPTION OF CORE (granular, soil, mineral content, texture, color, etc.
inter.tags: text, cements, and matrix details)

CORE LOSS H.A.D.
DEPTH

FRAC TURE LOG
REMARKS OR SOIL DESCRIPTION:
CRUSH ZONES. FRACTURE MORPHOLOGY:
(fracture, thickness, spacing, orientation, etc.)

WATER DRILL LEVEL
WATER PRESSURE

TESTS

LOGGED AREA

EXPLANATION

WEATHERING
HARNESS
FINITE LOG (cm)

DATE: 12/20/96
PROJECT:
HOLE NO. 6654

TRACED:

LENGTH: 67.94

CHECKED:

CORE BOXES:

SCALE:

SHEET: A, OF: 8  DRG NO.:
**LOG OF DRILL HOLE**

**PROJECT:** Northumberland
**FEATURE:** NZEO Hostel
**GRID REF:** 123.50 N 650.50 W
**ANGLED FROM HORIZONTAL:** 50°
**HOLE NO.:** 6654
**LOCATION:** Burke Place
**DATUM:** Geodetic
**H.A.D. GROUND:** 15.69

<table>
<thead>
<tr>
<th>DESCRIPTION OF CORE</th>
<th>ROCK OR SOIL TYPE</th>
<th>WEATHERING</th>
<th>HARDNESS</th>
<th>CORE HEIGHT</th>
<th>CORE LOSS (HA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIGHT YELLOW BROWN DINK</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>YELLOWISH WHITE POROUS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>YELLOWISH WHITE POROUS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>YELLOWISH WHITE CLAYISH</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WHITE CLAYISH</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**WEATHERING:**
- UW = Unweathered
- SW = Slightly weathered
- MW = Moderately weathered
- HW = Highly weathered
- CW = Completely weathered

**HARDNESS:**
- Vf = Very hard
- Mf = Moderately hard
- Sf = Slightly hard
- V = Very soft

**FRACTURE LOG:**
- Spacing of natural fractures
- Natural fractures
- Preferential fractures

**WATER PRESSURE TESTS:**
- DATE: 17/8/85
- LENGTH: 77.9

**CHECKED:**
- PROJECT:
- CORE BOXES:
- VERTICAL SCALE: 1:50
- SHEET: 3 OF A

**SHEEDED 3/25 M**
## LOG OF DRILL HOLE

**PROJECT:** Monthly Subsidence Feature N20.5 Hostel

**GRID REF:** 8331355 in 365 MW D. CO-ORD:

**LOCATION:** Hostel Grounds

**ANGLE FROM HORIZONTAL:**

**DATUM:** Geodetic

**H.D. GROUND:** 2778

### DESCRIPTION OF CORE

**FORMATION NAME:**

**ROCK OR SOIL TYPE:**

**DESCRIPTION OF CORE:**

- Tuffaceous grey brown altered gouge slate with some phyllosilicates.
- Light greenish brown and maroon brown tuffaceous sandstone with minor clay.
- Light brown tuffaceous mudstone to tuffaceous sand with minor clay.
- Yellowish white phyllitic sandstone with minor sandy clay.
- Light tan tuffaceous sandstone with minor phyllosilicates.

### WEATHERING

- UV - Unweathered
- SW - Slightly weathered
- MW - Moderately weathered
- VW - Very weathered

### HARDNESS

- V - Very hard
- H - Hard
- M - Moderately hard
- S - Soft

### CORE MAPPING

- Date: 01-05-85

### DRILLER:
- S. Gibson

### EXPLANATION

- 0-12.50 by S. Gibson

### WEATHERING

- UV - Unweathered
- SW - Slightly weathered
- MW - Moderately weathered
- VW - Very weathered

### HARDNESS

- V - Very hard
- H - Hard
- M - Moderately hard
- S - Soft

### FRAGMENTATION

- AV - Angular
- RA - Rouded
- FA - Flat

### FRAC. LOG

<table>
<thead>
<tr>
<th>Depth</th>
<th>Weathering</th>
<th>Hardness</th>
<th>Fracture</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>UV</td>
<td>V</td>
<td>AV</td>
<td>Tuffaceous grey brown altered gouge slate with some phyllosilicates.</td>
</tr>
<tr>
<td>1</td>
<td>UV</td>
<td>V</td>
<td>AV</td>
<td>Light greenish brown and maroon brown tuffaceous sandstone with minor clay.</td>
</tr>
<tr>
<td>2</td>
<td>UV</td>
<td>V</td>
<td>AV</td>
<td>Light brown tuffaceous mudstone to tuffaceous sand with minor clay.</td>
</tr>
<tr>
<td>3</td>
<td>UV</td>
<td>V</td>
<td>AV</td>
<td>Yellowish white phyllitic sandstone with minor sandy clay.</td>
</tr>
<tr>
<td>4</td>
<td>UV</td>
<td>V</td>
<td>AV</td>
<td>Light tan tuffaceous sandstone with minor phyllosilicates.</td>
</tr>
</tbody>
</table>

### LOGGED EQUIPMENT

- PROJECT: N20.5 Hostel
- HOLE NO: 6655
- DATE: 01-05-85
- LENGTH: 13.50
- CHECKED: 13.50
- VERTICAL SCALE: 1:20

### SHEET: 1/2 of 8

### ORG NO

- 3/13 M
### LOG OF DRILL HOLE

**PROJECT:** Hunter Surface
**FEATURE:** NIED Hostel
**LOCATION:** Hostel Grounds
**GRID REF.:** 21N 132973 E 5517213 M.W.D.
**COORD.:**
**DATE:** 14/4/89
**HOLE NO.:** 6655
**HAD. GROUND:** 27.78 ft
**HOLE RECORDED:**

<table>
<thead>
<tr>
<th>DESCRIPTION OF CORE</th>
<th>FORMATION NAME</th>
</tr>
</thead>
<tbody>
<tr>
<td>I grindstone grey brownphyric clayey silt with some morel to coarse sand</td>
<td></td>
</tr>
<tr>
<td>GREEN WHITE PHYRIC CLAYEY Silt with some gravel to coarse sand</td>
<td></td>
</tr>
<tr>
<td>LIGHT YELLOW BROWN CLAYEY Silt</td>
<td></td>
</tr>
</tbody>
</table>

**WEATHERING:**
- UW - Unweathered
- SW - Slightly weathered
- M - Moderately weathered
- GW - Gently weathered
- CW - Completely weathered

**HARDNESS:**
- VH - Very hard
- H - Hard
- M - Moderately hard
- MS - Moderately soft
- S - Soft
- VS - Very soft

**WEATHERING:**

**HARDNESS:**

<table>
<thead>
<tr>
<th>WEATHERING</th>
<th>HARDNESS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**EXPLANATION:**

**DRILLER:**
- E. J. R. G. G.

**STARTED:**
- 14/4/89

**FINISHED:**
- 14/4/89

**DRILL:**
- Carding Bros. Ltd.

**LOGGED MATERIALS:**

**DEPTH:**

**DIRECTION:**

**LOG:**

**SCALE:**

**DATE:**

**HOLE NO.:** 6655

**LENGTH:** 39.90 ft

**CHECKED:**

**DRAWN:**

**SHEET:**

**DRAWN BY:**

**COPY BY:**

**DRAWN ON:**

**COPY ON:**
**LOG OF DRILL HOLE**

**PROJECT:** Unnamed Site

**LOCATION:** Reserva, Behind Kotikia House

**DESCRIPTION OF CORE**

<table>
<thead>
<tr>
<th>Weathering</th>
<th>Hardness</th>
<th>Fracture Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>UV - Unweathered</td>
<td>Vs - Very soft</td>
<td>Spacing of fractures</td>
</tr>
<tr>
<td>SW - Slightly weathered</td>
<td>Vs - Very soft</td>
<td>Fracture of core</td>
</tr>
<tr>
<td>HW - Highly weathered</td>
<td>Ve - Very hard</td>
<td>Spacing of fractures</td>
</tr>
<tr>
<td>CV - Completely weathered</td>
<td>Ve - Very hard</td>
<td>Fracture of core</td>
</tr>
</tbody>
</table>

**Rocks A**

- **Dark grey brown sandy silt**
- **Dark grey brown sand**
- **Silty sand**
- **Light blue grey clayey silt**
- **Light brown, clouey silt**
- **Light yellow brown siltstone**

**Rocks B**

- **Dark grey brown sandy silt**
- **Dark grey brown sand**
- **Silty sand**
- **Light blue grey clayey silt**
- **Light brown, clouey silt**
- **Light yellow brown siltstone**

**Rock Structures (Defects)**

- **Joints Vents Shear Shatter Shave & Crushing Zones:**
  - **Sedimentary:**
  - **Sedimentary:**
  - **Sedimentary:**
  - **Sedimentary:**
  - **Sedimentary:**
  - **Sedimentary:**

**Fracture Log**

- **Spacing of fractures:**
  - **Fracture of core:**
  - **Fracture of core:**
  - **Fracture of core:**
  - **Fracture of core:**
  - **Fracture of core:**
  - **Fracture of core:**

**LOGGED:**creation date

**DATE:** 11/28/88

**HOLE NO.:** 6455

**LENGTH:** 4.60

**TRACING:**

**CHECKED:**

**CORE BOXES:**

**VERTICAL SCALE:** 1:50

**SHEET:** 1 of 10, DRILL NO.
LOG OF DRILL HOLE

PROJECT: Homestead Subsidence Feature N.Z.E.D. Haste
GRID REF.: 346.36. MS 270.72
M.W.D. CO-ORD.: 0.00
ANGLE FROM HORIZONTAL: 0.0°
CORE LOSS: 0.00

DESCRIPTION OF CORE
FORMATION NAME: N/A
ROCK OR SOIL TYPE: N/A
DESCRIPTION OF CORE (gran size, color, moisture content, hardness, etc.):

WEATHERING:
- LW - Unweathered
- SW - Slightly weathered
- MW - Moderately weathered
- CW - Completely weathered

HARDNESS:
- VH - Very hard
- H - Hard
- MH - Moderately hard
- M - Modestly hard
- S - Soft
- VS - Very soft

FRACTURE LOG:
- Spacing of natural fractures:
  - F - Fractures
  - N - No fractures

ROCK STRUCTURES (Defects):
- Jointing:
- Veins:
- Sceals:
- Shatter:
- Shearing:
- Crush Zones:
- Faulting:
- Schistosity:

(OR SOIL DESCRIPTION)
- Consistency:
- Porosity:
- Water content:
- Group symbol:

WATER PRESSURE TESTS
- Permeability:
- Logod:

EXPLANATION:
15.00-21.38 Evident
200 of core less integrated from geophysical log.

DRILLER:
I. Bown

STARTED:
197.85
FINISHED:
197.85

DATE:
25/02/78

CHECKED:
I. Bown

VERTICAL SCALE:
1:50

SHEET: P.O.A.

LOGGED:
M. Jones
PROJECT:

LOCATION:
M. Jones
Hole No.: 6457

LENGTH:
21.38

CORE BOXES:

M. Jones

ORG NO.

