FILL COMPACTION CRITERIA
FOR PORT HILLS GRAVEL
FORMATION SOILS, NELSON

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ABSTRACT

Port Hills Gravel Formation (PHG) is a Pliocene terrestrial soil-like deposit. It is located in Nelson as part of the Marsden and Port Hills Synclines. PHG is comprised of heterogeneous clay and silt bound gravel, with lenses of sandstone and occasional lignite seams and is intersected by northeast - southwest trending faults. PHG soils are considered good building material for the construction of earth structures such as embankment dams, gully fills and slope buttresses but there have been a number of failures in poorly constructed fills. The purpose of this thesis was to study the engineering properties of PHG soils, to review current fill operations and to identify any serious shortcomings affecting earthfill operations with PHG soils and to establish standard testing methodologies and compaction criteria for the PHG soils.
# TABLE OF CONTENTS

1.0 Intro  
1.1 Project Background .................................................. 2  
1.2 Project Objectives and Hypotheses .................................... 2  
1.3 Geological Setting ....................................................... 3  
  1.3.1 Port Hills Gravel Formation .................................... 3  
  1.3.2 Field Stratigraphy .................................................... 5  
2.0 SOIL SAMPLE DESCRIPTIONS AND CLASSIFICATION ................. 8  
2.1 Introduction ............................................................. 8  
2.2 Sample Descriptions ................................................... 9  
2.3 Soil Gradation Testing .................................................. 10  
  2.3.2 Methodology ......................................................... 10  
  2.3.2 Results ............................................................... 11  
2.4 Atterburg limits and Moisture Content ............................... 14  
  2.4.1 Methodology ......................................................... 14  
  2.4.2 Results ............................................................... 15  
2.5 Discussion and Synthesis .............................................. 16  
  2.5.1 Particle Size Distribution ...................................... 16  
  2.5.2 Plastic Limits ...................................................... 17  
  2.5.3 Indicated Permeability ........................................... 17  
  2.5.3 Index Behaviour Characteristics ................................ 17  
  2.5.4 Indicated Strength Characteristics .............................. 18  
3.1 Introduction ............................................................. 19  
3.1 Determination of the solid density ................................... 20  
  3.2.1 Sampling and Test Methodology ................................ 21  
  3.2.2 Results ............................................................... 21  
3.3 NZ Standard Compaction Test ......................................... 22  
  3.3.1 Test Methodology ................................................... 22  
  3.3.2 Results ............................................................... 23  
3.4 Field compaction trials ............................................... 24  
  3.4.1 Existing EarthFill Specification ................................ 24  
  3.4.2 Test Methodology ................................................... 25  
  3.4.3 Results ............................................................... 25  
3.5 Nuclear Densometer ..................................................... 28  
  3.5.1 Introduction ........................................................ 28  
  3.5.2 Testing Methodology ............................................... 29  
  3.5.2.1 Equipment Limitations ....................................... 30  
  3.5.3 Results ............................................................... 31
The project objectives were to:

- Identify differing engineering soil characteristics for PHG
- Ascertain optimum soil moisture and compactive efforts that will achieve the densest/strongest soil state possible
- Test the suitability of current monitoring/testing methodologies and identify potential areas of improvement
- Derive an empirical soils usage and compaction table for general use in the field
- Provide a model for the standardisation of field procedure and test methodologies in the Nelson region

1.3 Geological Setting

1.3.1 Port Hills Gravel Formation

The 1:25000 NZGS, ‘Geology of the Nelson Urban Area’ map (Johnston, 1979) shows PHG formation outcropping along the northeast - southwest striking Port Hills and Marsden synclines as far south as Champion Road, and along Atawhai Drive as far as Tui Glen, (Figure 1:1). The unit is intersected by a number of southwest – northeast and southeast – northwest trending faults.

Johnston (1979), describes the Port Hills Gravel Formation as being Miocene to early Pliocene in age and the basal member of the Tadmor Group; it is shown to unconformably underlie Oligocene to early Miocene Jenkins Group marine sediments and unconformably overlie Pleistocene, upper Tadmor Group, Moutere Gravel Formation.

It must be noted though that Johnston’s description of PHG being overlain by Moutere Gravel Formation can only be regarded as inferred, as there is currently no known physical contact between these units.
NOTES: Map is a composite based on NZMS 1:50000 Topographical Map and 1:25000 NZGS Geology of The Nelson Urban Area (1979)
According to Johnston PHG is a heterogeneous clay-bound gravel comprised of locally derived clasts. These clasts are reported to range in size between 10 mm to 800 mm long in a silty clay matrix. Thin sandstone, siltstone and mudstone beds are present throughout the Formation. Locally beds are carbonaceous with discontinuous coal seams (Figure 1.2 and Cut Log 1, Appendix A). Marine sequences within lower parts of the formation have also been identified; these units are often calcareous and can contain molluscs.

Figure 1.2: Photo shows the variety of soils typically encountered in the field. The location is the same as for Cut Log 1.

1.3.2 Field Stratigraphy

The general soil profile encountered in the field area consisted of a thin topsoil layer overlying between 0.2 m to 2.5 m of clayey silt with gravel, loess/loess colluvium
The permanent groundwater table has been found between 6.0 m and 20 m depth below surface and a perched water table is common at the colluvium/PHG interface.

Representative borehole, test pit and cut batter logs are shown in Appendix B.
2.0 SOIL SAMPLE DESCRIPTIONS AND CLASSIFICATION

2.1 Introduction

Soil classification testing was carried out to identify the physical/index properties of five different PHG samples.

Soil classifications are based on those of the Unified Soils Classification System adapted to British/New Zealand Standards.

PHG soil samples were collected from the following earthworks sites in the Nelson region:

- BRJV, playing field fill;
- GibAir water detention dam;
- Green Meadows subdivision (Stage IV).

Site locations are shown on Figure 1:1. Sampling was undertaken with the aide of mechanical excavators (diggers). The procedure was for the digger to excavate soil from the borrow face or fill platform and create a stockpile of approximately 2.0 m³. Soil samples were then taken from three different sides of the stockpile with a shovel. At each of the three sides soil was taken from the surface, centre and base of the pile to ensure a representative sample was obtained.

There is always a question of how representative a soil sample is in relation to the total area the sample has been taken from. As natural soils are generally not laterally or vertically continuous homogeneous bodies, inferring properties for a large area based on limited soil sampling may result in poorly set construction specifications and the use of plant equipment unsuitable for the type of soil.
To overcome the general problem of representativeness, and sampling bias, thorough observation of the borrow areas was undertaken to determine the range and approximate quantities of the different material types available. Following the identification of these soils large (approximately 40kg) bagged samples were taken. By taking the largest practical sample size under-representation of either fines or coarse materials was minimised. The average mass of the bagged samples was approximately 40 kg.

Field descriptions of the resultant bagged samples are listed in Section 2.2.

2.2 Sample Descriptions

Soil sampling was carried out in the field with the aide of mechanical excavators (diggers). The procedure was for the digger to cut the soil from the borrow face or fill platform and create a stockpile. This stockpile was then randomly sampled at three intervals with a shovel. At each interval soil was taken from the surface, centre and base of the pile. The field descriptions of the resultant bagged samples are listed below and their locations are shown in Figure 1.

BRJV B: SILTY CLAYEY-GRAVEL, very tightly packed, orange - brown, moist, dense, well graded, highly weathered, rounded, fine to cobble, granitic; in a low plasticity silty-clay matrix. This sample was obtained at 2.0 m depth below the original ground surface.

BRJV F: CLAYEY-GRAVEL with silt, very well compacted, orange brown, moist, dense to very dense, well graded, highly weathered, rounded with many broken faces, fine to coarse with small to medium cobbles, granitic; in a low plasticity silty clay matrix. This sample was obtained from the fill pad and was derived from the BRJV borrow.

GIBAIR B: SANDY-GRAVEL with silt, tightly packed, gap graded, moist to dry, dense, moderately weathered, rounded, medium to coarse with cobbles, granitic; in a coarse
The derived fines (passing the 63 μm sieve size) portion of the samples were analysed by the Saturn Digi Saturn 5200 Laser particle sizer, these tests were carried out on a 100 ml sample obtained by quartering of the bulk fines sample. The 100 ml samples were then treated with a 10 % hydrogen peroxide mix to remove any remaining organic material from the sample.

NZS: 4402 does not currently recognise the laser particle sizer as a means for attaining the PSD of fine soils. However for the purposes of soil classification it is the property of plasticity that is important rather than the measured grain size of the particle. In most cases, for engineering purposes it is only necessary to identify the portion of fines in the total soil mass and their plasticity characteristics, furthermore by acknowledging that there may be a small under reporting of the clay size fraction when utilising test results from the laser particle sizer, I believe this method of testing is sufficient.

Loss of some fines was expected but by taking care to minimise the amount of water used during wet sieving in conjunction with boundary separation techniques this loss was kept within acceptable margins (i.e. 1% as per NZS 4402: 1986).

Laboratory data sets are displayed in Appendix B.

2.3.2 Results
Summaries of gradation test results are shown in Table 2.1 and Figure 2.1. Individual test results are presented in Appendix C. Raw data sets are presented in Appendix D.

The PSD chart (Figure 2.1) clearly indicates a continuum of five different soil types, generally consistent with field engineering geological soil descriptions. These soil types are:

1. Clayey- Silt with gravel, gap graded (GMIV).
2. Silty Clayey Gravel, well graded (BRJV F);

3. Silty Gravel well graded (BRJV B);

4. Silty - Sandy Gravel, moderately well graded (GibAir F);

5. Sandy Gravel, poorly graded (GibAir B).

In engineering terms the most important observations of these results were:

- Substantial generation of fines during compaction (11 to 14 %). This increase in fines is likely to greatly reduce the permeability of the fill. It also has further implications on the design of filters as a decrease in D15 particle size will also require a reduction in the minimum effective filter size and constrains the maximum size as well. As the fines increase occurs predominantly in the silt size range it is possible that the soil may become more susceptible to erosion from fast flowing surface water.

- Soils are generally shown to be well graded or gap graded, this infers that tighter packing of soil grains during compaction and is likely, and it is likely that a denser soil profile will result compared to what would be expected from a poorly graded soil comprising the average grain size of the test samples. Well graded soils generally have low relative permeability.

- Coarse composite gravel soils with readily identifiable clay content (in the field) verge towards a fine soil (approaching 35% fines) after undergoing compaction. This has obvious implications for design, if the design assumptions are based on the assumed properties of the pre-compact ed soil;
- The effective particle sizes for compacted PHG fill material ranges from fine to medium silts, to clay. Again, this suggests soils of relatively low permeability. It also introduces the prospect of frost susceptibility for the fill, however Nelson does not have a high frost risk so this is unlikely to develop.

- A typical PHG fill can be expected to contain between 5% and 10% clay content. This is a definite indicator of low expected permeability. Gravel soils with clay are also considered to have very good workability properties, so placement of fill comprising these soils should be relatively easy.

Figure 2.1: Composite PSD plot for PHG soils.
2.4 Atterburg limits and Moisture Content

Atterburg Limits tests were carried out on four sub-samples (BRJVB, BRJVF, GibairF and GMIV). One sample, (GibAir B) had less than 4 % fines and displayed quick behaviour when an attempt was made to it. I decided on that basis, the sample was unlikely to have a liquid limit of $\geq 10 \%$ and assessed it as not plastic. Testing was conducted on the soil fraction passing a standard 425 $\mu$m sieve. These tests were performed in order to (i) determine the soils’ plasticity and (ii) to aid in soil classification of the samples and (iii) to infer engineering properties to the soils.

2.4.1 Methodology

The Atterburg Limits tests carried out for this project were:

- Liquid Limit ($L_L$) (NZS 4402:1986 Test 2.2)
- Plastic Limit ($P_L$) (NZS 4402:1986 Test 2.3)
- Plasticity Index ($P_i$) (NZS 4402:1986 Test 2.4)
- Linear Shrinkage ($L_s$) (NZS 4402:1986 Test 2.6)

Field moisture contents were derived for whole soil in accordance with NZS 4402: 1986, Test 2.1.

The raw data for all test sets is shown in Appendix C.

All tests were conducted in accordance with the stated New Zealand Standards.

The Atterburg Limits tests have two related limitations these are:
1. Operator bias; this is the most significant limiting factor affecting these tests because, (a) determination of the Plastic Limit is a matter of (subjective) judgement based on the experience of the person conducting the test. Results obtained for determining the plastic limit will have a direct effect on results for the plasticity index. Although any deviation should be within the accepted margin of error for a given operator, results should only be considered as indicative. In addition a series of sample results undertaken by different operators requires careful consideration before being accepted. (b) Determination of the liquid limit (via Cassegrande’s method, (NZS 4402:1986 Test 2.2)) requires judgement of the closing of the gap and is sensitive to both operator technique and the consistency of the rubberised base.

2. repeatability; Atterburg limits tests have been shown to have poor repeatability due to the factors described above.

The advantage of using these tests is that they require physical manipulation of the soil in the lab and are simply an extension of similar field tests conducted during site investigation. These tests tend to give results that are comparative to those of the soils encountered in the field.

2.4.2 Results
The composite results for the Atterburg Limits tests and Linear Shrinkage are displayed in Table 2.1 and data plots on the plasticity chart are presented in Figure 2.2. The field moisture content of the samples is also shown in Table 2.1 below:

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>USCS Soil classification (Fines)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PL (%)</th>
<th>LI (%)</th>
<th>LS (%)</th>
<th>M% (field)</th>
<th>Activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>BRJV B</td>
<td>ML (silt of low plasticity).</td>
<td>38</td>
<td>28</td>
<td>10</td>
<td>-0.1</td>
<td>8</td>
<td>7.6</td>
<td>6.7</td>
</tr>
<tr>
<td>BRJV F</td>
<td>CL (clay of low plasticity).</td>
<td>37</td>
<td>22</td>
<td>15</td>
<td>0.5</td>
<td>7</td>
<td>13</td>
<td>3.8</td>
</tr>
<tr>
<td>GIBAIR B</td>
<td>S (non plastic sand).</td>
<td>&lt; 10</td>
<td>NP</td>
<td>NP</td>
<td>NP</td>
<td>NP</td>
<td>6</td>
<td>NP</td>
</tr>
<tr>
<td></td>
<td>CL (clay of low plasticity)</td>
<td>31</td>
<td>13</td>
<td>18</td>
<td>2</td>
<td>7</td>
<td>10.5</td>
<td>7.5</td>
</tr>
<tr>
<td>----------</td>
<td>-----------------------------</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>---</td>
<td>---</td>
<td>------</td>
<td>-----</td>
</tr>
<tr>
<td>GM IV</td>
<td>MH (silt of high plasticity)</td>
<td>52</td>
<td>43</td>
<td>9</td>
<td>-2.5</td>
<td>23</td>
<td>14.5</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Note: The use of medium plasticity classifications is common to British standards, however this is not accepted industry practice in New Zealand or elsewhere outside the United Kingdom. It is generally accepted that a soil is either non-plastic (NP), low plasticity or high plasticity as per the original USCS classifications. Clay content to determine activity was provided from test results displayed in Table 2.

![Plasticity Chart for Port Hills Gravel Formation Soils](image)

2.5 Discussion and Synthesis.

2.5.1 Particle Size Distribution

Particle size distribution plots for samples BRJV and Gibair indicate considerable fines generation occurs during the compaction process. This fines generation is due to the release of fine silt and clay size particles from the compaction induced break-up of highly and completely weathered granitic cobbles and small boulders. The GM IV sample, grading curve indicates fine silt with a high clay portion (10 %) rather than the clay soil identified in the field.
2.5.2 Plastic Limits

Atterberg limits testing was supportive of the altered grading curves produced for the BRJV and GibAir samples. The gradation curves for these soils show an approximate doubling of the fines component (17.3 % to 31.7 %) for BRJV samples and almost a five fold increase (3.3 % to 14.4 %) for GibAir samples, following compaction. The over all increase in fines contained a tripling of the clay portion of the soils. Both soil groups show significant increases in soil plasticity as a result of this increase in clay fines.

The GM IV sample results had the soil plotting below the A-line indicating high plasticity silt.

2.5.3 Indicated Permeability

The high fines contents of the compacted PHG soils are likely to decrease the field permeability of the soils by at least an order of magnitude. The reasons for this are:

- Tighter packing of soil particles and a resultant decrease in air voids during compaction;

- The three fold increase in clay content. Clays are recognised, as being an impermeable material so any increase in the colloid content must translate into a decrease in permeability.

General materials based permeability estimates for these soils are in the range of $10^{-5}$ m/s to $10^{-8}$ m/s for GibAir F, BRJV B, BRJV F and GMIV samples. The poorly graded GibAir B sample is likely to be around $10^{-8}$ m/s (Barnes: 1988, Table 3.3).

2.5.3 Index Behaviour Characteristics

The Liquidity Index results indicate that at natural field moisture content PHG soils will exhibit the full range of behaviour between brittle and viscous flow when sheared. Brittle behaviour in the BRJV and GibAir field area are routinely observed (refer to Cut
Log 1 and Figure 1.2). In (localised) hand specimens of silts and clays at the GMIV site the soil can be extruded between the fingers, this area is the site of a complex series of landslides.

Activity for BRJV and Gibair materials indicate a highly active clay portion, this was not anticipated and not supported by linear shrinkage test results. However where failure planes have been identified in natural soils thin high plasticity shear seams are often present. Activity for the GM IV sample was normal this was also unexpected as indications were this would be a highly active soil (field observations and linear shrinkage). These results indicate that the clay in the more granitic PHG soils is likely to have a significant effect on the shear strength and behaviour of the bulk soil under differing moisture conditions. The GMIV soils will have shrink swell characteristics similar to a fat clay and careful control of this soil type will be required during placement.

Results indicate that GMIV and Gibair soils are likely to be quite sensitive to water content and may be susceptible to frost heave if placed too dry of optimum moisture content. However it should be noted that Nelson is an area with a very low frost risk.

2.5.4 Indicated Strength Characteristics

Based on gradation and plasticity characteristics and a fill compacted to at least 85 % relative density, the soils are likely to have typical effective cohesion and friction angle values ranges between:

- $c' = 2$ to $6$ and $\phi' = 33^\circ$ to $36^\circ$ for BRJV F;
- $c' = 0$ to $4$ and $\phi' = 34^\circ$ to $38^\circ$ for BRJV B;
- $c' = 0$ to $2$ and $\phi' = 33^\circ$ to $38^\circ$ for GibAir F;
- $c' = 0$ and $\phi' = 36^\circ$ to $41^\circ$ for GibAir B;
• \( c' = 6 \) to \( 8 \) and \( \phi' = 32^\circ \) to \( 34^\circ \) for GMIV;

The assessments of strength parameters were based on the empirical field density and shear strength tables presented in Appendix D.

CHAPTER 3 SOIL COMPACTION CHARACTERISTICS

3.1 Introduction

In engineering terms, compaction refers to the densification of a body of soil (earth fill, rock fill or natural ground) by the application of an applied force. For PHG soils these compactive forces are provided by mechanical tamping rollers (eg. sheeps foot) weighing between 6 and 22 tonnes, and these rollers may be vibratory.

Soil compaction specifications are typically derived from laboratory based density testing. In New Zealand the main test used is NZS 4402:1986: Test 4.1.1 New Zealand Standard Compaction Test, and this is also the test used most commonly in Nelson to establish compaction criteria for PHG soils.

Although NZ Standard Compaction is the most commonly used test in the Nelson region, it is worth noting that other options do exist. These are:

• Heavy Compaction. This test is the same as for the standard compaction test but with the use of a 4.5 kg rammer compacting thinner layers of soil in the same size mould.

• Vibrating hammer compaction, this method is generally used for coarse granular soils, such as road aggregate. It is not suitable for cohesive soils.
The development of the heavy compaction test came about primarily because of the development and use of heavier, more efficient field compaction plant. The heavier plant equipment tended to give field compaction results exceeding 100 % of the standard laboratory compaction test. The heavy compaction test was introduced as to keep the standard relevant.