SQ 3/75 M
**LOG OF DRILL HOLE**

<table>
<thead>
<tr>
<th>HOLE NO.</th>
<th>6599</th>
</tr>
</thead>
<tbody>
<tr>
<td>PROJECT</td>
<td>Hunt Oil Asset</td>
</tr>
<tr>
<td>GRID REF.</td>
<td>M54, 671, 743</td>
</tr>
<tr>
<td>M.W.D. CO-ORD.</td>
<td>171</td>
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<tr>
<td>ANGLE FROM HORIZONTAL</td>
<td>1°</td>
</tr>
<tr>
<td>DESCRIPTION OF CORE</td>
<td></td>
</tr>
<tr>
<td>FORMATION NAME</td>
<td></td>
</tr>
<tr>
<td>ROCK OR SOIL TYPE</td>
<td></td>
</tr>
<tr>
<td>DESCRIPTION OF CORE</td>
<td></td>
</tr>
<tr>
<td>CORE LOSS (LIT.)</td>
<td></td>
</tr>
<tr>
<td>CORE HARDNESS</td>
<td></td>
</tr>
<tr>
<td>CORE WEATHERING</td>
<td></td>
</tr>
<tr>
<td>DAMAGE LEVEL</td>
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<tr>
<td>WEATHERING LOG</td>
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</tr>
<tr>
<td>DEPTH</td>
<td></td>
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<td>INTERVAL</td>
<td></td>
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<tr>
<td>LOG</td>
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</tr>
<tr>
<td>ROCK STRUCTURES (Defects)</td>
<td></td>
</tr>
<tr>
<td>JOINTS, VEINS, SEAM, SHATTER, SHEAR &amp; CRUSH ZONES, FOLIATION, SCHISTOSITY</td>
<td></td>
</tr>
<tr>
<td>(Specified, increase spacing, smoothen, interpreted)</td>
<td></td>
</tr>
<tr>
<td>OR SOIL DESCRIPTION</td>
<td></td>
</tr>
<tr>
<td>WATER TESTS</td>
<td></td>
</tr>
<tr>
<td>WATER PRESSURE</td>
<td></td>
</tr>
<tr>
<td>DRAINS</td>
<td></td>
</tr>
<tr>
<td>SHEET</td>
<td></td>
</tr>
</tbody>
</table>

**Sedimentary White Silty Sand (Recent)**

**Sedimentary Brown Mottled Silty Sand (Recent) with some fine gravel. (Recent)**

**Light Yellowish Ochre Porous Clayey Silt with clayey silt. (Fluvial)**

**Light Yellowish Ochre Porous Clayey Silt with clayey silt. (Fluvial)**

**Clayey Silt with clayey silt. (Fluvial)**

**Driller:**

**Explanations:**

**Weathering:**

- Unweathered
- Moderately weathered
- Highly weathered
- Completely weathered

**Hardness:**

- Very hard
- Hard
- Moderately hard
- Moderately soft
- Soft

**Fracture Log:**

- Spacing of natural fractures
- Fractured (in mm)

**Logged:**

**Date:**

**Checked:**

**Length:**

**Core Boxes:**

**Scale:**

**Notes:**

- ...
### SUMMARY LOG

**PROJECT:** Huntley Subsidence  
**FEATURE:** N.E.D.D. Hostel  
**LOCATION:** Hostel Grounds  
**DATE:** Geodetic  
**H.O. GROUND:** 21 75  
**HOLE NO.:** 6651  
**PHOTO NO.:** 6  
**H.A. COLLAR:**

#### DESCRIPTION OF CORE
- **FORMATION NAME:**  
- **ROCK OR SOIL TYPE:**
  - Lenticular Vuggy Limestone
  - Fine to Medium Sandy Clay
  - Fine to Medium Sand
  - Sandy Clay
  - Medium Sand
  - Fine to Medium Sandy Clay
  - Fine to Medium Sand
  - Fine to Medium Sandy Clay

#### ROCK STRUCTURES (Defects)
- **JOINTS, VEINS, SEMI, SPLIT, SHEAR, & CRUSH ZONES:**
  - Lenticular Vuggy Limestone
  - Fine to Medium Sandy Clay

#### WEATHERING
- **UW - Unweathered**  
- **SW - Slightly weathered**  
- **MW - Moderately weathered**  
- **HW - Highly weathered**  
- **CW - Completely weathered**

#### DRILLING
- **DRILLER:**  
- **STARTED:** 8/25  
- **FINISHED:** 8/28  
- **HOLE:** 1150

#### GEOLOGY
- **WEATHERING:**
  - **UW - Unweathered**
  - **SW - Slightly weathered**
  - **MW - Moderately weathered**
  - **HW - Highly weathered**
  - **CW - Completely weathered**

#### HOLE INFORMATION
- **WEIGHT:**
  - **DRILLING:**
    - **ASTORE:**
      - **LENGTH:**
        - **DATE:**
          - **LOGGED:**
            - **PICKUP:**
              - **PROJECT:**
                - **HOLE NO.:**
                  - **TRACED:**
                    - **LENGTH:**
                      - **CHECKED:**
                        - **CORE BOXES:**
                          - **VERTICAL SCALE:**
                            - **DATE:**
                              - **ORIG:**
                                - **SHEET:**
                                  - **OK:**
                                    - **SIGN:**
### SUMMARY LOG

**PROJECT**: Holey Suspense, Feature N2E0, Hette
**LOCATION**: Rock, Page
**GRID REF**: SSW1 1883 1883 1883 1883
**MWD CO-ORD.**: GEOLOGIC
**ANGLE FROM HORIZONTAL**: 0.90°
**DIRECTION**: GEOLOGIC
**HOLE NO.**: 6654
**H.D. GROUND**: 41.59

<table>
<thead>
<tr>
<th>DESCRIPTION OF CORE</th>
<th>CORE LENGTH</th>
<th>HOLE DEPTH</th>
<th>ROCK STRUCTURES (Defects)</th>
<th>WATER PRESSURE TESTS</th>
<th>GRADING</th>
<th>H.D. COLLAR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Silty clay and silt (shale)</td>
<td>2.5</td>
<td>5.0</td>
<td>Fracture: None</td>
<td>None</td>
<td>Flat</td>
<td>6654</td>
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<tr>
<td>3. Silty clay and silt (shale)</td>
<td>2.5</td>
<td>7.5</td>
<td>Fracture: None</td>
<td>None</td>
<td>Flat</td>
<td>6654</td>
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<tr>
<td>4. Silty clay and silt (shale)</td>
<td>2.5</td>
<td>10.0</td>
<td>Fracture: None</td>
<td>None</td>
<td>Flat</td>
<td>6654</td>
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</table>

**DRILLER**: G. Keen
**STARTED**: 11/12/84
**FINISHED**: 21/12/84
**DRILL**: Godfrey Gaged 200T

**WEATHERING**
- UV = Unweathered
- SW = Slightly weathered
- MW = Moderately weathered
- HW = Highly weathered
- CW = Completely weathered

**WEATHERING**
- VH = Very hard
- H = Hard
- MH = Moderately hard
- M = Medium hard
- V = Very soft

**FRACTURE LOG**
- Spacing of fracturesses

<table>
<thead>
<tr>
<th>Fracture/Dimension</th>
<th>Log</th>
<th>Fracture</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>UV</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SW</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MW</td>
<td></td>
</tr>
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</table>

**LOGGED**: 27/12/84
**PROJECT**: Holey Suspense
**DATE**: 27/12/84
**HOLE NO.**: 6654
**TRACED**: 27/12/84
**LENGTH**: 20.00 m
**CHECKED**: 27/12/84
**CORE COCKS**: 200T
**SCALE**: 1:200

**SHEET**: 6, 6, 6, 6
**ORG NO.**:
SUMMARY LOG

PROJECT: Hurley Subsidence Feature N30 E Hostel
GRID REF: NZ 63 31 38 48 F E MWD
LOCATION: Hotel, Gueiros
ANGEL FROM HORIZONTAL: PHOTO NO:
DIRECTION:

DESCRIPTION OF CORE

FORMATION NAME
ROCK OR SOIL TYPE
DESCRIPTION OF CORE (grain size, color, texture, etc.)

WEATHERING
HARDNESS
WEIGHT
DRILL AND TEST

CORE LOSS: HAD. LOG
HAD. COLLAR

WEATHERING
HARDNESS

LOGGED: PROJECT:
DATE: 11/08/88
HOLE NO: 6655
LENGTH: 68.50

CHECKED:
CORE BOX:

VERTICAL SCALE:

SHEET: 8 OF 8
DRAWN NO:

Driller: Derek (signature)
Started: 11/08/88
Finished: 11/08/88

Explanation
APPENDIX 3: Geophysical logs and penetrometer soundings
Figure A33: Field Data Logs for Borehole 6654
Figure A3.4: Field Data Logs for Borehole 6655
Figure A.35: Field Data Logs for Borehole 6656
Figure A.3.6: Field Data Logs for Borehole 6657
APPENDIX 4: Field permeability falling head test data
### Falling Head Test

<table>
<thead>
<tr>
<th>Borehole co-ordinates</th>
<th>Collar elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth to top of test section, $H_w$</td>
<td>27.90 m</td>
</tr>
<tr>
<td>Length of test section, $L$</td>
<td>2.5 m</td>
</tr>
<tr>
<td>Depth of static water level, $H_w$</td>
<td>14 m</td>
</tr>
<tr>
<td>Radius of borehole, $r$</td>
<td>0.1 m</td>
</tr>
<tr>
<td>Excess head, $h_e$</td>
<td>7.44 m</td>
</tr>
<tr>
<td>Radius of standpipe or casing, $r_c$</td>
<td>0.02 m</td>
</tr>
</tbody>
</table>

| Time, $T$ (min) | 0.17 | 0.29 | 0.25 | 0.30 | 0.35 | 0.40 | 0.45 | 0.50 | 0.55 | 0.60 | 0.66 |
| Depth to water, $h_w$ (m) | 6.89 | 6.39 | 6.14 | 5.64 | 5.14 | 4.64 | 4.14 | 3.64 | 3.14 | 2.64 | 2.14 |
| Excess head, $h'_e = h_w - h_w$ (m) | 0.97 | 0.93 | 0.86 | 0.79 | 0.72 | 0.65 | 0.58 | 0.51 | 0.44 | 0.37 | 0.30 |

**Head-time graph (slope of graph is $S$)**

**Calculations**

- **Permeability** $k = 0.133 \frac{L^2}{S}$ m/sec
- $S = \frac{\log_{10} \left( \frac{10}{h_e} \right)}{15} = 9.52 \times 10^{-2}$
- $k = 1.77 \times 10^{-5}$ m/s
**FALLING HEAD TEST**

<table>
<thead>
<tr>
<th>Borehole co-ordinates</th>
<th>Collar elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth to top of test-section</td>
<td>16.20 m</td>
</tr>
<tr>
<td>Length of test section, L</td>
<td>2.7 m</td>
</tr>
<tr>
<td>Depth of static water level, Hw</td>
<td>6.09 m</td>
</tr>
<tr>
<td>Radius of borehole, r</td>
<td>m</td>
</tr>
<tr>
<td>Excess head, he</td>
<td>6.09 m</td>
</tr>
<tr>
<td>Radius of standpipe or casing, r0</td>
<td>0.02 m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Time, T (min)</th>
<th>0</th>
<th>68</th>
<th>115</th>
<th>225</th>
<th>308</th>
<th>488</th>
<th>688</th>
<th>922</th>
<th>1193</th>
<th>15</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth to water, hw (m)</td>
<td>0.25</td>
<td>0.5</td>
<td>0.75</td>
<td>1.00</td>
<td>1.50</td>
<td>2.0</td>
<td>2.5</td>
<td>3.0</td>
<td>3.48</td>
<td></td>
</tr>
<tr>
<td>Excess head, h_e = H_w - h_w (m)</td>
<td>5.84</td>
<td>6.59</td>
<td>7.34</td>
<td>8.09</td>
<td>8.74</td>
<td>9.59</td>
<td>10.39</td>
<td>11.18</td>
<td>12.16</td>
<td></td>
</tr>
<tr>
<td>h_t/h_e</td>
<td>0.96</td>
<td>0.92</td>
<td>0.88</td>
<td>0.84</td>
<td>0.80</td>
<td>0.75</td>
<td>0.70</td>
<td>0.65</td>
<td>0.60</td>
<td></td>
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</tbody>
</table>

**Head - time graph (slope of graph is S)**

**Calculations**

Permeability

\[ k = 0.133 S \left( \frac{r_c^2}{L} \right) \text{ m/sec.} \]

\[ S = \frac{\log_{10} \left( \frac{H_w}{L} \right)}{15} \]

\[ k = 6.81 \times 10^{-7} \text{ m/sec} \]
APPENDIX 5: Consolidation test data
Determination of the One Dimensional Consolidation Properties

Sample Date: 6551-6-08
Sample Test: 6551-6-08

Sample Location: 5123-6-08
Sample Depth: 6-08

Sample Size: 12.70

**Test Details**

- **Soil Description:** Undisturbed/air-dried/compressed from field.
- **Laboratory Tests:**
  - **Soil Type:** Silt
  - **Sample No.:** 3
  - **Cell No.:** 3
  - **Diameter of ring (d):** 76.20 mm
  - **Height of ring (h):** 76.20 mm
  - **Ring No.:** 5.152
    - **Area of ring (A):** 1,520.77 mm²

<table>
<thead>
<tr>
<th>Stage</th>
<th>Measured thickness of specimen (t1)</th>
<th>Initial</th>
<th>Final</th>
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<tbody>
<tr>
<td></td>
<td>25</td>
<td>18.94</td>
<td>13.98</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>25.38</td>
<td>21.44</td>
</tr>
<tr>
<td></td>
<td>100</td>
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<td>27.95</td>
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<tr>
<td></td>
<td>200</td>
<td>46.58</td>
<td>21.05</td>
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<td></td>
<td>400</td>
<td>132.10</td>
<td>21.88</td>
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<tr>
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<td>800</td>
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<td>21.88</td>
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<tr>
<td></td>
<td>2,100</td>
<td>132.10</td>
<td>21.88</td>
</tr>
</tbody>
</table>

**Laboratory Calculations**

- **Mass of ring with glass + empty:** 61 g
- **Mass of glass:** 55 g
- **Mass of dry specimen:** 5 g
- **Mass of water:** 7 g
- **Water content in:** 11.6

**Laboratory Results**

- **Dry density:** 19.84
- **Gradation ratio:** 2.98
- **Gradation ratio:** 3.11
- **Degree of saturation:** 92.3

**Applied Pressure vs. Time**

<table>
<thead>
<tr>
<th>Applied Pressure</th>
<th>Time and Time of Application</th>
<th>Incremental drained Def.</th>
<th>Incremental thickness of specimen</th>
<th>% Change in sample</th>
<th>B (W-H1)</th>
<th>Voids ratio (W-H1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>50</td>
<td>0.025</td>
<td>18.94</td>
<td>13.98</td>
<td>11.6</td>
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<td>0.052</td>
<td>25.38</td>
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<td>200</td>
<td>0.130</td>
<td>32.30</td>
<td>27.95</td>
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<td>0.249</td>
<td>46.58</td>
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<td>0.567</td>
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<td>21.88</td>
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<td>0.956</td>
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<td>1.154</td>
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<td>1.886</td>
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<td>21.88</td>
<td>2.98</td>
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</tbody>
</table>

* Denotes the inappropriate word.
* Corrected where necessary for the computation of apparent.
FORM D2
Determination of the One-Dimensional Consolidation Properties

Job: NZ92 Hostel Subsoil  Sample No.: 6651-646
Sampling date: 01/05  Treated by: K. Lesser
Sampling location: AH 4441  Date: 19/03/85
Sampling depth: 0.65  Checked by: K. Lesser
Ground surface elevation: 21.73  Date: 23/10/85
Water table elevation: 13.70

Test details:
- Soil description: Unstained/Unstained/Unstained/Unstained/Unstained
- Loading cycle: 24
- Specimen No.: 2
- Cell No.: 2
- Diameter of ring (D): 76.28 mm
- Height of ring (H): 15.09 mm
- Area of ring (A): \( \frac{\pi D^2}{4} \) mm²
- Solid density of soil particles measured (sand) 275 g/cm³

<table>
<thead>
<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured thickness of specimen (H)</td>
<td>mm</td>
<td>H₁ = 10.90</td>
</tr>
<tr>
<td>Mass of ring + wash glass + wet specimen</td>
<td>g</td>
<td>M₁ = 16.055</td>
</tr>
<tr>
<td>Mass of ring + wash glass + dry specimen</td>
<td>g</td>
<td>M₃ = 10.906</td>
</tr>
<tr>
<td>Mass of ring</td>
<td>g</td>
<td>M₄ = 9.567</td>
</tr>
<tr>
<td>Mass of wash glass</td>
<td>g</td>
<td>M₆ = 11.460</td>
</tr>
<tr>
<td>Mass of dry specimen</td>
<td>g</td>
<td>M₈ = M₉ = M₁ - M₂</td>
</tr>
<tr>
<td>Mass of water</td>
<td>g</td>
<td>M₉ = M₈ = 6.732</td>
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<tr>
<td>Water content w</td>
<td>%</td>
<td>w₁ = 37.3</td>
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<tr>
<td>Dry density ρ_d = ( \frac{M}{H \times A} )</td>
<td>g/cm³</td>
<td>1.85</td>
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<tr>
<td>Height of soil particles h</td>
<td>m</td>
<td>( \frac{M \times 1000}{D \times A} )</td>
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<tr>
<td>Void ratio e = ( \frac{V}{V₀} )</td>
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<td>1.256</td>
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<tr>
<td>Degree of saturation S = ( \frac{S}{S₀} )</td>
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<td>97</td>
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</table>

<table>
<thead>
<tr>
<th>Applied pressure (σ)</th>
<th>Date and time of application, hours</th>
<th>Incremental deflection (θ)</th>
<th>Thickness of specimen (H₁−D₁)</th>
<th>% change in thickness</th>
<th>Height of wash glass (H₂−H₁)</th>
<th>Void ratio (V/V₀)</th>
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</thead>
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<tr>
<td>25</td>
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<td>14.294</td>
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<td>0.151</td>
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<td>1600</td>
<td>0.143</td>
<td>13.780</td>
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<td>13.165</td>
<td>8.461</td>
<td>1.458</td>
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</tr>
</tbody>
</table>

* Delete the inappropriate words.
### Determination of One-Dimensional Consolidation Properties

**Site:** M.7 E.0 North Island  
**Sample No.:** 6651 - 6.68 m  
**Sampling date:** 6.1.85  
**Sampling location:** 644451  
**Sampling depth:** 6.68 m  
**Ground surface elevation:** 12.70 m  
**Water table elevation:** 12.70 m

**Test Details:**
- **Soil description:** Undisturbed, tilled, composted, silty sand, heath soil.
- **Loading cycle:** 7 days, 1,000 psi.

**Specimen No.:** 3  
**Cell No.:** 3  
**Diameter of ring (D):** 75.59 mm  
**Height of ring (H):** 18.09 mm  
**Ring No.:** 3  
**Area of ring (A):** 3.54721 mm²  
**Solid density of soil particles (g/cm³):** 2.75 g/cm³

<table>
<thead>
<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
</tr>
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<tbody>
<tr>
<td>Measured thickness of specimen (D) mm</td>
<td>M₁: 28.300</td>
<td>M₂: 44.503</td>
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<tr>
<td>Mass of ring + wash glass + net specimen g</td>
<td>M₃: 217.919</td>
<td>M₄: 327.502</td>
</tr>
<tr>
<td>Mass of ring + wash glass + dry specimen M₁ g</td>
<td>108.912</td>
<td></td>
</tr>
<tr>
<td>Mass of ring g</td>
<td>M₂: 87.575</td>
<td>M₃: 87.575</td>
</tr>
<tr>
<td>Mass of wash glass g</td>
<td>M₄: 77.907</td>
<td></td>
</tr>
<tr>
<td>Mass of dry specimen M₂ - M₃ - M₄ g</td>
<td>77.907</td>
<td></td>
</tr>
<tr>
<td>Mass of water g</td>
<td>M₅: 257.025</td>
<td></td>
</tr>
<tr>
<td>Water content w</td>
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<td>Water density ρw = M₅/1000</td>
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<td>Dry density ρd = M₁/1000</td>
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<tr>
<td>Height of soil particles H s = D - 2D/2 mm</td>
<td>5.922</td>
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<tr>
<td>Void ratio e = H s / D s</td>
<td>1.41</td>
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<tr>
<td>Degree of saturation S = D w / D s</td>
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<th>Applied pressure (kPa)</th>
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<th>50</th>
<th>100</th>
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<th>800</th>
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<th>4,000</th>
<th>8,000</th>
<th>16,000</th>
<th>40,000</th>
<th>800,000</th>
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<tbody>
<tr>
<td>Thickness of specimen (D)</td>
<td>18.33</td>
<td>18.13</td>
<td>17.90</td>
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<td>17.56</td>
<td>17.44</td>
<td>17.86</td>
<td>17.31</td>
<td>17.12</td>
<td>16.94</td>
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<td>Thickness of specimen (D)</td>
<td>0.120</td>
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<td>0.461</td>
<td>0.897</td>
<td>1.795</td>
<td>3.591</td>
<td>7.186</td>
<td>14.372</td>
<td>28.744</td>
<td>57.488</td>
<td>114.976</td>
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<td>Height of soil particles H s / D s</td>
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<td>0.200</td>
<td>0.397</td>
<td>0.795</td>
<td>1.591</td>
<td>3.186</td>
<td>6.372</td>
<td>12.744</td>
<td>25.488</td>
<td>50.976</td>
<td>101.952</td>
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<td>Void ratio e = H s / D s</td>
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<td>1.08</td>
<td>1.05</td>
<td>1.04</td>
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</table>

* Define the inappropriate words.
† Corrected when necessary for the comparison of apparatus.
**Determination of the One-Dimensional Consolidation Properties**

**Job:** NZE08 Hostel Subsoil  
**Sample No:** 6657-7-21  
**Sampling date:** 8-1-85  
**Sampling location:** SH 4661  
**Sampling depth:** 7.21m  
**General surface elevation:** 21.73  
**Waste table elevation:** 12.79

**Test Details:**  
- **Soil Description:** Isleworth loamclay  
- **Loading cycle:** Intermediate

**Specimen No:** 5  
**Cell No:** 1  
**Machine No:** 1  
**Ring No:** A  
**Area of ring (A) in cm²:**  
**Solid density of soil particles measured (ps):** 2.64 g/cm³

---

<table>
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<tr>
<td>Measured thickness of specimen (d) in mm</td>
<td>m₁</td>
<td>m₂</td>
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<tr>
<td>Mass of ring + wash + glass + wet specimen</td>
<td>M₁</td>
<td>245.434</td>
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<tr>
<td>Mass of ring + wash + glass + split specimen</td>
<td>M₂</td>
<td>240.812</td>
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<td>Mass of ring</td>
<td>m₃</td>
<td>92.208</td>
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<td>Mass of wash glass</td>
<td>m₄</td>
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<td>m₅ = m₂ - m₃ - m₄</td>
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<td>m₆</td>
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<td>Dry density ρ_d = m₂ / V_d</td>
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<td>Height of soil particles H = M₂ x 1000 / d/A</td>
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**Applied Pressure:**

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<th>kPa</th>
<th>mm</th>
<th>mm</th>
<th>% change thickness</th>
<th>Height of waste</th>
<th>Void ratio</th>
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<td>25</td>
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<tr>
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<tr>
<td>200</td>
<td>0.472</td>
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<tr>
<td>400</td>
<td>0.621</td>
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<tr>
<td>800</td>
<td>1.271</td>
<td>15.283</td>
<td>5.93%</td>
<td>1.532</td>
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<tr>
<td>1600</td>
<td>1.527</td>
<td>15.626</td>
<td>7.31%</td>
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<tr>
<td>3200</td>
<td>0.103</td>
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<tr>
<td>6400</td>
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<tr>
<td>12800</td>
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<td>25600</td>
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<tr>
<td>51200</td>
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<td>12.754</td>
<td>9.54%</td>
<td>1.303</td>
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---

* Delete the inappropriate words.  
† Corrected where necessary for the comprehension of apparatus.
**Determination of the One-Dimensional Consolidation Properties**

**Job:** 1974.3A.164.7

**Sample No.:** 0857

**Sampling date:** 8/1/95

**Sampling location:** E4

**Sampling depth:** 9.2m

**Ground surface elevation:** 210.9

**Water table elevation:**

**Test details:**

- *Self-description:* Undisturbed
- *Loading cycle:* 7 days

**Specimen No.:**

**Cell No.:** 1

**Diameter of ring (mm):** 76.5

**Height of ring (mm):** 45

**Ring No.:** 20

**Area of ring (mm^2):** 4208

**Soil density of soil particles measured (mm^2):** 500

<table>
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<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
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<tr>
<td>Measured thickness of specimen (mm)</td>
<td>M1 13.785</td>
<td>M2 13.795</td>
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<td>Mass of ring + wash glass + wet specimen</td>
<td>M1 237.871</td>
<td>M2 237.241</td>
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<td>Mass of ring + wash glass + dry specimen</td>
<td>M1 202.407</td>
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<tr>
<td>Mass of ring</td>
<td>M1 9.4</td>
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<tr>
<td>Mass of wash glass</td>
<td>M2 10.9</td>
<td></td>
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<td>Mass of dry specimen</td>
<td>M1 - M2</td>
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</tr>
<tr>
<td>Mass of water</td>
<td>M1 - M2</td>
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<td>Water content w (%)</td>
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</tr>
<tr>
<td>Density of specimen</td>
<td>0.795</td>
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<tr>
<td>Height of soil particle H = M2 x 1000</td>
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<tr>
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<td></td>
</tr>
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<td>Degree of saturation S = 100</td>
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<table>
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<tr>
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<th>65</th>
<th>100</th>
<th>150</th>
<th>200</th>
<th>400</th>
<th>800</th>
<th>1200</th>
<th>1600</th>
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<td>Increase in height of specimen (mm)</td>
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<td>2.28</td>
<td>2.30</td>
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<td>1.59</td>
<td>1.35</td>
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<td>Height of voids (mm)</td>
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<td>4.95</td>
<td>4.95</td>
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<tr>
<td>Void ratio (e)</td>
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<td>1.96</td>
<td>1.59</td>
<td>1.35</td>
<td>1.17</td>
</tr>
</tbody>
</table>

* Deleted the inappropriate words.