One of the objectives of this thesis is to determine if the standard compaction test gives sufficiently reliable information (compared to field results) to support its continued use as a guideline for the setting of earthworks - fill placement specifications. The earthworks specification forms part of the contract documents for a project.

3.1 Determination of the solid density

The solid (particle) density of a soil is a useful parameter to determine. This is especially so if air voids are intended to be utilised as a measure of the effect of compaction. The solid density of a body of soil is often given an assumed average parameter of either 2.65 Mg/m³ or 2.70 Mg/m³. While primarily of use for calculating the percentage of air voids in a soil it is worth noting that it holds the same numerical value (in Mg/m³) as the Specific Gravity. This means that once determined it can also be used to calculate the bulk density, degree of saturation, ratio of voids and bulk unit weights for a body of soil.

If an assumed solid density is incorrect it can affect air voids by as much as 2.5 % (Table 3.1). This could be the difference between a strong fill and a much weaker one.

Table 3.1: Example of the affect solid density values can have on air void results for the same body of soil.

<table>
<thead>
<tr>
<th>Wet density Kg/m³</th>
<th>Dry density Kg/m³</th>
<th>Water density Kg/m³</th>
<th>Dry density calc Pd</th>
<th>Solid density assumed</th>
<th>water content measured</th>
<th>ratio of voids</th>
<th>%voids</th>
</tr>
</thead>
<tbody>
<tr>
<td>1970</td>
<td>1590</td>
<td>1000</td>
<td>1589.99</td>
<td>2700</td>
<td>0.239</td>
<td>0.03</td>
<td>3.11</td>
</tr>
<tr>
<td>1970</td>
<td>1590</td>
<td>1000</td>
<td>1589.99</td>
<td>2680</td>
<td>0.239</td>
<td>0.03</td>
<td>2.67</td>
</tr>
<tr>
<td>Year</td>
<td>Sample</td>
<td>Density (Mg/m³)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>------</td>
<td>--------</td>
<td>-----------------</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1970</td>
<td>BRJV F</td>
<td>2.67</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>BRJV B</td>
<td>2.70</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>GIB Air B</td>
<td>2.65</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>GIB AIR F</td>
<td>2.70</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>GMIV</td>
<td>2.67*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: *GMIV solid density was not tested for this project the result is derived from T&T file copies.

The solid density data shown in Table 3.2 shows a wide range of results. In general the soil passing the 19 mm sieve falls between 2.70 Mg/m³ and 2.75 Mg/m³ for the BRJV and GibAir soil samples. The considerably lower solid densities (2.58 Mg/m³ and
2.60 Mg/m³) of the fraction retained on the 19 mm sieve for these soil samples are probably related to air voids within the clasts, and in the case of the BRJV samples may also be affected by chemical and mechanical weathering processes.

The GMIV sample averaged results were not affected as > 85 % of the soil mass is passing the 19 mm sieve size.

3.3 NZ Standard Compaction Test

3.3.1 Test Methodology

A NZ Standard compaction test (NZS 4402:1986: Test 4.1.1) was carried out on four of the five PHG soil samples. Sample GibAir B was not tested, as the PSD results showed it was too coarse for the test to be conducted (< 35 % of the total soil mass passed the 20 mm sieve and < 55 % passed 37.5 mm) and results would not have been meaningful. I concluded that as the GibAir F sample was derived from The GibAir B soil, testing of the matrix material would not provide any critical additional information.

The NZ Standard Compaction tests were conducted on the soil fraction passing the 19 mm British Standard sieve. The soil sample is placed into a 1 litre mould in three even layers with each layer being subjected to a specified compactive effort supplied by a 2.5 kg rammer dropped a total of 27 times from a height of 300 mm. The sample was removed from the mould and a sub-sample cut, weighed and placed in an oven for drying, and on removal from the oven the specimen was weighed again. The bulk and dry unit weights obtained from the above process were then used to determine the moisture content, bulk (wet) and dry densities of the compacted sample. After the initial results were obtained a stone correction was carried out on all samples except for GMIV, and the final result recorded (Table 3.3).

For each soil sample the process was repeated 4 times, each time at different moisture content. The dry density results were plotted against the moisture content results. The
purpose of these plots is to produce a compaction curve, from which an optimum moisture content and dry density combination is derived.

The optimum moisture content (OMC) and Maximum dry density values are then used to devise a field testing assessment criteria (compaction targets), for the construction of an earth fill structure.

3.3.2 Results

Table 3.3 represents a summary of the test results and displays the range of maximum dry densities and optimum moisture contents obtained. Compaction curves are presented in Appendix B.

<table>
<thead>
<tr>
<th>Sample Identity</th>
<th>Solid density (t/m3)</th>
<th>Optimum Dry density (t/m3)</th>
<th>OMC (%)</th>
<th>Stone corrected density</th>
<th>Stone corrected M%</th>
</tr>
</thead>
<tbody>
<tr>
<td>BRJVB 1</td>
<td>2.7</td>
<td>1.66</td>
<td>12</td>
<td>1.85</td>
<td>8.53</td>
</tr>
<tr>
<td>BRJVF 1</td>
<td>2.71</td>
<td>1.64</td>
<td>16.6</td>
<td>1.8</td>
<td>12.6</td>
</tr>
<tr>
<td>GMIV 1</td>
<td>2.67</td>
<td>1.81</td>
<td>16.6</td>
<td>N/a</td>
<td>N/a</td>
</tr>
<tr>
<td>GIBAIRB 1</td>
<td>2.65</td>
<td>N/a</td>
<td>N/a</td>
<td>N/a</td>
<td>N/a</td>
</tr>
<tr>
<td>GIBAIRF 1</td>
<td>2.75</td>
<td>2.17</td>
<td>10.4</td>
<td>2.21</td>
<td>9.16</td>
</tr>
</tbody>
</table>

There is a noticeable difference between stone corrected and raw data values for the soils. In addition the expected pattern of higher dry density and lower optimum moisture content appears to hold true for the coarse soils. The fine(r) soils (Samples GMIV and BRJV F) show good correlation between samples once a stone correction has been made.

The purpose of the stone correction is to account for the low moisture content of the gravels in the fraction retained on the 19 mm sieve, and to enable a more realistic comparison between laboratory and field compacted dry density and moisture content values. The effect of the correction is to move the compaction curve up and to the left (eg. denser and drier), Table 3.3 demonstrates the effect of the correction very well.
3.4 Field compaction trials

3.4.1 Existing EarthFill Specification

The existing earthfill, placement and compaction specification currently being used in Nelson stipulates the following compaction criteria for an engineered earth fill to be considered adequate:

- 95% of laboratory (standard compaction) maximum dry density;

- The average of 10 tests to be \( \leq 8\% \) air voids and no single test to exceed 10% for non-cohesive soils, with values of 6% and 8% respectively for cohesive soils;

- Fill is to be placed at no more than 2% less than or 4% more than laboratory derived (standard compaction) OMC;

- The fill is to meet the following strength requirements:
  
  o \( \geq 5 \) blows per 100 mm of the Scala penetrometer, to a depth of at least 300 mm for non-cohesive soils;

  o An average of \( \geq 140 \text{ kPa} \) (per 5 tests within a 0.5m radius) corrected vane strength.

  o Scala penetrometer and shear vane results are discussed in Chapter 6.

The performance criteria listed above were set to construct an earthfill of high strength with very low consolidation rates and meet the requirements of NZS 4431: 1989 Code of Practice for Earthfill for Residential Development.

Test frequency requirements vary with the area of the fill, but as a general rule occurs at no less than every 0.5 m compacted thickness of fill.
The amount of compactive effort required to achieve the desired specification densities of an earth structure are determined by field compaction trials utilising the same compaction equipment intended for use during construction. The purpose of these trials is to assess the minimum compactive effort required to achieve the specified density/air void and strength targets.

3.4.2 Test Methodology

Field compaction trials were carried out for two PHG soil types. These were a silty-gravel (BRJV B) and a clayey-silt with gravel (GMIV). The fill trials comprised the following procedures:

- Compacting one 200 mm thick loose layer of fill with 4 passes of a sheep foot tamping roller, followed by placement of a further 200 mm loose layer and two passes of the tamping roller;

- Insitu density and strength testing consisting of one nuclear densometer test followed by one adjacent Scala penetrometer test within a half meter radius;

- A further two passes of the tamping roller, followed by further density and strength testing;

This process was repeated until either no practical density improvement or a density reduction occurred. Results were plotted as dry density against number of passes. This gave an optimum required compactive effort for the particular plant and soil type. Data plots are presented in Figures 3.1a to 3.1c. Test sheets are presented in Appendix C.

3.4.3 Results

The trial results displayed in Figures 3.1a to 3.1c indicate that for coarse soils with appreciable cobble content the minimum plant required must be greater than 10 tonnes in order to effectively break up the clasts and achieve the specification. For finer GMIV soils
the 6 tonne plant used for trialling was sufficient. However larger plant would be an advantage.

The trial test results indicate that at least 6 passes of suitable plant over soils at or near the optimum moisture content is required to achieve the specification.

The trial test results show good correlation between the achieved dry densities and air voids contents. For the silty-gravel soil (BRJV B) compaction trials with the 21 tonne roller achieved greater than 100 % of maximum laboratory dry density and had air voids below 8 % for the suggested 6 passes. This soil was within 3 % of laboratory OMC. The 21 tonne Rex roller is the standard roller used on both the BRJV and GibAir sites.

Figure 3.1a: Compaction trials of a PHG clayey-silt soil.
Figure 3.1b: Compaction trial of silty-gravel PHG soil.

Figure 3.1c: Compaction trial of PHG silty gravel soil with insufficient compaction plant.

For the clayey-silt soil the curve clearly indicates that there is a risk of over compaction for these soils and an upper maximum of 10 passes should also be included the field
methodology for this soil type. Typically 10 tonne and 13 tonne rollers are also used on this material especially in summer when it is usually dry of OMC and harder to compact.