+ Corrected where necessary for the computation of apparent.
### Determination of the One-Dimensional Consolidation Properties

**Job:** NZE 4099  
**Sample Weight:** 6651.4-10.60  
**Sample Moisture:** 6651.4-10.60  
**Sample Size:** 61.8-85  
**Sample Density:** 81.66%  
**Sample Depth:** 10-50 cm  
**Sample Location:** Checked by: P. Kelleher

**Test Details:**
- **Soil description:** Unstressed, decomposed, compacted, hydrated, unknown.
- **Loading cycle:** 7 days
- **Specimen No.:**
  - **Cell No.:** 2
  - **Diaphragm Diameter:** 26.20 mm
- **Machine No.:** 2
- **Height of Cell:** 26.20 mm
- **Ring No.:** 5
- **Area of Ring (A) = \( \pi \times (26.20 - 20.50)^2 \) mm²
- **Solid density of soil particle measured:** 2.70 g/cm³

<table>
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<th>Stage</th>
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<td>Mass of ring + wash glass + wet specimen</td>
<td>505.26 g</td>
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<td>Mass of ring + wash glass + dry specimen</td>
<td>( M_2 )</td>
<td>288.47 g</td>
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<td>Mass of ring</td>
<td>( M_4 )</td>
<td>97.0 g</td>
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<td>Mass of wash glass</td>
<td>( M_3 )</td>
<td>91.87 g</td>
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<td>Mass of dry specimen</td>
<td>( M_4 - M_3 - M_5 )</td>
<td>72.23 g</td>
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<tr>
<td>Mass of water</td>
<td>( \frac{M_2 - M_4}{M_4 - M_5} )</td>
<td>0.5763 g</td>
</tr>
<tr>
<td>Water content</td>
<td>( \frac{M_2 - M_4}{M_4 - M_5} )</td>
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<tr>
<td>Dry density ( \rho_d = \frac{M_4}{A} )</td>
<td>( \text{g/cm}^3 )</td>
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<td>Height of soil particles ( H = \frac{M_4}{\rho_d} )</td>
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<td>Void ratio ( e = \frac{H - H_0}{\rho_d} )</td>
<td>( e )</td>
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<td>Degree of saturation ( S = \frac{\rho_d}{\rho_s} )</td>
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<th>% Change at press. (H - H)</th>
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<td>( k_{\text{pa}} )</td>
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* Denotes the inappropriate words.
† Corrected where necessary for the compression of specimens.
**Determination of the One-Dimensional Consolidation Properties**

**Notes:**
- **Sample No.:** 1111-1112-B3
- **Test Method:** NZS 4402
- **Prepared by:** C. K. P. S.
- **Checked by:** J. G. K.
- **Date:** 2-5-85
- **Water table elevation:** 5.7 m
- **Test details:**
  - Soil description: Undisturbed, unvegetated lean sand, fine grained, medium to high density
  - Loading cycle: 24 hours/minute

<table>
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<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
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<tbody>
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<td>mm</td>
</tr>
<tr>
<td>Measured thickness of specimen (d)</td>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td>Mass of ring with glass + dry specimen</td>
<td>g</td>
<td>g</td>
</tr>
<tr>
<td>Mass of ring with glass + dry specimen</td>
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<td>Mass of ring</td>
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<td>Mass of dry specimen</td>
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<td>Mass of water</td>
<td>g</td>
<td>g</td>
</tr>
<tr>
<td>Water content</td>
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<td>Dry density</td>
<td>ρ_d</td>
<td>g/cm³</td>
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<td>Height of soil particles</td>
<td>h</td>
<td>mm</td>
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<tr>
<td>Voids ratio e = ( \frac{h_d}{h} )</td>
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<tr>
<td>Degree of saturation s = ( \frac{V_d}{V} )</td>
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</table>

* Denotes the discrepancy words.

**Graph:**
- Graph shows the relationship between applied pressure and voids ratio (e). The curve indicates an upward trend as pressure increases.
- Corrected values are necessary for the computation of apparatus.
Determination of the One-Dimensional Consolidation Properties

Job: NZC 0001, Site 1200
Sample No.: 6667-01-55
Sampling time: 08:00
Sampling location: 154.61
Sampling depth: 7.62
Ground surface elevation: 21.77
Water table elevation: 17.74

Test details:

- Soil description: Untreated/Impermeable/unconsolidated/uncompressed
- Loading cycle: 2kPa /hr /cyclic
- Specimen No:
  - Cell No.: 6
  - Diameter of ring (D): 92.2 mm
  - Height of ring (H): 10.56 mm
  - Area of ring (A) = \( \frac{\pi D^2}{4} \) = 6.66 m^2
  - Solid density of soil particles measured (\( \rho_s \)) = 2.63 g/cm^3

<table>
<thead>
<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured thickness of specimen (( D ))</td>
<td>mm</td>
<td>M_1</td>
</tr>
<tr>
<td>Mass of ring + watch glass + wet specimen</td>
<td>g</td>
<td>M_3</td>
</tr>
<tr>
<td>Mass of ring + watch glass + dry specimen</td>
<td>g</td>
<td>M_2</td>
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<td>g</td>
<td>M_3</td>
</tr>
<tr>
<td>Mass of watch glass</td>
<td>g</td>
<td>M_2</td>
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<tr>
<td>Mass of dry specimen</td>
<td>g</td>
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<td>Mass of water</td>
<td>g</td>
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<td>( \frac{M_5 - M_6}{M_5 - M_6} )</td>
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<td>( \frac{M_6}{V} )</td>
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<td>Height of self-penetration</td>
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<td>( \frac{M_6}{D} \times 1000 )</td>
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<td>Void ratio</td>
<td></td>
<td>( \frac{P}{\rho_s \cdot \frac{1}{D}} )</td>
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<tr>
<td>Degree of saturation</td>
<td></td>
<td>( \frac{P}{\rho_s \cdot \frac{1}{D}} )</td>
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Applied stress | Stress-strain relationship | Thickness of specimen | % change thickness | Height of void | Void ratio |
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<td>mm</td>
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* Delete the inappropriate words.
† Corrected when necessary for the correction of apparent.
### Table: Measured Thickness of Specimen (mm)

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<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured thickness of specimen (mm)</td>
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<td>15.6</td>
</tr>
<tr>
<td>Mass of ring + wash glass + wet specimen</td>
<td>4.20</td>
<td>4.20</td>
</tr>
<tr>
<td>Mass of ring + wash glass + dry specimen</td>
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<td>Mass of wash glass</td>
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<td>Mass of dry specimen</td>
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<tr>
<td>Mass of water</td>
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<td>Water content w</td>
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<td>4.6</td>
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<tr>
<td>Dry density ( \rho_d = \frac{M}{V} \text{ t/m}^3 )</td>
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<td>1.17</td>
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<td>Height of soil particles ( h = \frac{M}{\rho_d} \text{ mm} )</td>
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<tr>
<td>Void ratio ( e = \frac{H - h}{H} )</td>
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<tr>
<td>Degree of saturation ( S = \frac{H_1 - h_1}{H_1} )</td>
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<td>0.715</td>
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### Applied Stress (kPa)

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<th>2500</th>
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<td>0.04</td>
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</table>

* Denote the inappropriate words.

**Corrected water content for the comparison of apparatus.**
**DETERMINATION OF THE ONE-DIMENSIONAL CONSOLIDATION PROPERTIES**

**Lake/Unit:** M.R.L. HOSTI 12km 4.50E

**Sample No.:** 6651 - 235.50\* + 1

**Tracing:** M.R.L. HOSTI 4.50E

**Sampling date:** 9/1/54

**Sampling location:** M.R.L. 4451

**Date:** 9/6/54 - 2.7.54

**Sampling depth:** 22.50 m

**Connected by:** M.R.L. 4451

**Ground surface elevation:** 21.71 m

**Date:** 19.7.54

**Water table elevation:** 15.90 m

**Test details:**

- Soil description: Undisturbed (unconsolidated/uncompressed/untouched)
- Loading cycle: 24 hours
- Specimen no.: 1
- Cyl No.: 1
- Diameter of ring (D): 76.50 mm
- Height of ring (H): 19.57 mm
- Ring No.: 1/2
- Area of ring (A): 55.007 mm²
- Solid density of soil specimen: 1.80 g/cm³

<table>
<thead>
<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured thickness of specimen (D)</td>
<td>mm</td>
<td>H₁ = 12.890</td>
</tr>
<tr>
<td>Mass of ring + water glass + dry specimen</td>
<td>g</td>
<td>M₁ = 12.890</td>
</tr>
<tr>
<td>Mass of ring + water glass</td>
<td>g</td>
<td>M₁ = 12.890</td>
</tr>
<tr>
<td>Mass of ring</td>
<td>g</td>
<td>M₁ = 12.890</td>
</tr>
<tr>
<td>Mass of water</td>
<td>g</td>
<td>M₂ - M₁ = 6.781</td>
</tr>
<tr>
<td>Mass of water</td>
<td>g</td>
<td>M₂ - M₁ = 6.781</td>
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<tr>
<td>Water content</td>
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</tr>
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<tr>
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<tr>
<td>Degree of saturation</td>
<td>%</td>
<td>184.0</td>
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<th>Load and time of application of pressure</th>
<th>Incremental thickness of specimen (ΔH)</th>
<th>Height of soil specimen (H₀)</th>
<th>Height of excess water (Hₑ)</th>
<th>Voids ratio (k)</th>
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* Does not apply to the inappropriate words.

**Note:** Corrected values necessary for the description of apparatus.

---

**Image 0x0 to 843x595**
**FORM 21**

**DETERMINATION OF THE ONE-DIMENSIONAL CONSOLIDATION PROPERTIES**

<table>
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<th>Soil sample:</th>
<th>Sample No: 6551 - 27.85</th>
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<tbody>
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<td>Sampled from:</td>
<td>6.44 e6</td>
</tr>
<tr>
<td>Tested by:</td>
<td>D. Neely</td>
</tr>
<tr>
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<td>19.6.85</td>
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<tr>
<td>Sampling depth:</td>
<td>27.85 m</td>
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<tr>
<td>Ground surface elevation:</td>
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<tr>
<td>Water table elevation:</td>
<td>12.70 m</td>
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**Test details:**

- **Soil description:** Undisturbed fine sand
- **Loading cycle:** 24 hr
- **Specimen No:** 9
- **Cell No:** 4
- **Diameter of ring (D):** 76.15 mm
- **Height of ring (H):** 19.67 mm
- **Ring No:** 12 x 155.9 mm
- **Area of ring (A):** 12 x 155.9 mm
- **Solid density of soil particles measured (d):** 2.65 g/cm³

<table>
<thead>
<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured thickness of specimen (D)</td>
<td>mm</td>
<td>H</td>
</tr>
<tr>
<td>Mass of ring + wash glass + wet specimen</td>
<td>g</td>
<td>M₁</td>
</tr>
<tr>
<td>Mass of ring + wash glass + dry specimen</td>
<td>g</td>
<td>M₄</td>
</tr>
<tr>
<td>Mass of ring</td>
<td>g</td>
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<td>M₈</td>
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<td>H₀ x A</td>
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<td>Voids ratio</td>
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<td>n/N₀</td>
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<td>Degree of saturation</td>
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**Applied pressures**

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<th>Thickness of specimen</th>
<th>% change in thickness</th>
<th>Height of water</th>
<th>Void ratio</th>
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</table>

* Define the inappropriate words.
† Corrected where necessary for the compensation of appearance.
Determination of the one-dimensional consolidation properties

Job: N6S 4492
Sampling date: 8-17-87
Sampling location: 4492
Sampling depth: 31'-7.5"
Sample No.: 66 51-31.90

Test details:

Sample No.: 6
Diameter of ring (d) 76.2 mm
Height of ring (H) 12.5
Area of ring (A) = \( \pi \times d^2 \times 0.827 \) mm²
Solid density of soil particles measured/assumed (g)/cc 2.45 g/cc

<table>
<thead>
<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured thickness of specimen (d₁) mm</td>
<td>H₁ = 19.400</td>
<td>H₂ = 17.055</td>
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<td>Mass of ring + water glass + wet specimen</td>
<td>M₂ = 44.648</td>
<td>M₂ = 42.491</td>
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<tr>
<td>Mass of ring + water glass + dry specimen M₂</td>
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<tr>
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<td>M₁ = 91.159</td>
<td>M₁ = 89.079</td>
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<td>Mass of water</td>
<td>M₂ - M₁ = 49.682</td>
<td>M₂ - M₁ = 43.312</td>
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<td>Water content w</td>
<td>( \frac{M₂ - M₁}{M₁} \times 100 )</td>
<td>( \frac{M₂ - M₁}{M₁} \times 100 )</td>
</tr>
<tr>
<td>Dry density</td>
<td>( \frac{M₂}{H₂ \times A} ) g/cm³</td>
<td>( \frac{M₂}{H₂ \times A} ) g/cm³</td>
</tr>
<tr>
<td>Height of soil particles</td>
<td>( \frac{M₂}{H₂ \times A} ) mm</td>
<td>( \frac{H₂}{H₂ \times A} ) mm</td>
</tr>
<tr>
<td>Voids ratio e</td>
<td>( \frac{M₂}{H₂ \times A} )</td>
<td>( \frac{M₂}{H₂ \times A} )</td>
</tr>
<tr>
<td>Degree of saturation</td>
<td>( \frac{M₂}{H₂ \times A} )</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Applied pressure</th>
<th>Date and time of application of pressure</th>
<th>Incremental stress ( \Delta \sigma )</th>
<th>Thickness of specimen, d₁</th>
<th>Height of voids ( \frac{H₂ - H₁}{H₂ - H₁} )</th>
<th>Voids ratio e</th>
<th>Degree of saturation</th>
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* Delete the inappropriate words.
† Corrected where necessary for the compression of apparatus.
## Determination of the One-Dimensional Consolidation Properties

**Job:** 5285-3600, 3601

**Sample No(s):** 6651, 60-10, 44-214

**Sampling date:** 10-10-10

**Tested by:** P. [Signature]

**Sampling location:** 26-4

**Date:** 10-10-10

**Sampling depth:** 40.0, 10.0

**Ground surface elevation:** 10.0, 10.0

**Water table elevation:**

**Test details:**

- **Soil description:** Unfractured, unconsolidated, unweathered, unknown

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**Measured thickness of specimen (H)**

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**Mass of ring + wash glass + wet specimen**

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**Mass of ring**

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**Mass of water**

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**Water content**

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**Height of soil particles**

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**Application of pressure**

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<th>Final</th>
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**Table of results**

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**Graphical representation**

- **Graph showing stress-strain relationship**

- **Graph showing consolidation curve**

---

* Delete the inappropriate words.

1. Corrected when necessary for the compression of apparatus.
**FORM 1**

**DETERMINATION OF THE ONE-DIMENSIONAL CONSOLIDATION PROPERTIES**

- **Sample No.:** 6651 - 95.32
- **Tamped by:** M. Holsey
- **Date:** 24-6-61
- **Sample depth:** 45.75 mm
- **Ground surface elevation:** 15.7 m
- **Water table elevation:**

**Test details**

- **Sample No.:** 6651 - 95.32
- **Diameter of ring:** 76.19 mm
- **Height of ring:** 49.82 mm
- **Area of ring (A):** 465.77 mm²
- **Solid density of soil particles measured:** 2.64 g/cm³

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<th>Stage</th>
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<th>Final</th>
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<tbody>
<tr>
<td>Measured thickness of specimen (H) mm</td>
<td>H₁</td>
<td>H₂</td>
</tr>
<tr>
<td>Mass of ring + wax = wet specimen g</td>
<td>M₁</td>
<td>253.798</td>
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<td>Mass of ring + wax = dry specimen g</td>
<td>M₂</td>
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<td>Mass of ring g</td>
<td>M₃</td>
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<td>Mass of wax g</td>
<td>M₄</td>
<td>86.053</td>
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<tr>
<td>Mass of water g</td>
<td>M₅ = M₁ - M₃ - M₄</td>
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<td>Water content w</td>
<td>%</td>
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<tr>
<td>Dry density ρ_d = M₃ / H x A</td>
<td>g/cm³</td>
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<td>Height of soil particle H = M₅ / A cm</td>
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<tr>
<td>Void ratio e = H / H₀</td>
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<td>Degree of saturation S = e / H₀</td>
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<td>mm</td>
<td>mm</td>
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*Denote the inappropriate words.
† Corrected where necessary for the compression of graphs.
Determination of the One-Dimensional Consolidation Properties

John Nriel, Project: Subasence
Sample No.: 665?4265-g766a (conventional)

Sampling date: 1-1-55
Tested by: P. H. E. (1)
Sampling location: CHUETU
Date: 6-6-55-4-55
Sampling depth: 5-64m
Ground surface elevation: 32-0-1

Visual sample elevation:

Test details:

Soil description: Undisturbed sample
Loading cycle: 24 hours/masses

Specimen No.
Cell No. 2
Diameter of ring (D): 76.1 mm
Height of ring (H): 20 mm
Ring No. 2
Area of ring (A): $\frac{\pi D^2}{4} = 5033.7$ mm²

Solid density of soil particle: measured/assumed (g) 2.65 gm³

<table>
<thead>
<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
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<tbody>
<tr>
<td>Measured thickness of specimen (H)</td>
<td>mm</td>
<td>H₁ 19.157</td>
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<tr>
<td>Mass of ring + wash glass + test specimen</td>
<td>g</td>
<td>M₂ 255.402</td>
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<tr>
<td>Mass of ring</td>
<td>g</td>
<td>M₁ = 99.679</td>
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<tr>
<td>Mass of wash glass</td>
<td>g</td>
<td>M₃ = 57.756</td>
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<tr>
<td>Mass of dry specimen</td>
<td>g</td>
<td>M₁ = 141.921</td>
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<tr>
<td>Mass of water</td>
<td>g</td>
<td>M₂ = M₂ - M₁ = 154.487</td>
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<td>%</td>
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<td>g/cm³</td>
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</tr>
</tbody>
</table>

* Delete the inappropriate words.
† Correct where necessary for the compression of apparatus.
Determination of the One-Dimensional Consolidation Properties

Job: NZEO Hotel Subsurface Sample No.: 6651-53.32
Sampling unit: B - 3.45
 Tested by: A. R. Kaye
Sampling block: 6651
Date: 29-8-85
Sampling depth: 56.92 m
Checked by: P. K. M. W. 85
Consolidation elevation: Date: 13-3-85
Wet-soluble elevation:

Test details:

- Soil description: Undisturbed, granular, medium sand
- Loading cycle: 2.4 kN/m²
- Specimen No.: 6651-53.32
- Cell No.: 5
- Diameter of ring (D): 76.08 mm
- Height of ring (H): 17.08 mm
- Ring No.: 15
- Area of ring (A): 4.464 mm²
- Solid density of soil particles measured/assumed (ρ): 2.43 g/cm³

<table>
<thead>
<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured thickness of specimen (H₀)</td>
<td>mm</td>
<td>M₁</td>
</tr>
<tr>
<td>Mass of ring + wash glass + wet specimen</td>
<td>Kg</td>
<td>289.150</td>
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<tr>
<td>Mass of ring + wash glass + dry specimen</td>
<td>Kg</td>
<td>179.357</td>
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<td>Mass of ring</td>
<td>Kg</td>
<td>83.301</td>
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<td>Mass of wash glass</td>
<td>Kg</td>
<td>186.616</td>
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<td>Mass of dry specimen</td>
<td>Kg</td>
<td>58.918</td>
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<td>Mass of water</td>
<td>Kg</td>
<td>M₂-M₄</td>
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<tr>
<td>Water content</td>
<td>%</td>
<td>m₁</td>
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<tr>
<td>Dry density ρ₀ = ρ</td>
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<td>Height of soil particles (H₀) = M₂/A</td>
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<td>Voids ratio e = H₀/H₁</td>
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<td>Degree of saturation S = (H₀-H)/H₀</td>
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Applied pressure

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<tr>
<th>Applied</th>
<th>Date of application of</th>
<th>Incremental</th>
<th>Thickness of specimen, H</th>
<th>% Throat thickness</th>
<th>Height of</th>
<th>Voids ratio</th>
</tr>
</thead>
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<tr>
<td>kN/m²</td>
<td>of application of</td>
<td>deflection</td>
<td>(H₀ - Hₙ)</td>
<td>(H₀ - Hₙ)/H₀</td>
<td>value (H₀ - Hₙ)</td>
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</tr>
<tr>
<td>0.05</td>
<td>mm</td>
<td>mm</td>
<td>%</td>
<td>mm</td>
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<tr>
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<td>0.625</td>
<td>10.213</td>
<td>1.041</td>
<td>5.621</td>
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</tbody>
</table>

* Denote the inappropriate words.
† Corrected where necessary for the compensation of properties.
Determination of the one-dimensional consolidation properties

Sample No: (6451 - 25/69)
Sampled by: "...
Sampled at: 1.45 - 154.65'
Sampled depth: 59 - 205'
Ground surface elevation: Date: 12.10.1981
Water table elevation:
Test details:

- Soil description: Undisturbed (nonaggrigated)
- Laboratory testing:
  - Loading cycle: 30 days
  - Specimen No: 6451 - 59/69
  - Cell No: 6
  - Diameter of cell (D) = 76.40 mm
  - Height of cell = 150.65 mm
  - Ring No: q = 1/4
  - Area of ring (A) = 4.673.99 mm²
  - Solid density of soil particles measured (g/m³) 20.55 g/m³

<table>
<thead>
<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured thickness of specimen (W)</td>
<td>mm</td>
<td>M_1 = 10.49 M_2 = 10.67</td>
</tr>
<tr>
<td>Mass of ring + wash glass + wet specimen</td>
<td>M'</td>
<td>578.126 M_1 = 246.000</td>
</tr>
<tr>
<td>Mass of ring + wash glass + dry specimen</td>
<td>M_2</td>
<td>262.323</td>
</tr>
<tr>
<td>Mass of ring</td>
<td>M_3</td>
<td>27.487</td>
</tr>
<tr>
<td>Mass of wash glass</td>
<td>M_4</td>
<td>38.123</td>
</tr>
<tr>
<td>Mass of dry specimen</td>
<td>M_0 = M_1 - M_2</td>
<td>77.956</td>
</tr>
<tr>
<td>Mass of water</td>
<td>M_1 = M_0 - M_4</td>
<td>59.454</td>
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<tr>
<td>Water content w</td>
<td>%</td>
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<td>Dry density ρ_d = M_0 / A</td>
<td>g/cm³</td>
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<tr>
<td>Height of soil particles H = 1000</td>
<td>mm</td>
<td>74.4</td>
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<tr>
<td>Voids ratio e = V / V_s</td>
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<td>1.853</td>
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<tr>
<td>Degree of saturation S = (1 - e)</td>
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<td>0.77</td>
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- Applied pressure: Data and unit of application of applied pressures
- Incremental deformation: Thickness of specimen (M_1 - M_2)
- Incremental deformation: Thickness of specimen (M_1 - M_2)
- Height of voids (V - V_s)
- Voids ratio (V / V_s)

<table>
<thead>
<tr>
<th>Applied pressure (kPa)</th>
<th>M_1</th>
<th>M_2</th>
<th>M_3</th>
<th>M_4</th>
<th>M_5</th>
<th>M_0</th>
<th>Mass of dry specimen (g)</th>
<th>Voids ratio (V / V_s)</th>
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<td>1.20</td>
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</tbody>
</table>

* Delete the inappropriate words.
† Corrected where necessary for the compression of apparatus.
Determination of the One-Dimensional Consolidation Properties

Date: 11/21/85
Sample No.: 653-10-85
Tested by: P. K. Taylor
Sampling location: 653-10-85
Date: 10/6/85 - 10/7/85
Sampling depth: 40-47
Ground surface elevation: 24.03'
Water table elevation: 24.03'

Test details:
- Undisturbed/unconsolidated/untensioned/unsheared
- Loading cycle: 24 hours/cycle

<table>
<thead>
<tr>
<th>Specimen No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
</tr>
</tbody>
</table>

| Cell No. 3 | Diameter of ring (D) 75.20 mm |
| Machine No. 3 | Height of ring 15.25 mm |
| Ring No. 2 | Area of ring (A) = \( \pi \times (0.375)^2 \) mm² |