The peak of the compaction curve does not meet 95 % of laboratory maximum dry density and the soil is between 5 % and 10 % wet of OMC. These figures are more consistent with the soils liquid and plastic limits than the laboratory derived OMC. This may suggest that excessive stone content influenced the laboratory compaction sample.

3.5 Nuclear Densometer

3.5.1 Introduction

The device used to carry out insitu field-testing of PHG fills was the Humboldt scientific, HS-5001EZ moisture density gauge (NDM). The NDM was designed to measure the field moisture and density of insitu construction materials (asphalt, compacted fill or natural ground). Density measurements are obtained by the reduction of gamma radiation due to Compton scattering and photoelectric absorption (Figure 5.2). The obtained results are directly related to the electron density of the measured materials (Humboldt Scientific Inc 1999).

The standard calibration provided with the machine is based on a material consisting of 50% limestone and 50% granite, which in turn is based on an assumption of the average engineering soils usually encountered during construction. The calibration is needed to provide a reference point to compare the gauge readings with a known standard to reduce the risk of excessive error. This standard calibration can be adjusted to achieve a better match, for materials of different chemical composition. An adjustment was not necessary for PHG soils as the composition of the unit is comparable to the assumptions of the standard calibration.

Measurement of moisture content is based on the slowing down of fast moving neutron radiation, which occurs primarily, as a result of the hydrogen content of the material. It is
affected to a lesser degree by other low atomic elements such as carbon and oxygen. The standard calibration for moisture is based on a water saturated silica sand standard, which is in turn used to calibrate a working standard. This standard can also be adjusted to compensate for other materials if necessary.

Figure 3.2: Nuclear densometer, diagram shows the radiation being emitted from the source, travelling through the ground and being picked up by the units receiver.

3.5.2 Testing Methodology
The HS500 was within its' calibration period at all times. The testing standards used for these tests are (ASTM D2922 (& D3017). The test methodology consisted:

- Preparation of a flat lying test pad (Scraping the fill surface to below the depth of the tamping feet imprints);
- Driving a round metal pin up to 300 mm into the fill (two holes within 0.5 m apart);
• Positioning the NDM over the hole, setting the solid density and inserting the probe;

• Average the reading for the two holes;

• Conducting either one Scala penetrometer test to at least 300 mm depth and recording blow counts per 100 mm, or for fully cohesive soils taking the average of five shear vane readings. (Both test sets were conducted within a radius of 0.5 m of the NDM test pad);

• Using engineering judgement assess the fill layer as either meeting specification or not. The testing specification used was the same as that detailed in Section 3.4.1;

• Issue and record instructions given to the contractor if required.

A total of 178 tests were conducted for BRJV F, 93 tests for GibAir dam and 63 for GM IV, data sheets are presented in Appendix C.

3.5.2.1 Equipment Limitations

Factors affecting the accuracy of NDM readings are as follows:

**Equipment error**: This refers to a series of potential variations due to measurement precision, chemical error, surface error, depth of measurement and drift.

• Precision is the statistical variation that occurs with repetitive measurements (standard deviation); it is a function of time and varies as the square root (i.e. a four fold increase in count time will improve precision by a factor of 2).

• Chemical error is that caused by variations in the chemical composition of the material being measured in relation to the standard. This is particularly important
for soils containing (1) hydrated minerals such as gypsum or (2) crystals such as mica, both of which can be the cause of considerable error in water content measurements and by connection bulk and dry densities. These errors can be minimised by identifying the presence of these materials during the investigation stages of a project and adjusting the calibration settings accordingly.

**Operator error.** This may occur for a number of reasons, primarily though it is due to inexperience and unfamiliarity with the equipment. Also the use of the recommended levelling sand or sieved local material can have a marked affect on moisture measurements and consequently density and air void measurements as well. Site comparisons undertaken using sand, sieved fill material and a cut surface gave consistently lower moisture contents (between 1 and 2 percent) than direct contact with a cut surface.

3.5.3 Results

NDM fill compliance monitoring field test results for all three-fill sites are presented in Appendix C. Summary graphs are shown in Figures 3.3a to 3.3c. The results display a wide range of variability in, material type and achieved dry density ‘pass’ results across all three sites.
Figure 3.3a Field compaction curve for the BRJV fill areas.

Figure 3.3b: Field compaction curve for GibAir Embankment Dam.
The range of dry density and moisture content readings that met specification for the adjacent BRJV and GibAir sites, and from GMIV are displayed in Table 3.4:

Table 3.4: Range of soil density and moisture contents for compacted fill at BRJV, GibAir and GMIV sites.

<table>
<thead>
<tr>
<th>Fill site</th>
<th>Moisture (%)</th>
<th>Dry density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BRJV &amp; GibAir</td>
<td>6.75 to 23.95</td>
<td>1638.9 to 2121.6</td>
</tr>
<tr>
<td>GMIV</td>
<td>11.9 to 23.6</td>
<td>1565.5 to 1916.5</td>
</tr>
</tbody>
</table>

It can be surmised from Table 3.4 that there is too vast a range of fill test pass results to indicate a homogeneous soil. It is also apparent that with such a vast range of results setting a density specification would not be much use. It can also be appreciated that setting a generic window of acceptable moisture contents is also impractical.
On closer inspection of the fill test data sheets it becomes apparent that the differences are almost entirely the result of changes in soil type.

The soil densities and moisture contents generally fit into four different and recognisable (in the field) soil groupings these are shown in Table 6.

**Table 3.5: Soil type and compacted density/moisture characteristics of PHG fill meeting specification:**

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Clayey-Silt</th>
<th>Clayey-Gravel</th>
<th>Silty-Gravel</th>
<th>Sandy-Gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dry density (kg/m³)</strong></td>
<td>1565 – 1780</td>
<td>1700 - 2300</td>
<td>1820 - 2140</td>
<td>1790 - 2050 (All Fails in this range)</td>
</tr>
<tr>
<td><strong>Moisture content (%)</strong></td>
<td>14.5 to 22</td>
<td>11 - 18</td>
<td>6.7 - 12</td>
<td>&lt;7.3</td>
</tr>
</tbody>
</table>

Table 3.5 clearly shows some overlap of the compacted dry density and moisture contents between the identified soil groupings represented. This indicates that the PHG soils are not so easily broken down into discreet soil groups. However what is apparent is that a continuum of soil properties, ranging from clayey-silt through to sandy-gravel exists for these soils and that changes are gradational. This ties in with what has been observed during site investigation work in these areas. Field Logs showing these variations in the natural soil profile are presented in Appendix A.

**3.6 Discussion and Synthesis**

The standout observation from the compaction testing data is the wide range of moisture and density results achieved (in tests assessed as meeting the required specification) within the identified soil groups in the field. The most likely reason for this wide variation is transitional soil types between the main groupings. Poor fill control could
also be a factor in some cases, although personal experience has shown me that wet weak soils with low air voids do not usually pass strength testing requirements.

These transitional soils are likely to be the result of:

- Natural variation in the soil profile;
- Soil change as a result of the compaction process;
- Mixing of soils in the borrow;
- Mixing and spreading of fill on the fill platform;
- Poor moisture or layer control during placement;
- Different testing operators.

This variability between field results precludes the use of compacted dry density as a measure of fill compliance simply because there would not be a consistent target to achieve. What is needed is a variable that can be controlled, and will give a good indication of the effectiveness of the compaction effort, and also to provide information on the likely strength of the compacted fill.

Using a percentage of air voids as the default measure of acceptability provides a clear target for the contractor to achieve. It is well recognised however that it is easy to attain low air voids simply by wetting the soil, and providing a bare minimum of compaction effort. This does not produce a strong, dense fill, however if the range of laboratory maximum dry density and OMC values are known then air voids can be used in conjunction with strength testing (Scala penetrometer and field vane) as a good measure of fill quality.
All NDM pass results for BRJV and Gibair fill types GC and GM generally exceed the laboratory standard compaction, maximum dry densities. For practical purposes all achieve 95%. This indicates that for the typical compaction plant utilised (21 tonne Rex roller) at these sites the New Zealand Standard Compaction test does not provide a meaningful measure. Compacted moisture contents for these soils were between -2% and +5% of the laboratory OMC which suggests that if looked at as individual soil units upper and lower limits can be set for soil moisture content.

The average air voids for these fills is approximately 6%. These material types comprise the majority of PHG fill material used in the Nelson area. As these soils are easily compacted to achieve under 10% air voids it may be prudent to reduce the maximum allowable air voids content to 8% and bring the specification into line with that of the cohesive soils

The GP-GM (silty-sandy gravel) soil type did not attain 95% of laboratory dry density. This soil type did not record any pass results testing indicated it fell outside the allowable grading curve for the fill and it was excavated from the fill and stockpiled for future use elsewhere. Compacted moisture content for the material was 2% dry of laboratory OMC, and was too wet, weaving was observed during trafficking. Air voids for this material also exceeded the 10% maximum allowance. Testing of this soil type should probably be conducted using the Standard Penetration Test (SPT) and related to relative density rather than laboratory derived test parameters. Future work is required to resolve issues with this material type.

Pass results for the clayey-silt material at GMIV were below laboratory maximum dry density and also did not achieve 98%. 95% of maximum dry density was regularly achieved. However the majority of tests achieved less, the lowest value being equivalent to 86% of the laboratory maximum. Field observations of this area have led me to believe that a lot of the fill placed on this site was compacted well wet of optimum, however
there is also a marked variability between the two dominant materials at this site. They range from, that of the GMIV test sample, to silty gravel with much clay, the silty gravel soil is similar in properties to the BRJV F test sample.

Compacted moisture contents for these soils were between -2% and +5% of OMC. Air voids for this material were generally less than 5% and the fill demonstrated good strength characteristics. The range of moisture contents above and below laboratory OMC is very similar to that of the BRJV and GibAir soils, accordingly -2% and +5% of OMC appears to be the range these soils remain workable in.

The compaction charts for BRJV show increasing dry densities with decreasing moisture content below OMC, especially within silty gravel soils. This relationship is not consistent with conventional wisdom and does not track laboratory results. The most likely reason for this observation is a prolonged compaction effort by the contractor. This soil type was often placed dry of optimum due to problems with obtaining a suitable water cart for site. In this particular project the client was also the contractor and was prepared to expend the extra time and effort to achieve the required air voids specification under these soil moisture conditions. This is unlikely to be a practical option under most circumstances.

The air voids versus moisture content plot for BRJV soils (Appendix C) shows an inverse relationship consistent with expected trends (eg. high moisture/low air voids)

Soil workability and construction issues are covered in Chapter 5.
4.0 POST COMPACTION ENGINEERING PROPERTIES OF PORT HILLS GRAVEL SOILS

4.1 Introduction

Most soil assessments are concerned with determining the shear strength and shear strength parameters of the soil for the purpose of conducting either natural slope or embankment stability analysis.

For the purposes of this project strength refers to the undrained strength \((S_u, C_u \text{ and } q_u)\) of laboratory compacted PHG soil samples. The undrained condition was selected because of, the low expected soil permeability's indicated by the soil index testing detailed in Chapter 2 which would make fully drained conditions in the field unlikely.

The samples selected for this testing were the BRJV F and GibAir F soils as these will provide a good range of the material properties existing within PHG Formation soils.

To determine the post compaction characteristics of typical PHG fill materials, a programme of direct strength testing consisting of shear box and triaxial tests were carried out in the laboratory, and field vane and Scala penetrometer testing was undertaken in the field. The purpose of these tests was to gain an insight into how the soils would react under differing stress conditions and how that would affect the soils strength, permeability and volume change characteristics.

An assessment is also made about PHG fill workability, erodibility and deterioration characteristics.

Permeability testing was carried out on the compacted fill embankment of GibAir Dam. Laboratory testing was not considered relevant because a specimen sample recompacted in the laboratory at a reduced scale and artificially graded was unlikely to give a better
measure of permeability to a test conducted on insitu soils in the field. As a result of this assumption no laboratory testing was conducted.

4.2 Triaxial Testing

4.2.1 Undrained Unconsolidated Testing Procedures

The BRJV F and GibAir F soils used for this test had been previously scalped to all passing the 19 mm sieve size. The soil was compacted to either 98 % of New Zealand Standard Compaction dry density, or where 98% wasn’t achieved the highest value attainable with the sample provided. The purpose of compaction was to densify the soil as much as possible for the moisture content it was to be tested at, and to approximate a range of possible fill conditions immediately following construction.

Compaction was undertaken in a 300 mm long by 150 mm wide (internal diameter) split mould. Soils were compacted in 6 layers each receiving 62 blows from a 2.5 kg rammer.

The equipment used consisted of a Humboldt triaxial testing machine with a large diameter (150 mm) load cell (Figure 4.1). The use of the large cell enabled testing to be conducted on the portion of soil passing a standard (British) 19 mm sieve size and slightly reduce the introduced fines to coarse ratio bias of typical 100 mm diameter load cells.

The samples were tested in accordance with NZS 4402:1986 Test 6.2.1

The samples were compacted in a split mould at close to optimum moisture content.

Once the specimen was prepared it was placed on the pedestal, the end cap fitted and the cell lowered onto and fastened to the pedestal. Checks were made to ensure taps had been switched to the closed position and strain rate set for the loading ram.
The load cell was lowered onto the top of the cell and cell pressure applied. For the multi-stage test, loading was suspended when the stress/strain curve appeared to flatten off. The cell pressure was then adjusted to the next increment and loading continued.

Testing continued until either failure of the sample or steady state conditions were reached.

Figure 4.1: Humboldt triaxial machine and load cell.

The undrained unconsolidated test (UU) was conducted to determine the immediate post-construction strength of the fill, and also to provide a comparison with the direct shear (shear box) test results. The samples were tested partially saturated to model insitu ground conditions above the water table.
The original intention was to test three samples in the fully consolidated state at different normal stresses, but time and availability constraints of the equipment did not allow for this option. It was decided to conduct one test each at moderate to high confining stress (200 and 400 kPa) and then a second follow-up multi-stage test on each soil sample.

4.2.2 Undrained Unconsolidated Test Results

4.2.2.1 Sample BRJV F

Result sheets are presented in Appendix B.

- These samples only managed to achieve compaction at 93 % of maximum laboratory dry density, and had saturation ratios (Sr) between 58 and 61 %.

- The failure mode for both samples was plastic and exhibited the classic shortening and fattening associated with this failure mechanism (Figure 4.3).

- Normal stress versus time plots (Appendix C) for these tests show a steady rise to peak stress then a gradual flattening beyond this point.

- Plots of average deviator stresses (Figure 4.2) and Mohr circle representations (Appendix B) show $C_{UU}$ at around 50 kPa and a small apparent friction angle of $\phi_{UU}$ 2°. This indicates undrained unconsolidated shear strength over the testing range of 66 kPa.

Figure 4.2: Plot of Total deviator stress versus shear stress for Sample BRJV F.
• The strain versus time plot for the multistage test indicates peak strength occurs at around 3.5 % strain.

Figure 4.3: Triaxial specimen BRJV F on failure. This specimen exhibits uniform plastic shear.
4.2.2.2 Sample GibAir F

- The GibAir F samples proved hard to compact in the laboratory and only achieved as little as 76 % of maximum laboratory dry density for the multistage sample and 86.8 % for the single stage test; high gravel and sand content are a possible cause of this. The post compaction $S_c$ values for these samples were 133 % and 106 % respectively. These values tend to indicate that the sample was probably dominated by fine sand and silt and that laboratory OMC for this material is possibly too high. Alternatively it may be a simple case of the air voids closing and rearrangement of soil grains during compression that has led to this state.

- The failure mode for both GibAir F samples was brittle, and both exhibited a planar diagonal shear (Figure 4.6).

Figure 4.4: GibAir F Triaxial sample at failure exhibiting a brittle shear mechanism.
- Normal stress versus time plots (Appendix B) for these tests show a rapid rise to peak stress followed by failure and complete loss of strength. Given the $S_r$ values for this soil it is likely the sample underwent a sharp rise in pore water pressure under increasing normal load.

- All plots indicate a short time (between 16 and 32 minutes) to failure this is approaching double the rate of the BRJV F samples for the same load rate of 0.27 mm/min.

- Plots of average deviator stresses (Figures 4.2 and 4.4) and Mohr circle representations (Appendix B) show good correlation. Both plots show $c_{UU}$ at around 19 kPa and an apparent friction angle of $\Phi_{UU} 11^\circ$. This indicates undrained unconsolidated shear strength over the testing range equivalent to 86 kPa. The high friction angle for this test is likely to be, primarily, a result of rapid consolidation and strength gain during early stage loading. A larger percentage of coarse particles (relative to BRJV F) may also have contributed.

- The strain versus time plot for the multistage test indicates peak strength occurs at around 3% strain, which is about the same as the BRJV sample.

4.2.2.3 Discussion
A comparison of the two (BRJV F & GibAir F) sets of test results indicates that peak strength is probably dependant on the initial confining pressures the soil is subjected to prior to the start of loading. It also suggests that and that increasing the normal load without allowing for consolidation between steps is not likely to provide an increase in shear strength.
The partially saturated state of the soils may have resulted in a slight over estimation of shear strength due to the generation of negative pore water pressures during early stage loading, especially in the GibAir F sample set.

The GibAir F sample was poorly compacted and may have experienced artificially accelerated strain during loading.

Both test sets only comprised two end member-confining stresses and the mid points have been generated by extrapolation. Normally testing would occur over at least 3 different confining stresses to enable a curve to be generated. Strictly
speaking results should be viewed as specific to the initial confining stress for each test.

4.2.3 Consolidated undrained test (with pore pressure measurements)

The undrained consolidated test (CU) was undertaken to assess the long-term strength of the fill once sufficient time had past to allow 90% consolidation. Pore pressure measurements were taken to enable effective strength parameters to be derived. All consolidated undrained test results are presented in Appendix C.

4.2.3.1 Methodology

These tests were carried out by Geotechnics Ltd of Auckland utilising 100 mm diameter load cells, in accordance with BS1377: Part 8:1990:Test 7.

The base assumptions used for determining the type of triaxial testing to be conducted were:

- The insitu soils comprise inhomogeneous composite clayey or silty gravel;
- The test samples were consistent with the typical PHG fill materials;
- The lab compacted samples are sufficiently comparable to compacted fill in the field, that results are meaningful.
- The compacted fill was of sufficiently low permeability to severely restrict the passage of water into or out of the soil mass;
- The fill material was constructed above the water table and/- was sufficiently drained to prevent full saturation occurring (UU);
- Very low, immediate and long term settlement rates are expected in fills meeting placement specifications;
• Deviator stress was expected to gradually increase with increasing cell pressure until such time as all air voids were expelled from the soil mass and a state of saturation reached;

4.2.3.2 Test Results
Sample BRJV F

• The Mohr circle plot for this sample (Figure 4.6) gave values c and c' of zero and \( \Phi' \) of 30°. Actual c plots were negative and had to be made to zero. This may indicate the sample reached a state of full saturation during loading. The low effective friction angle is likely to be over conservative and may indicate silt dominance of the sample.

• The failure mode for the sample was planar.

• The stress path analysis (Appendix B) does show the expected soil behaviour and indicates an over consolidated clay soil.

• Pore pressure is shown to increase linearly with increasing normal stress.