Solid density of soil particles measured/calculated (G) 3.15 g/cm³

<table>
<thead>
<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured thickness of specimen (D) mm</td>
<td>H₀</td>
<td>H₁</td>
</tr>
<tr>
<td>Mass of ring + wash glass + wet specimen</td>
<td>M₁</td>
<td>212.946</td>
</tr>
<tr>
<td>Mass of ring + wash glass + dry specimen</td>
<td>M₂</td>
<td>212.946</td>
</tr>
<tr>
<td>Mass of ring</td>
<td>M₃</td>
<td>59.198</td>
</tr>
<tr>
<td>Mass of wash glass</td>
<td>M₄</td>
<td>35.349</td>
</tr>
<tr>
<td>Mass of dry specimen</td>
<td>M₅ = M₆ - M₃ - M₄</td>
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</tr>
<tr>
<td>Mass of water</td>
<td>M₆</td>
<td>204.474</td>
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<tr>
<td>Water content</td>
<td>w</td>
<td>62.7</td>
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<tr>
<td>Dry density</td>
<td>ρd = M₆ / (A x D)</td>
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</tr>
<tr>
<td>Height of self-particle</td>
<td>H₀ x 1000 / (A x D)</td>
<td></td>
</tr>
<tr>
<td>Voids ratio</td>
<td>e = H₀ / H₀</td>
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</tr>
<tr>
<td>Degree of saturation</td>
<td>S = ρ / ρ₀</td>
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<table>
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<tr>
<th>Applied pressure (pₚ)</th>
<th>25</th>
<th>50</th>
<th>100</th>
<th>200</th>
<th>400</th>
<th>900</th>
<th>1400</th>
<th>3500</th>
<th>5600</th>
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<td>Voids ratio</td>
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<td>0.057</td>
<td>0.063</td>
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<td>Degree of saturation</td>
<td>1.848</td>
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<td>1.69</td>
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<td>1.26</td>
<td>1.01</td>
<td>1.06</td>
<td>1.17</td>
<td>1.35</td>
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</tbody>
</table>

* Delete the inappropriate words.
† Converted where necessary for the computation of apparatus.
**Determination of the One-Dimensional Consolidation Properties**

**John: NZ62**  
**H.R. 11**  
**Sample No.: 6252-3**  
**Sample Size: 21.1 cm**

**Sampling Date:**  
21.1 1981

**Soil Description:**  
Undisturbed soil sample  
Undisturbed soil sample

**Leading Cycle:** 24 hours/1000

**Test Details:**

- **Soil description:** Undisturbed soil sample
- **Leading cycle:** 24 hours/1000
- **Specimen No.:**
- **Cell No.:** 5
- **Diameter of ring (D):** 76.20 mm
- **Height of ring (H):** 76.34 mm
- **Ring No.:** 5
- **Width of ring (D):** 76.37 mm
- **Height of dry specimen:** 29.7
- **Height of wet specimen:** 29.7
- **Height of wet specimen:** 29.7
- **Water content:** 29.7
- **Dry density:** $\rho_d = \frac{\rho_w}{\rho_s}$
- **Height of soil particles:** $H = \frac{1}{\rho_s}$
- **Void ratio:** $e = \frac{\rho_s}{\rho_w}$
- **Degree of saturation:** $S = \frac{\rho_w}{\rho_d}$

**Applied Pressure:**

<table>
<thead>
<tr>
<th>Applied Pressure</th>
<th>Date and time</th>
<th>Incremental deformation $\Delta h$</th>
<th>Thickness of specimens (D)</th>
<th>$%$ change of $\frac{L}{D}$</th>
<th>Height of voids (D - $\Delta h$)</th>
<th>Void ratio $\frac{\rho_d}{\rho_w}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>kPa</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
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<td>0.012</td>
<td>0.012</td>
<td>0.012</td>
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<tr>
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<td>0.012</td>
<td>0.012</td>
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<td>0.012</td>
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<td>0.012</td>
<td>0.012</td>
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<td>1600</td>
<td>0.027</td>
<td>0.012</td>
<td>0.012</td>
<td>0.012</td>
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<td>0.012</td>
<td>0.012</td>
<td>0.012</td>
<td>0.012</td>
<td>0.012</td>
</tr>
</tbody>
</table>

* Denote the inappropriate values

**Calculated Data for the Description of Apparatus:**
Determination of the One-Dimensional Consolidation Properties

Sample Number: 6653-16-05
Sample Date: 06-24-1981
Sample Location: 55146 12.5
Sample Depth: 16.05 m
Ground surface elevation: Date:
Water table elevation:

Test details:
- Soil description: Undisturbed
- Undisturbed sample contained:
- Undisturbed sample contained:
- Loading cycle: 24 hours
- Specimen No.:
- Cell No.: 5
- Diameter of ring (D) 75.06 mm
- Machine No.: 5
- Height of ring 17.93 mm
- Ring No.: 11
- Area of ring (A) = 4.0425 cm
- Solid density of soil particles measured: 2.77 g/cm³

<table>
<thead>
<tr>
<th>Stage</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured thickness of specimen (D)</td>
<td>17.860 mm</td>
<td>17.724 mm</td>
</tr>
<tr>
<td>Mass of ring + wash glass + wet specimen</td>
<td>17.591 g</td>
<td>17.586 g</td>
</tr>
<tr>
<td>Mass of ring + wash glass + dry specimen</td>
<td>17.591 g</td>
<td>17.586 g</td>
</tr>
<tr>
<td>Mass of ring</td>
<td>17.492 g</td>
<td>17.492 g</td>
</tr>
<tr>
<td>Mass of wash glass</td>
<td>17.492 g</td>
<td>17.492 g</td>
</tr>
<tr>
<td>Mass of dry specimen</td>
<td>17.492 g</td>
<td>17.492 g</td>
</tr>
<tr>
<td>Mass of water</td>
<td>17.492 g</td>
<td>17.492 g</td>
</tr>
<tr>
<td>Water content w</td>
<td>1.4%</td>
<td>1.4%</td>
</tr>
<tr>
<td>Dry density ρ_d = M_d / D x A</td>
<td>1.055 g/cm³</td>
<td>1.052 g/cm³</td>
</tr>
<tr>
<td>Height of soil particles H_s = M_s / ρ_s x A</td>
<td>7.915 mm</td>
<td>7.915 mm</td>
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<tr>
<td>Void ratio e = V_v / V_s</td>
<td>1.257</td>
<td>1.257</td>
</tr>
<tr>
<td>Degree of saturation s = V_s / V_v</td>
<td>97.3%</td>
<td>100%</td>
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<table>
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<th>Applied pressure (kPa)</th>
<th>Sand and water present</th>
<th>Incremental change</th>
<th>Thickness of specimen (D)</th>
<th>% change</th>
<th>Thickness of specimen (D)</th>
<th>Height of soil particles (H_s)</th>
<th>Void ratio (e)</th>
<th>Degree of saturation (S)</th>
</tr>
</thead>
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<td>17.721</td>
<td>2.045</td>
<td>1.046</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>50</td>
<td>0.045</td>
<td>17.721</td>
<td>1.2451</td>
<td>1.2451</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>0.045</td>
<td>17.721</td>
<td>1.372</td>
<td>1.372</td>
<td></td>
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<tr>
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<td>0.045</td>
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<td>1.372</td>
<td>1.372</td>
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<tr>
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<td>1.372</td>
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<td>1.372</td>
<td>1.372</td>
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</table>

* Denotes the inappropriate word.
* Corrected where necessary for the compaction of specimen.
APPENDIX 6: Back-analysis of ultimate settlement and associated piezometric head drop

Figure A 6.1: Analysis boundary conditions

sandy silt
\( \rho = 1.56 \, \text{t/m}^3 \)

silty sand
\( \rho_{\text{sat}} = 1.68 \, \text{t/m}^3 \)

fine sand
\( \rho_{\text{sat}} = 1.50 \, \text{t/m}^3 \)

clayey silt
\( \rho_{\text{sat}} = 1.90 \, \text{t/m}^3 \)

sand
\( \rho_{\text{sat}} = 1.55 \, \text{t/m}^3 \)

consolidating layers

Layer 1
ignimbritic sandy silt
\( \rho_{\text{sat}} = 1.48 \, \text{t/m}^3 \)
(LOWER IGNIMBRITIC SILT UNIT)

Layer 2
ignimbritic silty sand
\( \rho_{\text{sat}} = 1.57 \, \text{t/m}^3 \)
Figure A 6.2: e-log σ plot for 6651 - 55.28m (representative of layer 1).

Figure A 6.3: e-log σ plot for 6651 - 58.89m (representative of layer 2).
1.0 Assumptions

- layers 1 and 2 (Figure A6.1) were normally consolidated prior to the hostel subsidence
- the e-log $\delta$ plots for BH6651 - 55.28m and 6651 - 58.89m (Figures A6.2 and A6.3) are representative of layers 1 and 2
- consolidation is restricted to layers 1 and 2 (adjacent materials are relatively incompressible) and is one-dimensional
- maximum past $\delta$ is defined by $\delta_p$ as measured by oedometer tests for each layer
- maximum past $\sigma$ is due to dewatering.

Note: The subscripts 'o' and 'c' represent initial and consolidated conditions respectively.

\[ \delta_p = \text{preconsolidation pressure} \]

2.0 Calculations

2.1 For layer 1 (ignimbritic silt)

i) Initial effective stress at mid-height of layer 1

\[ \delta_0 = 764.79 \text{kPa} + 5\text{m.}1.48\text{tm}^{-3} \cdot 9.81\text{ns}^{-2} - 395.34 \text{kPa} = 442.04 \text{kPa} \]

ii) Initial void ratio at mid-height of layer 1

from e-log $\delta$ curve for 6651 - 55.28m (Figure A6.2) and boundary conditions

\[ \delta_c \ (= \delta_p) = 605 \text{ kPa} \rightarrow e_c = 2.512 \]

\[ C_c = 1.631 \quad H_c = 10.0 \text{m} \]

since $e_c = e_0 - C_c \log \left( \frac{\delta_c}{\delta_0} \right) \rightarrow e_0 = e_c + C_c \log \left( \frac{\delta_c}{\delta_0} \right)$

\[ \rightarrow e_0 = 2.734 \]
iii) Original height of layer 1

\[ H_C = H_0 \left( \frac{1 + e_C}{1 + e_0} \right) \Rightarrow H_0 = \frac{H_C}{\left( \frac{1 + e_C}{1 + e_0} \right)} \]

\[ \Rightarrow H_0 = 10.632 \text{m} \]

for layer 1, 632mm of calculated settlement from \( \overline{\sigma}_0 \) to \( \overline{\sigma}_p \)

[Check of \( \overline{\sigma}_0 \)]

\( \overline{\sigma}_0 \) is calculated in (i) on the basis of 'consolidated' conditions. Since \( \rho_{\text{sat}_0} < \rho_{\text{sat}_C} \), we need to consider \( \overline{\sigma}_0 \) on the basis of calculated \( e_0 \).

For \( e_0 = 2.734 \)

\[ \rho_{\text{sat}} = 1.45 \text{ t.m}^{-3} \text{ (from oedometer test data)} \]

\[ \Rightarrow \overline{\sigma}_0 = 441.98 \text{ kPa} \]

which is not significant over previously calculated \( \overline{\sigma}_0 = 442.04 \text{ kPa} \)

iv) Find \( \Delta h_p \), the drop of piezometric head

assuming no change of \( \sigma \),

\[ \Delta \mu = \overline{\sigma}_C - \overline{\sigma}_0 = 605 \text{ kPa} - 442.04 \text{ kPa} = 162.96 \text{ kPa} \]

\[ \mu_C = \mu_0 - \Delta \mu = 398.48 \text{ kPa} - 162.96 \text{ kPa} = 235.52 \text{ kPa} \]

\[ h_C = \frac{\mu_C}{\rho_w g} = 24.01 \text{m} \]

\[ \Delta h_p = h_0 - h_C = 40.62 \text{m} - 24.01 \text{m} = 16.61 \text{m} \]
to get 632mm of settlement from consolidation of layer 1, 
the piezometric head at the mid-height of the layer has 
to drop 16.61m.

2.2 For layer 2 (ignimbritic silty sand)

i) Initial effective stress at mid-height of layer 2

\[
\bar{\sigma}_0 = 909.98 \text{kPa} + 1.63 \text{m} \cdot 1.57 \text{tm}^{-3} \cdot 9.81 \text{ms}^{-2} - 460.38 \text{kPa} = 474.70 \text{kPa}
\]

ii) Initial void ratio at mid height of layer 2

from e-log \( \bar{\sigma} \) curve for 6651 - 58.89m (Figure A6.3)

and boundary conditions

\[
\bar{\sigma}_c = (\bar{\sigma}_p) = 795 \text{kPa} \quad e_c = 1.708
\]

\[
C_c = 0.973 \quad C_c = 3.25m
\]

from 2.1 - ii) \( e_0 = 1.708 + 0.973 \log\left(\frac{795 \text{ kPa}}{474.70 \text{ kPa}}\right) - 1.926
\]

iii) Original height of layer 2

from 2.1 - iii) \( H_0 = \frac{3.25m}{(1 + 1.708)} = 3.512 \)

for layer 2, 262mm of calculated settlement from \( \bar{\sigma}_0 \) to \( \bar{\sigma}_p \).

iv) Find \( \Delta h_p \), the drop of piezometric head

assuming no change of \( \sigma \),

\[
\Delta \mu = \bar{\sigma}_c - \bar{\sigma}_0 = 795 \text{ kPa} - 474.70 \text{ kPa} = 320.30 \text{ kPa}
\]

\[
\mu_c = \mu_0 - \Delta \mu = 461.66 \text{ kPa} - 320.30 \text{ kPa} = 141.36 \text{ kPa}
\]
\[ h_c = \frac{\mu_c}{\rho_w g} = 14.41 \text{m} \]

\[ \Delta h_p = h_0 - h_c = 47.06 \text{m} - 14.41 \text{m} = 32.65 \text{m} \]

to get 262mm of settlement from consolidation of layer 2, the piezometric head at the mid-height of the layer has to drop 14.41m.

The calculated ultimate settlement due to consolidation of the ignimbritic silt and underlying silty sand is 894mm.
APPENDIX 7: Theoretical basis and computer program for finite difference settlement-time analysis.

1.0 Introduction

The theoretical basis for this analysis was developed by Drs R O Davis and B W Hunt of the Civil Engineering Department, University of Canterbury. This study used the theoretical basis to develop DECONS, a Basic computer program for the finite difference solution.

2.0 Theoretical basis

2.1 Preliminary relationships

Consider a saturated soil layer being drained from its base

i) Darcy's Law: \( V = -k \frac{\partial h}{\partial z} \)

where \( V \) = superficial velocity (which is not the velocity of the free surface)

\( h \) = total head

ii) Continuity Equation: \( \frac{\partial e}{\partial t} = -(1 + e_0) \frac{\partial V}{\partial z} \)

(from Darcy's Law) \( = k (1 + e_0) \frac{\partial^2 h}{\partial z^2} \) \( \ldots \ldots (1) \)

iii) Constitutive Relationship

The constitutive relationship relates \( \bar{\sigma} \) to \( e \). In this case the coefficient of compressibility \( (a_v) \) is used

\[ a_v = -\frac{de}{d\bar{\sigma}} \]

For finite difference development

\[ e - e_0 = a_v (\bar{\sigma}_0 - \bar{\sigma}) \] is assumed
iv) The Effective Stress Principal

\[ \bar{\sigma} = \sigma - \mu \]

here \( \sigma = \int_0^z \rho g \, dz \) which is not constant because \( \rho \) changes.

Therefore \( \frac{\partial \bar{\sigma}}{\partial t} = \frac{\partial \sigma}{\partial t} - \frac{\partial \mu}{\partial t} \)

\[ \rightarrow \frac{\partial e}{\partial t} = -a \sqrt{\frac{\partial \sigma}{\partial t} - \frac{\partial \mu}{\partial t}} \] \hspace{1cm} (2)

For \( \frac{\partial \sigma}{\partial t} \), \( \sigma (t_0) = \rho_d g z_w + \rho_{sat} g (z-z_w) \)

for \( t_0 = 0 \)

\[ \sigma (t_0 + \Delta t) = \rho_d g (z + dz_w \cdot \frac{\Delta t}{dt}) + \rho_{sat} g (z-z_w - dz_w \cdot \frac{\Delta t}{dt}) \]

where \( \Delta t \) = finite time step

\[ \sigma (t_0) + \frac{dz_w}{dt} \cdot \frac{\rho_d g - \rho_{sat} g}{dt} \]

\[ = \sigma (t_0) - \frac{\rho_w g e}{1+e} \cdot \frac{dz_w}{dt} \]

\[ \rightarrow \frac{d\sigma}{dt} = \frac{\sigma(t_0 + \Delta t) - \sigma(t_0)}{\Delta t} = -\frac{\rho_w g e}{1+e} \cdot \frac{dz_w}{dt} \]

Now \( \frac{dz_w}{dt} \) = velocity of free surface = \( \frac{V}{n} = \frac{V}{e/(1+e)} \)

Hence \( \frac{d\sigma}{dt} = -\rho_w g V \) evaluated at \( z = z_w \) \hspace{1cm} (3)

So if the superficial velocity of the free surface is known, we can find \( \frac{\partial \sigma}{\partial t} \)
- For $\frac{\partial u}{\partial t}$, for any $z > z_w$ we have $h = h_e + h_p$

where $h_e$ = elevation head

$h_p$ = pressure head

$= h_e + \frac{\mu}{\rho_w g}$

$\Rightarrow \mu = \rho_w g (h - h_e)$

$\Rightarrow \frac{\partial u}{\partial t} = \rho_w g \frac{\partial h}{\partial t}$ ...... (4)

v) Deriving an expression for change of total head with time $\frac{\partial h}{\partial t}$

Equation (2), $\frac{\partial e}{\partial t} = -a_v \left( \frac{\partial a}{\partial t} - \frac{\partial u}{\partial t} \right)$

$= -a_v (-\rho_w g V - \rho_w g \frac{\partial h}{\partial t})$ from equations 3 and 4

$= \rho_w g V \left( \frac{\partial h}{\partial t} + V \right)$

And using this in equation (1) gives

$\frac{\partial h + V}{\partial t} = \frac{k (1 + e)}{\rho_w g a_v} \frac{\partial^2 h}{\partial z^2}$ ...... (5)

Initial conditions within the layer $h = h^o = constant$ everywhere.

Boundary conditions are $h = 0$ at bottom (which is fixed)

and $h = h_e$ at top (which moves with velocity $V/n$)

Note: The $V$ term (equation 5) enters because we assume the density changes from $\rho_{sat}$ to $\rho_d$ as the water table falls.

In fact, the density change will probably not be so dramatic because of capillary water retained in soil pores. Because of the capillary effect $\frac{\partial \theta}{\partial t}$ is assumed to be small and the $V$ term is omitted.
2.2 Derivation of finite difference representation

i) Finite difference mesh:

Consider a saturated soil profile, divided into zones (finite elements) numbered from the base:

- At any time $t$, the uppermost zone to be considered will be the last complete zone before the water table is reached.

- If during some time increment $\Delta t$ the water table moves into a new zone, then that zone will no longer be used for the remainder of the solution.

- Assume that $a_v = \text{constant}$.

ii) Conservation of pore fluid mass.

- Consider $j$, a typical zone:

Let $\Delta z_j = \text{zone height}$

$V_j = \text{pore fluid velocity}$

(downward velocities positive)

$e_j = \text{void ratio at zone mid-height}$

$A = \text{zone sectional area}$ (constant)

- Zone volume, $\bar{V} = A \Delta z_j$

but $\bar{V} = \bar{V}_s + \bar{V}_v$ where $\bar{V}_s = \text{volume of solids}$ $\bar{V}_v = \text{volume of voids}$

$\bar{V}_v = \bar{V}_s (1 + e_j)$

$\Rightarrow A \Delta z_j = \bar{V}_s (1 + e_j)$
- since soil solids are incompressible, \( \frac{v}{v} \) is a constant and not a function of time.

- therefore \[ \frac{\Delta z_j}{1+e_j} \]

so \[ \frac{\Delta z_j}{1+e_j} = \frac{\Delta z_j^0}{1+e_j} \]

where \( \Delta z_j^0 \) and \( e_j^0 \) are initial values at time \( t = 0 \)

- Mass of pore fluid = \( \rho_w \frac{\Delta s_j A e_j}{1+e_j} \) where \( \rho_w \) = density of water

the conservation of pore fluid mass requires that change of mass = mass flowing in top - mass flowing out of base

\[ \frac{\partial}{\partial t} \frac{\rho_w \Delta z_j A e_j}{1+e_j} = \rho_w V_{j+1} A - \rho_w V_j A \]

(this time derivative has a negative value since pore fluid mass decreases with time).

since \( \rho_w A \) = constant

\[ + \frac{\partial}{\partial t} \frac{\Delta z_j e_j}{1+e_j} = V_{j+1} - V_j \]

\[ + \frac{\partial e_j}{\partial t} = \frac{1+e_j^0}{\Delta z_j^0} V_{j+1} - V_j \]

The Mass Balance Equation

\[ \text{...... (6)} \]

iii) Darcy's Law

- Darcy's Law, \( V = -k \frac{\partial h}{\partial z} \) where \( h \) = total head in direction of flow

\( (h = h_e + h_d) \)

- For finite difference solution

- define \( h_j \) at the centre of the zone

- let \( h \) represent the total head at line \( j \)

- \( k_j, k_{j-1} \) = zone permeabilities
Then \[ V_j = \frac{k_j \cdot h_j - \bar{h}}{\frac{1}{2} \Delta z_j} = \frac{k_{j-1} \cdot \bar{h} - h_{j-1}}{\frac{1}{2} \Delta z_{j-1}} \]
since velocity above \( j \) = velocity below \( j \)

Combine to eliminate \( \bar{h} \)

\[
\rightarrow \quad V_j = \frac{2k_{j-1} k_j (h_j - h_{j-1})}{\Delta z_{j-1} k_j + \Delta z_j k_{j-1}} \quad \text{(equation 7)}
\]

iv) Constitutive relationship

\[
\frac{\partial e}{\partial t} = -a v \frac{\partial \sigma}{\partial t} = -a v \left( \frac{\partial \sigma}{\partial t} - \frac{\partial \mu}{\partial t} \right) \quad \text{(equation 2)}
\]
since \( \frac{\partial \sigma}{\partial t} = 0 \) is assumed, \( \text{(section 2.1 - v)} \)

\[
\rightarrow \quad \frac{\partial e_j}{\partial t} = \rho w g a v \frac{\partial h_j}{\partial t} \quad \text{(from equation 4)}
\]

\[
\rightarrow \quad \frac{\partial e_j}{\partial t} = \rho w g a v \frac{\partial h_j}{\partial t} \quad \text{......... (8)}
\]

v) Zone height

\[
\Delta z_j = (1 + e_j) \frac{\Delta z_j^*}{1 + e_j} \quad \text{......... (9)}
\]

vi) Initial solution conditions, \( t = 0 \)

\[
\Delta z_j^* \quad \text{specified everywhere}
\]
\[ e_j \quad " \quad " \]
\[ V_j = 0 \quad " \quad " \]
\[ h_j = H_0 \quad " \quad " \]

\[ l = \frac{1}{2} \Delta z_{k-1} \] where \( l \) is the distance to the free surface above the last complete zone.
vii) Boundary conditions

- Bottom boundary

for all \( t > 0 \)

\[
V_1 = \frac{k_1 h_1}{\frac{1}{2} \Delta Z_1} \quad (\text{from Darcy's Law}) \quad \ldots \ldots \quad (10)
\]

- Upper boundary

\[
h_e = 1 + \sum_{j=1}^{K-1} \Delta Z_j \quad \ldots \ldots \quad (11)
\]

then Darcy's Law gives

\[
V_K = \frac{k_{K-1} k_K (h_e - h_{K-1})}{\frac{1}{2} \Delta Z_{K-1} k_K + 1} \quad \ldots \ldots \quad (12)
\]

the velocity of the free surface is then given by

\[
\frac{dl}{dt} = -\frac{V_K (1+e_e)}{e_K} \quad \ldots \ldots \quad (13)
\]

(this is the "pore velocity", \( V_{/n} \))
3.0 Computer Program

3.1 Program approach

The logic steps followed by DECONSL are as follows:

Let \( \hat{e} \) mean evaluated at \( t + \Delta t \)

**Steps**

\[
\hat{e} = 1 - V_k \left( 1 + e_k \right) \Delta t \quad \quad \quad \quad \text{(equ 13)}
\]

0:  
check that \( \hat{e} > 0 \). If \( \hat{e} < 0 \), then set \( K = K - 1 \)

and \( \hat{e} = \Delta Z^e_k + \hat{e} \)

1:  
\[
\hat{e} = e_j + (1 + e_j^o) \left( V_{j+1} - V_j \right) \frac{\Delta t}{\Delta Z_j^o} \quad \quad \quad \quad \text{(equ 6)}
\]

2:  
\[
\Delta Z_j = (1 + e_j^o) \frac{\Delta Z_j^o}{1 + e_j} \quad \quad \quad \quad \text{(equ 9)}
\]

\[
\text{note: } \frac{\Delta Z_j^o}{1 + e_j^o} = \frac{\Delta Z_j}{1 + e_j} \quad \text{used in DECONSL}
\]

3:  
\[
\hat{h}_j = h_j + \hat{e} - e_j \quad \quad \quad \quad \text{(equ 8)}
\]

4:  
\[
\hat{V}_j = \frac{2k_j - k_j^o}{\Delta Z_j - k_j^o} \left( \hat{h}_j^o - h_{j-1} \right) \quad \quad \quad \quad \text{(equ 7)}
\]

for \( 1 \leq j \leq K - 1 \)

At the lower boundary:  
\[
\hat{V}_1 = \frac{k_1 \hat{h}_1}{\frac{1}{2} \Delta Z_1} \quad \quad \quad \quad \text{(equ 10)}
\]

5:  
At the upper boundary:  
\[
\hat{h}_e = \hat{e} + \sum_{j=1}^{K-1} \Delta Z_j \quad \quad \quad \quad \text{(equ 11)}
\]

\[
\hat{V}_k = k_k - k_k^o \left( \hat{h}_e - \hat{h}_{k-1} \right) \quad \quad \quad \quad \text{(equ 12)}
\]

\[
\frac{1}{2} \Delta Z_k - k_k^o \hat{k}_e + \hat{k}_{k-1}
\]

Now everything updated at \( t + \Delta t \)
3.2 Numerical stability

The explicit finite difference technique used, because of possible numerical instability requires certain restrictions on input parameters.