• Consolidation plots show increasing volume change with increasing confining stress.
CONсолIDATED-UNDRAINED TRIAxIAL COMPRESSION TEST (3 StAGEs)
MOHR CIRCLES OF TOTAL AND EFFECTIVE STRESSES

Initial Sample Height: 199.99 mm
Initial Sample Diameter: 100.34 mm
Initial B Value: 10 %
B Value before Consolidation: 92 %

<table>
<thead>
<tr>
<th>Consolidation Stage</th>
<th>Cell Pressure (kPa)</th>
<th>Back Pressure (kPa)</th>
<th>Eff. Consol. Stress (kPa)</th>
<th>Deviator Stress (kPa)</th>
<th>Pore Pressure Change During Shearing $\Delta u$ (kPa)</th>
<th>Effective Principal Stress (kPa)</th>
<th>Vertical Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>STAGE 1</td>
<td>550</td>
<td>500</td>
<td>50</td>
<td>31.44</td>
<td>33.7</td>
<td>47.74</td>
<td>16.30</td>
</tr>
<tr>
<td>STAGE 2</td>
<td>600</td>
<td>500</td>
<td>100</td>
<td>67.51</td>
<td>66.1</td>
<td>101.41</td>
<td>33.90</td>
</tr>
<tr>
<td>STAGE 3</td>
<td>900</td>
<td>500</td>
<td>400</td>
<td>281.59</td>
<td>261.7</td>
<td>419.89</td>
<td>138.30</td>
</tr>
</tbody>
</table>

Angle of Frictional Resistance: $\phi = 15 ^\circ$
Cohesion: $c = 0$ kPa
Linear Regression Coefficient: $r = 1.000$

Sample History: The sample was remoulded at the optimum moisture content to 98% of the maximum dry density (OMC=16.6 %, MDD=1.64 t/m³)

Soil description: SILT, with some clay, sand and gravel, firm, light brown with light grey, orange brown and black mottles, medium to high plasticity, dilatant.

Failure Mode: Planar

Test Remarks: Failure for each stage was determined by the maximum deviator stress. Strength parameters have been derived by using a linear regression fitting method. $C$ and $C'$ derived were negative, and therefore were both forced to be zero with $\phi$ and $\phi'$ recalculated.

Entered by: [Name] Date: 9/11/07 Checked by: [Name] Date: 9/11/07
Figure 1.7

CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST (3 STAGES)
MOHR CIRCLES OF TOTAL AND EFFECTIVE STRESSES

Initial Sample Height: 205.85 mm
Initial Sample Diameter: 100.38 mm
Initial B Value: 2 %
B Value before Consolidation: 90 %

<table>
<thead>
<tr>
<th>Consolidation Stage</th>
<th>Failure Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Coll Pressure (kPa)</td>
<td>Back Pressure (kPa)</td>
</tr>
<tr>
<td>STAGE 1</td>
<td>500</td>
</tr>
<tr>
<td>STAGE 2</td>
<td>550</td>
</tr>
<tr>
<td>STAGE 3</td>
<td>850</td>
</tr>
</tbody>
</table>

Angle of Frictional Resistance: $\phi = 24^\circ$
Cohesion: $c = 59$ kPa
Linear Regression Coefficient: $r = 0.988$

Sample History: The sample was remoulded at the optimum moisture content of 10.4 % to the highest density achievable in the Laboratory.

Soil description: GRAVEL, silty and sandy, loose (disturbed), brown with dark grey and yellowish brown.

Failure Mode: Planar
Test Remarks: Failure for each stage was determined by either the maximum effective stress ratio or the maximum deviator stress. Strength parameters have been derived by using a linear regression fitting method.
CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST (3 STAGES)

MOHR CIRCLES OF TOTAL AND EFFECTIVE STRESSES

PROVISIONAL ONLY

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Initial Sample Height: 198.19 mm
Initial Sample Diameter: 100.18 mm
Initial Bulk Density: 1.98 t/m³
Initial Water Content: 14.6%
Initial Dry Density: 1.73 t/m³
Final Water Content: 19.9%

<table>
<thead>
<tr>
<th>Consolidation Stage</th>
<th>Cell Pressure (kPa)</th>
<th>Back Pressure (kPa)</th>
<th>Consolidation Pressure (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>STAGE 1</td>
<td>650</td>
<td>600</td>
<td>50</td>
</tr>
<tr>
<td>STAGE 2</td>
<td>700</td>
<td>600</td>
<td>100</td>
</tr>
<tr>
<td>STAGE 3</td>
<td>800</td>
<td>600</td>
<td>200</td>
</tr>
<tr>
<td>Deviator Stress</td>
<td>σ1-σ3 (kPa)</td>
<td>Pore Pressure Change During Shearing (kPa)</td>
<td>Effective Principal Stress (kPa)</td>
</tr>
<tr>
<td>67.85</td>
<td>29</td>
<td>88.85</td>
<td>21.00</td>
</tr>
<tr>
<td>113.23</td>
<td>53</td>
<td>160.23</td>
<td>47.00</td>
</tr>
<tr>
<td>229.22</td>
<td>97</td>
<td>332.22</td>
<td>103.00</td>
</tr>
</tbody>
</table>

Total

Effective

Angle of Frictional Resistance: φ = 21°
Cohesion: c = 3 kPa
Linear Regression Coefficient: r = 0.999

Sample History: Remoulded at the optimum moisture content to 98 % of the maximum dry density obtained from New Zealand Standard Compaction test.

Soil Description: Wheathered GRAVEL (fine to coarse), sandy, loose, brown, dilatant, low to medium plasticity, dilatant.

Failure Mode: Plastic

Test Remarks: Failure for each stage was determined by either maximum effective stress ratio or maximum deviator stress. Strength parameters have been derived by using a linear regression fitting method.

Entered by: Date: Checked by: Date:
Sample GibAir F

- The Mohr circle plot for this test (Figure 4.7) indicates values of $c'$ equal to 26 kPa and $\Phi'$ of 36°. The very high $c'$ value was not expected, and may be affected by the low B value (90%). The B value is a measure of the soils saturation. A fully saturated soil would have a B value of unity (eg.1 or 100%). A low B value indicates that there are air voids present in the soil structure; this can lead to negative pore water pressures and a stiffening of the soil when it comes under load. This stiffening takes the form of an apparent cohesion and it is this phenomenon that is likely to have occurred with this sample.

- Pore water pressure appears to be independent of deviator stress at low confining pressures (50 and 100 kPa), but shows a considerable increase at high confining pressure. This tends to indicate that during compaction of most small to medium structures (vertical height not exceeding 20 m) a build-up of pore water pressure as a result of compaction is unlikely. However for larger structures it may become critical as the rise in pore pressures in this sample coincide with fairly large volume changes.

- Stress path analysis (Appendix B) shows the soil to be lightly over consolidated.

- Very little volume change occurred during the first two consolidation phases, but a significant reduction occurred during stage 3.

- For this sample total stresses equal the effective stresses at low confining pressures (50 and 100 kPa) for normal stresses of up to 250 kPa. This is indicative of the strength of the soil skeleton and it's apparent cohesion being able to withstand considerable load before the particles start to break down.
There is no obvious relationship between total stresses from this test or those of the unconsolidated undrained test. However that is not surprising when only comparing with single samples. The Consolidated test would be expected to have a higher undrained shear strength than the unconsolidated sample simply because it has been consolidated.

**Sample GMIV**

This sample was tested as part of early subdivision works at Green Meadows. The soils observed on this site have been consistent for each stage of the subdivision. So the original tests are assessed as being valid for present use.

- The Mohr circle test results the effective stress parameters for this soil as c' of 7 kPa and Φ' of 30°. The plot is presented in Figure 4.8.

- In this sample the pore water pressures increase in conjunction with the deviator stress. This is the normal relationship between the parameters.

- Based on the c' and Φ' values alone it would appear that the GMIV and BRJV F samples have very similar strength characteristics. This is not the case as the GMIV sample is more silt dominant and the sample accounts for the whole soil whereas the BRJV F sample 82 % of the total soil.

**4.3 Shear Box Testing**

**4.3.1 Testing Methodology**

Shear box testing was conducted on the BRJV F and GIB F samples as these were considered the most common fill type in use, and they also provide good representation of the PHG fill in general.

The shear box testing had two objectives:
1. To determine if a direct relationship between the undrained unconsolidated triaxial tests could be established.

2. To observe the relationship between moisture content and shear strength over a range of placement moisture contents.

4.3.2 Test Methodology
Equipment consisted of a hand fabricated 300 mm$^2$ x 100 mm deep split shear box attached to a lathe frame. The normal load was applied by a 95 kPa load plate and a pneumatically operated load frame (Figure 4.9). Electronic gauges connected to a computer recorded vertical displacement and time readings. Horizontal displacements were recorded manually via dial gauge.

Figure 4.9: Set-up of the shear box apparatus.
Figure 4.10: BRJV F shear box test showing diagonal shear profile, encountered during this test set. A solution was not found.

The rate of shear was applied manually by turning a calibrated handle attached to the ram and load cell. The average rate of shear for the test was approximately 2.1 mm/minute. Three tests were conducted on each sample. The original intention was for one at OMC, one above OMC and the other below. The samples were compacted in the shear box in three layers of equal thickness utilising the 2.5 kg rammer and a standard compaction mould, the edges of the mould were slightly overlapped to ensure adequate coverage of each layer.

Following compaction the load plate was placed over the top plate and the loading frame positioned and levelled. Dial gauges were then set in place and a further 50 kPa normal load applied.
For the BRJV F sample an averaging correction was required for all vertical displacement measurements (dilation).

4.3.2 Test Results

Table 4.1 below provides a summary of the testing results. Data plots and calculation sheets are presented in Appendix C.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Sample ID</th>
<th>Moisture content (%)</th>
<th>Dry density (Mg/m³)</th>
<th>Normal Stress (kPa)</th>
<th>Peak Shear Strength (kPa)</th>
<th>Residual Strength (kPa)</th>
<th>Strain at Peak Strength (%)</th>
<th>Time to Peak Strength (min)</th>
<th>Average Strain Rate (mm/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BRJV F</td>
<td>13.7</td>
<td>1.54</td>
<td>144</td>
<td>247.6</td>
<td>51</td>
<td>0.6</td>
<td>0.8</td>
<td>2.1 mm/min</td>
</tr>
<tr>
<td>2</td>
<td>BRJV F</td>
<td>10.7</td>
<td>1.38</td>
<td>144</td>
<td>83.4</td>
<td>52</td>
<td>1.5</td>
<td>1.3</td>
<td>2.1 mm/min</td>
</tr>
<tr>
<td>3</td>
<td>BRJV F</td>
<td>9.4</td>
<td>1.38</td>
<td>144</td>
<td>107</td>
<td>58</td>
<td>0.9</td>
<td>2.2</td>
<td>2.1 mm/min</td>
</tr>
<tr>
<td>4</td>
<td>GIB AIR</td>
<td>12.2</td>
<td>1.69</td>
<td>144</td>
<td>78.6</td>
<td>46</td>
<td>1</td>
<td>1.4</td>
<td>2.1 mm/min</td>
</tr>
<tr>
<td>5</td>
<td>GIB AIR</td>
<td>5.4</td>
<td>1.52</td>
<td>144</td>
<td>51</td>
<td>18</td>
<td>0.7</td>
<td>1.1</td>
<td>2.1 mm/min</td>
</tr>
<tr>
<td>6</td>
<td>GIB AIR</td>
<td>5</td>
<td>1.49</td>
<td>144</td>
<td>51</td>
<td>22</td>
<td>0.7</td>
<td>1.1</td>
<td>2.1 mm/min</td>
</tr>
</tbody>
</table>

- Compacting the soil in the shear box and maintaining moisture control proved difficult especially for the Gib Air sample, because of these difficulties the intended moisture profiles for the test were not achieved. The BRJV F sample compacted at 13.7% moisture achieved 94% of laboratory maximum dry density, the other BRJV F samples achieved 84% of maximum lab dry density. The

- The GibAir F sample compacted at 12.2% moisture content was only 78% of laboratory maximum dry density. The other samples achieved 69% and 70% of laboratory maximum dry density.

- As shown in Table 4.1 there appears to be a linear relationship between the undrained unconsolidated peak shear strength and the relative % of maximum dry density. This apparent relationship shows that under the same normal stress, the peak shear strength increases as relative densities increase and is independent of soil type.
• The test results indicate that for soils within the normal range of placement moisture contents (all samples except GibAir F samples 5 & 6) soils at peak stress can be expected to have $c'$ values between 5 to 10 kPa and $\Phi'$ values between 30 and 34°, relating to stiff to hard soils (extrapolated effective stress parameters are derived from the undrained shear strength of the soil using the Tables presented in Appendix D). These values are likely to be correct for the BRJV F samples as they represent 82% of the total soil sample. For GibAir F these values are likely to be conservative as the scalped sample only represents 34.7% of the total soil sample. Of real interest is that the table derived effective stress parameters closely resemble those of the undrained consolidated triaxial tests for these soils.

• The residual undrained unconsolidated shear strength is shown to be relatively consistent throughout what is expected to be the normal range of placed moisture contents. The residual shear strength range for the soils is between 46 and 56 kPa.

• Peak shear strength across all tests is shown to occur at or near to the point of maximum compression.

• Shearing of the BRJV F sample was diagonal to the horizontal and has influenced the dilation results for these samples (Figure 4.10). This was not observed in the GibAir F samples.

• Shear strengths appear to correlate well with the total stress results for BRJV F and GMIV consolidated undrained triaxial tests. This may be related to the state of the soil at low confining stresses, especially in the case of the GibAir F, which had very little pore pressure response until it was subjected to pressures in excess of 250 kPa. This relationship needs to be explored further.
• If further testing was supportive of these test observations shear box testing would be a viable alternative for testing the insitu/post construction strengths of soils in the field.

The main limitations of this test are:

• Results apply to soils in the partially saturated state and may indicate higher peak shear strength results than would be achieved in the fully saturated condition;

• Problems with non horizontal shear in the BRJV F samples require vertical displacements to be considered as indicative only;

• For this type of testing a larger data set would be required to be statistically valid;

• Peak shear strength results may have been affected by anisotropic strain concentrations within the samples, especially for BRJV F soils.

4.4 Field Permeability Testing

Field permeability testing was carried out on the GibAir Dam fill platform once it had reached approximately 3 m vertical height of placed fill. The compacted soils comprised clayey gravel similar to the BRJV F sample and silty gravel similar to the BRJV B sample. Three-barrel permeability tests were undertaken.

The calculation sheets and construction sketch are presented in Appendix C. Figure 4.11 shows a constructed test barrel.
4.4.1 Test Methodology

A barrel permeability test was constructed in the following way;

- A hole about 300mm deep was dug in the area to be tested, and the ground levelled off with all loose material removed. Then a 44 gallon oil drum, with both ends cut off was placed in the hole.
- Backfill was tamped around the sides of the hole and some dry bentonite was added to the fill to help make a seal.
- A bentonite seal was placed around the inside edge of the base of the drum. Then the ground was built up around the sides of the barrel (above original ground level) to help seal it all in.
- A 50 mm layer of coarse, clean gravel was placed in the base of the barrel.
- The barrel was filled with water and the test started. Measurements were taken of the height and width of the barrel (inside and outside) and of the height of water.
- Each test ran for three to four days, until steady state conditions were achieved for at least 1.5 days of that time. At least 6 readings are required over the test period.
- The average of the steady values was used to calculate the permeability. A running plot of head loss over time maintained (Figure 4.12) to enable the steady state condition to be identified.

4.4.2 Results

The results of the barrel permeability test gave k values in the order of $3 \times 10^{-8}$ m/sec for the clayey gravels, and $3 \times 10^{-7}$ m/sec for the silty gravel. The minimum acceptable result for the dam was $1 \times 10^{-7}$ m/sec. These k values are similar to what was derived empirically via soil index testing in Chapter 2.
Figure 4.11: Gib Air Dam Barrel permeability test

Figure 4.12: Head loss over Log(10) time barrel permeability plot
4.5 Field vane (19 mm)

The 19 mm field shear vane is used to give an approximation of the undrained shear strength of an insitu clay soil, or in this case compacted clay fill. It is used in conjunction with field density and moisture testing to assist in quality control.

4.5.1 Methodology

The methodology used for field vane testing was BS1377: 1990, Part 9, Method 4.4 and as stated previously in Section 3.3.1 the result obtained from an average of at least 5 tests within a 0.5 m radius.

Only GMIV site soils were suitable for testing with the field vane as most other areas in PHG had too much stone content for the tests to be meaningful.

4.5.2 Results

- Field vane strengths within GMIV soils meeting the current specification (refer to Chapter 3), range between 120 and 160 kPa and indicate a very stiff soil. Actual results may exceed 160 kPa as this is the limit of corrected vane strengths for the apparatus.

- Figure 4.13a shows, expected increasing vane strengths with increasing dry density, it also shows shear strength increasing as water content increases, which is opposite to what would be expected in a stiff to very stiff soil. Good correlations were obtained with the Scala penetrometer, which averages approximately 4.5-blows/100 mm, and again indicated a very stiff soil.

- Field vane undrained strengths were not correlative to either undrained unconsolidated or undrained consolidated triaxial test results or shear box results of the same or similar (BRJV F) strength soils. In all cases the vane strength exceeds the triaxial test derived strengths by between 17 and 50 kPa. The Bjerrum
correction was considered but due to the low $I_p$ value of the soil increased $S_u$ by a factor of 1.15 (increasing the disparity between results). Research conducted by Varga and Khan (1992) suggesting field vane $S_u$ for over consolidated clay soils should be related to the (anisotropic) horizontal stress field within the soil rather than the conventionally assumed isotropic stress conditions deserves consideration. Varga and Khan found that by correcting for the $K_0 > 1$ case good correlations were achieved between field vane and triaxial $S_u$ values but that further work was required to validate this theory.

4.5.3 Limitations

Limitations concerning use of the shear vane are mostly concerned with the over estimate of shear strength. Because of the small size of the vanes any contact with a hard object (coarse sand, stone, root) will tend to give an anomalous strength reading.

In addition it is important that a slow constant rate of shear is applied to the vane as an increase in the rate of shearing tends to have a forcing effect on the blades and results in a relative decrease in $S_u$.

All soils tested with a field vane under go a margin of disturbance during insertion of the vane head, research by Chandler (1988) suggests that for sensitive clays shear strength could be underestimated by as much as 25 % due to the destruction of soil structure and a related increase in pore pressure around the vane.

Also the vane must be calibrated regularly to ensure that the change in spring tension over time is corrected to the dial readings.
Figure 4.14a: Comparisons between the 19 mm field vane, Scala penetrometer and compacted dry density.

Figure 4.13b: Comparisons between the 19 mm field vane, Scala penetrometer and compacted moisture content.
4.6 Scala Penetrometer testing

Scala penetrometer testing is carried out as a matter of routine during fill testing in an effort to assess the strength of the fill, and used in conjunction with air voids, moisture and density relationships for quality control.

4.6.1 Methodology (NZS4402: Test 6.5.2, 1988)

The Scala penetrometer test consists of forcing a graduated steel rod tipped by a 20 mm diameter cone into the ground by dropping a 9 kg weight over a distance of 510 mm. The purpose of the test is to give an approximation of the effective friction angle (\(\Phi'\)) and serviceability limits (allowable bearing capacity (\(q_a\)) of a soil. Correlations are based on the curves developed by Stockwell. A reproduction of Stockwell’s correlation curves is presented in Appendix D. It is generally accepted that Stockwell’s results are on the conservative side.

4.6.2 Results

Generally with the exception of GMIV soils the Scala has proved ineffective for the purpose of fill testing and does not give repeatable/reliable results in gravel soils. Results are almost always adversely affected by cobbles and refusal is often achieved within the top 200 mm of fill. As previously covered in Section 4.4 Good correlations between measured density and field vane results were achieved within GMIV soils (Figure 4.14a and 4.14b).

Although Fill test results (Appendix C) clearly indicate the limitations of the Scala as a stand-alone tool for assessing fill, it has proved useful for identifying areas of poorly placed or unsuitable fill at both BRJV and GibAir sites. Table 4.2 displays data extracts of scala fails from the BRJV site.
In every instance these weak areas were checked by excavation and poor fill practice was apparent. Examples of these poor practises included, placement of layers well in excess of what plant could reasonably be expected to compact (up to 0.8 m in one case) and wet compressible organic soils that should have been cut to waste.