Criteria used for numerical stability is:

\[ \frac{C_v \Delta t}{\Delta z^2} < 0.5 \]

Theoretical values of \( C_v \) are evaluated from the input data and numerical stability is tested by DECONSL for each layer.
10 PRINT "DECONE1 is a Basic Program representing one-dimensional"
20 PRINT "consolidation numerically. The program calculates settlement"
30 PRINT "in the floor of a building. Stresses are introduced gradually in"
40 PRINT "the form of effective stress. The program can be used for"
50 PRINT "design and analysis of floor floor consolidation problems."
60 PRINT "The number of layers can be increased by changing the"
70 PRINT "values of the parameters in the program."
80 PRINT "The program takes input data from the keyboard and"
90 PRINT "calculates the settlement of the floor."
400 NEXT L
410 PRINT\PRINT
415 PRINT "Do you want to continue (Y/N)?"; INPUT A$  
420 IF A$="Y" THEN 425
430 G0 TO 130
435 PRINT\PRINT
436 PRINT "$1%","Initial conditions at t=0   
437 PRINT "$1%","==================================")
438 PRINT "$1%
440 REM **********************
442 REM * INITIAL CONDITIONS *
444 REM **********************
446 PRINT "Initial conditions at t=0"
448 PRINT "=================================") PRINT
449 N=99 \ M=99
450 L0=Z1(M)/2
455 N1=N+1 \ V(N1)=0
460 PRINT "Distance of water table above top of sequence, l=n10190 m"
462 PRINT "Superficial water velocity at top boundary, V=v(n1)190 m/s"
464 IF X3=1 THEN 473
466 PRINT "$1%","Distance of water table above top of sequence, l=n10190 m"
468 PRINT "$1%","Superficial water velocity at top boundary, V=v(n1)190 m/s"
473 PRINT "$1%
475 PRINT\PRINT "$1%","J","E","DZ","Ht","V"
480 PRINT "$1%","--------","-------","--------","--------","--------")
482 IF X3=1 THEN 500
484 PRINT "$1%","J","E","DZ","Ht","V"
486 PRINT "$1%","--------","-------","--------","--------","--------")
500 I1(1)=I(1)
510 IF M=1 THEN 550
520 FOR L=2 TO M
530 L1=L-1 \ I1(L)=I1(L1)+I(L)
540 NEXT L
550 TH=0
560 FOR L=1 TO M
570 TH=TH+D(L)
580 NEXT L
590 HE=TH\L0
600 L1=L \ YD=0
610 E1(1) \ Z(1)=D(1)/I(1)
620 E1(1)=E1(1) \ KJ(1)=K(1)
630 H(1)=HE \ V(1)=(K(1)*HE)/(0.5*Z(1))
640 YD=YD+Z(1)
650 FOR J=2 TO N
660 E(J)=E1(L) \ Z(J)=D(L)/I(L)
670 Z(J)=Z(J) \ KJ(J)=K(L)
680 H(J)=HE \ V(J)=0
690 IF J=I1(L) THEN L=L+1
700 YD=YD+Z(J)
710 NEXT J
704 FOR J=N TO 1 STEP -1
705 PRINT J\E(J),Z(J),H(J),V(J)
707 IF X3=1 THEN 709
708 PRINT "$1%","E(J),Z(J),H(J),V(J)
709 NEXT J
710 PRINT\PRINT
711 PRINT "$1%","PRINT$1%
712 REM **********************
714 REM * TIME MARCHING *
716 REM * PROCEDURE *
718 REM **********************
720 T1=0
730 S=0
740 PRINT "Target time in seconds \"; INPUT T2 \PRINT
745 Y=0
750 T1=T1+T
755 FZ=Z(1)
760 IF T1<T2 THEN 790
761 PRINT \"Solution at t=";T2;\" seconds \";
762 PRINT \"Solution \";
763 PRINT \"Solution at t=";T2;\" seconds \";
764 PRINT \"Solution \";
765 PRINT \"Solution at t=";T2;\" seconds \";
766 PRINT \"Solution at t=";T2;\" seconds \";
767 PRINT \"Solution at t=";T2;\" seconds \";
768 PRINT \"Solution at t=";T2;\" seconds \";
769 PRINT \"Solution at t=";T2;\" seconds \";
770 S=1 \PRINT \"J \* E \* DZ \* Ht \* V \";
772 PRINT \"J \* E \* DZ \* Ht \* V \";
774 PRINT \"J \* E \* DZ \* Ht \* V \";
776 PRINT \"J \* E \* DZ \* Ht \* V \";
778 PRINT \"J \* E \* DZ \* Ht \* V \";
780 Z1=(1+(E2*Z1)/(1+E1))
781 K1=K1
782 H1=H1+(E2-E1)/(9.81*Av(1)))
784 V1=V1-(K1*K2)/(0.5*Z1))
786 E1=E2 \ H1=H1
788 Y1=Z1 \ Y=Y+Y1
790 L=1
792 FOR J=2 TO N
794 PZ=Z(J)
796 J1=J-1 \ J2=J-1
798 E2=E2+((1+E2)*(V2-V(J)*T)/Z(J))
800 Z(J)=(1+E2)*Z(J)/(1+E2))
802 K2=K2
804 H2=H2+(E2-E(J)/(9.81*Av(L)))
806 V(J)=V(J)+K1*K2*(H2-H1))/((K2*Z(J))+K1*Z(J))
808 E(J)=E2 \ H1=H2 \ H(J)=H1
810 Y1=Z1 \ Y=Y+Y1
814 IF J=I THEN L=L+1
816 NEXT J
818 REM \" TOP BOUNDARY CONDITION \"*
820 V(N1)=(K1*K9*(HE-H(N1)))/(0.5*Z(N)*K(N)+(LO*K(N))
824 L1=LO-(V(N1)*T+(1+E(N))*T/E(N))
828 L0=L1
830 IF L0>0 THEN 1000
832 L2=Z(N)+L1
836 L0=L2
840 N1=N1-1 \ N=N-1
842 K9=K9(N1)
846 ST=0
850 FOR J=1 TO N
852 ST=ST+Z(J)
856 NEXT J
860 HE=ST+LO
864 FOR J=1 TO N
866 IF Z(J)=Zo(J) THEN H(J)=HE
868 NEXT J
870 IF S=0 THEN 750
874 FOR J=N TO 1 STEP -1
876 PRINT J,E(J),Z(J),H(J),V(J)
878 PRINT J,E(J),Z(J),H(J),V(J)
880 PRINT \"Velocity at sequence tor is \";V(N1);\" m/s \";
882 PRINT \"Velocity at sequence tor is \";V(N1);\" m/s \";
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1100 PRINT "Consolidated sequence height =";YQ-Y"m"
1101 PRINT "Consolidated sequence height =";YO-Y"m"
1102 PRINT "CHECK USING TH-Y TH-Y"m"
1110 PRINT "Settlement =";Y"m"
1111 PRINT "Settlement =";Y"m"
1120 PRINT PRINT
1125 X3=1
1130 PRINT "Any more calculations (Y/N)"; INPUT A#
1140 IF A#="Y" THEN 446
1145 CLOSE X1
1150 END
APPENDIX 8: Representative calculations for settlement rate analysis using DECONSL computer program.
Echo input data and check for numerical stability

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Laver 1 numerically stable

Initial conditions at t=0

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Distance of water table above top of sequence, l = 2 m

Superficial water velocity at top boundary, V = 0 m/s

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Pore water velocity at sequence top is 0 m/s
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Settlement = 0.150459 m

**Solution at t= 21600 seconds**

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Pore water velocity at sequence top is 0 m/s
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Settlement = 0.264128 m
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Pore water velocity at sequence top is 0 m/s
Consolidated sequence height = 50.2288 m
Settlement = 271205 m

### Solution at t= .3456E+07 seconds

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Settlement = 409047 m
Solution at t = 432E+08 seconds

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<td>1.31153</td>
<td>1.07871</td>
<td>822018E-02</td>
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Pore water velocity at sequence top is 304496E-07 m/s
Consolidated sequence height = 49.8234 m
Settlement = 676571 m
Legend to Figures 3.17 and 3.18

Material Symbols

<table>
<thead>
<tr>
<th>Igminbitic</th>
<th>Fluvialite</th>
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</thead>
<tbody>
<tr>
<td><img src="image1" alt="Igminbitic Symbol" /></td>
<td><img src="image2" alt="Fluvialite Symbol" /></td>
</tr>
<tr>
<td>clay(ey)</td>
<td></td>
</tr>
<tr>
<td>silt(y)</td>
<td></td>
</tr>
<tr>
<td>sand(y)</td>
<td></td>
</tr>
<tr>
<td>gravel (ly)</td>
<td></td>
</tr>
<tr>
<td>peat and highly organic soils</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Particle Size (mm)</th>
<th>Fine (f)</th>
<th>Medium (m)</th>
<th>Coarse (c)</th>
</tr>
</thead>
<tbody>
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<td>0.2-0.6</td>
<td>0.6-2.0</td>
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<tr>
<td></td>
<td>2.0-20</td>
<td>20-60</td>
<td>&gt;60</td>
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</table>

Material Contacts

--- position approximate
---- position inferred

Stratigraphy

- Hn  Hinuera Formation
- KH  Hamilton-Kauroa Ashes
- Kp  Karapiro Formation
- Wg  Whangamarino Formation
- W.C.M. Waikato Coal Measures

--- Tauranga Group

--- Te Kuiti Group
GEOTECHNICAL INTERPRETATION

south-west
diagram to accompany "An Engineering Geological Investigation of Ground Subsidence above the Huntly East Mine Area."

phreatic surface of upper aquifer

north-east

BH 6599

BH 6651

Upper Ignimbritic Silt Unit

Upper Aquifer

Middle Aquifer

BH 6651

Kp

fms

CZ

SZ

Kp

BH 6599

ZS
GEOLOGICAL INTERPRETATION

Horizontal Scale = Vertical Scale
(Original Scale 1:500)
Panel 1
Irregular Pillar Widths
Due To Section Orientation

Renown Seam
WCM.
Kupakupa Seam
Basement

See Figure 3.19 for Section Location

Figure 3.17: Geological and Geotechnical Cross-sections for A-A'.
GEOTECHNICAL INTERPRETATION

Upper Aquifer

Middle Aquifer

Lower Aquifer?

phreatic surface of upper aquifer

north-west
Figure 3.18: Geological and Geotechnical Cross-sections for B-B'

Horizontal Scale  Vertical Scale
(Original Scale 1:500)

See Figure 3.19 for Section Location

Diagram to accompany thesis by Philip Ian Kelway:
"An Engineering Geological Investigation of Ground Subsidence above the Huntly Coal Mine Area."