4.7 Discussion and Synthesis

- A summary of the engineering properties of typical PHG soils is shown in Table 5.1 it can be seen that the effective stress parameters for the soil have a considerable range between $\Phi'$ 30° (BRJV B and GMIV) to $\Phi'_{\text{est}}$ 38° (GIB B). The upper bound figure is likely to be accurate for the GIB B soil type provided compaction can attain a relative density of $\geq$ 80 % (SPT N= > 39). This standard of compaction is very hard to achieve with plant equipment less than 20 tonne.

- The lower bound $\Phi' = 30^\circ$ relates well to the observed field qualities of the GMIV soil and is within what can reasonably be expected for a silt dominant soil. However the $\Phi' = 30^\circ$ for sample BRJV F does not seem reasonable and is likely to seriously under-represent the true strength of the soil. As mentioned in Section 4.1.2 this result is likely to have been dominated by the silt and clay portion of the soil. The reason for this apparent dominance is directly related to the high fines content identified during gradation testing and the scalping of the soil to AP 19 mm for the triaxial test; effectively removing 20 % of the gravel content of the soil. Research into the effect of gravel content on the strength of compacted composite clay soils by Jafari and Shafiee (2004) clearly demonstrated an increase in $\Phi'$ values of approximately 3°/20 % increase in gravel content. Using this study as the basis for making a correction BRJV F is likely to have an actual $\Phi'$ equivalent to 33°. This value is more consistent with actual field observations for this material.
• The Gib F sample result is likely to be correct albeit with an anomalous apparent $c'$ value. This apparent cohesion can be addressed in terms of critical state soil mechanics theory, which acknowledges that plastic soils in a fully saturated effective stress state are not cohesive and that the apparent cohesion often exhibited is the result of negative pore water pressures stiffening the soils prior to reaching a state of full saturation. The independent behaviour of pore water pressures exhibited at low confining stresses during triaxial loading tend to indicate that this was the case for this sample.

• Viewing effective stress parameters in relation to PSD curves it would be reasonable to assume $c' 3$ kPa and $\Phi'$ 35° for the BRJV B type soils.

• Shear box derived undrained unconsolidated shear strength values for soils compacted to at least 75 % of maximum dry density appear to give good correlation to effective stress parameters indicated in Moore’s (1991) field assessment charts for both non-cohesive and cohesive soils. Further detailed study would be useful to further establish this indicative relationship.

• Laboratory strength testing on soils scalped at AP 19 mm should not be accepted without reference to field observations and soils index testing. As field observations provide a reference to the reliability of the result, while the index properties of the soil give clear indications as to the likely behaviour of the soil, both on loading and when approaching failure.

• Field vane and Scala penetrometer testing provide useful indicators of the insitu $S_u$ and $q_{ull}$ of soils in the GMIV field area. Further work is required to determine whether the use of a horizontal stress field correction proposed by Varga and Khan (1992) is applicable to these soils.
• Field vane and Scala penetrometer testing for other PHG soil types does not provide repeatable results and are not recommended for use in determining fill quality. However the Scala has still proved useful in identifying areas where substandard fill materials or fill placement has occurred.

• Field permeability testing indicates that BRJV soils have very low permeabilities and are likely to be suitable for use in water retention and detention structures provided sufficient quantities of material can be sourced close to site. Field permeability results were successfully indicated from the index and classification testing covered in Chapter 2. The good correlation between field permeability and empirical charts relating to laboratory index testing give considerable confidence for relying on these relatively inexpensive test methods for preliminary design purposes.
CHAPTER 5 Engineering Use & Fill Construction Issues For PHG Soils

5.1 Introduction
Understanding how to apply the results of both laboratory and field-testing to the construction and design of an earth fill structure is critical to achieving a successful engineering outcome. It is important to realise the limitations of laboratory test results conducted on a small sub-sample of the soil, and how test assumptions, methodology and sample selection relate to the actual soils and equipment encountered or utilised in the field. For the most part these interrelationships are empirical and are gained by experience from working with the soils, and by observing and recording their specific behaviours under a range of conditions over a period of time. Standard empirically derived tables have been developed (e.g. USBR engineering use and properties tables, 1988; and the British Specifications for Highway Works, 1992) in order to provide guidelines for earth fill construction and to relate the engineering properties of different soils to appropriate earth structures and the necessary construction equipment.

The construction of similar tables for PHG soils was one of the goals of this thesis. Tables 5.1 and 5.2 provide a synthesis for the engineering use and properties of PHG soils, and information on minimum compaction plant and effort for these soils. As can be seen in Table 5.2, PHG soils, are very versatile and capable of being utilised for a range of different applications. However, with the exception of bulk fill or very small detention/retention dams, use is heavily constrained by the availability of material in sufficient quantity. Because of the fluvial origins of the soil, heterogeneity is the rule rather than the exception, and the ‘typical’ PHG soil is as varied as their potential applications.
5.2 Material Workability

Workability is the term used to describe the ease with which a soil can be economically won, conditioned and placed to a prescribed set of standards (earthworks specification).

I have made the following observations of the workability of the five dominant PHG soil types described in the previous chapters:

- The GM-GC and GM soils are generally good to work with, are relatively abundant and, have natural moisture contents that usually fall within the allowable placement range for the soils. Large cobbles are abundant within these soils, however, and compaction plant below 13 tonnes gross weight has difficulty achieving compaction targets. Maintaining soil moisture in stockpiles during summer months is very difficult, and the GM soils tend to become quite powdery which can present layer separation problems if not adequately managed. In addition the silt is able to be (wind) blown off site, and needs to be addressed by the site erosion and sediment control plan. In winter slope washed silts (Figure 5.1) are often identified along the fill margins, and have been up to 0.3 m deep at times. These saturated loose silts tend to quicken rapidly, and need to be removed from the fill at the earliest opportunity, a flat bladed digger bucket has been used to good effect for this purpose. Because of the highly to completely weathered condition of the gravel clasts, oversize material can be allowed in the fill as this tends to break up easily during compaction.

- Site accessibility to non 4 wheel drive vehicles especially in the BRJV and GibAir areas is restrictive, and as a result of this access difficulty it has not been possible to utilise a water cart on site for much of the project. A water hose has been able to tap water mains adjacent to finished stages of the BRJV subdivisions to allow wetting of soils in the ridge borrow areas, but dust is becoming a problem as more subdivision stages near completion.
Figure 5:1. Slope derived silts bilt up on the fill platform of GibAir Dam following a fifty-year rain storm.

- GM-GP and GP soils are especially difficult to place to specification. The GM-GP soils tend to be very moisture sensitive, and are easy to saturate if care is not taken when adding water. Where possible these soils are mixed with GM-GC soils to improve their workability and placement characteristics. The GM-P and GP soils often contain fresh gravel clasts that do not break down easily during compaction, and additional care must be taken not to include oversize material in the fill. Silty sandy gravel soils tend to be susceptible to surface water erosion, and un-grassed batters tend to deteriorate quickly during periods of extended or high intensity rainfall (Figure 5:2). Erosion tends to terminate against layers of clay rich fill. I have not seen rill erosion on a fill batter exceeding 0.4 m vertical
thickness, and the average would be between 0.1 m and 0.2 m depth. Rilling has not been observed on well-grassed batter slopes in these materials (Figure 5:3).

Figure 5:2. Surface water erosion on the un-grassed downstream batter of GibAir Dam. The rills are about 0.4 m deep and terminate against a clayey gravel layer. Luckily this batter had been purposely over-filled to allow for some deterioration during construction, and prior to being cut back and seeded.

• ML-CL soils at or slightly above OMC are easy to place and attain good strength characteristics, however where these soils become dry they do not compact well and are characterised by excessively high air voids. It is very difficult to get the moisture back into the soil body (as opposed to the soil surface) once excessive drying has occurred. Care must be taken when stockpiling fill during cutting operations to ensure excessive drying is not allowed to occur. Conversely if
allowed to saturate the soil is equally difficult to dry within a reasonable time frame. Both of these scenarios occur regularly at the GMIV site and has been the cause of many construction hold-ups over the past three years.

**Figure 5:3.** Cross section view of the previously rilled slope shown in Figure 5:2 after cutting back and grassing.

5.3 Fill Placement

Establishing a methodology that is consistent and allows concurrent activity to occur is the key to a successful earthworks programme. Good communication between parties at the start of a project usually leads to good fill practice and good quick resolution of problems should they arise. The following paragraphs contain observations I have made at various PHG sites around Nelson.
Maintaining good control of loose fill layer thickness is critical to a good filling operation. Good control over layer thickness allows problems associated with variations in moisture content to be overcome (to a degree) by varying the compactive effort by a known amount. Layer thickness is probably the easiest variable to control, but is often the most ignored. Placing fill layers that are too thick has been the causes of many failed fill tests, and it is worth discussing placement methodology and establishing the necessary field compaction methodology with the contractor at the start of earthworks. Generally speaking loose 200 mm thick layers give optimum results for all but the coarsest PHG soils.

- Careful attention needs to be paid to haul roads where they ramp onto the fill platform. Most contractors are of the opinion that if a 35 tonne dump truck has been over it a couple of hundred times it is compacted, and they will attempt to construct the fill around and over the ramp. These areas should be targeted regularly during monitoring inspections to ensure they aren't forgotten and create a weak area in the fill foundation.

- On sloping sites fill must be placed in horizontal layers and not rolled out along the grade of the slope, as the full compactive force of the equipment is not utilised under these circumstances.

- Poorly prepared fill foundations have been observed in various previously developed subdivisions in Nelson. Although the fill itself often appears to be well placed, the underlying ground surface has not been properly stripped and benched, and variable thicknesses of soft compressible organic soils are visible. Where these conditions are exposed in cuts they often lead to failure of the cut face and extra time and cost to the client to remedy the problem.
• Relaxation of both fill and natural PHG soils into retained back fill behind basement foundation retaining walls has been observed. In most cases the houses affected are on hill slopes with relatively high natural water tables. The relaxation failure appears to be related to soil shrinkage, at least partially related to drying of the soil below its natural water content. In most cases this relaxation does not affect the serviceability limits of building foundations. However where footings are in close proximity to and located above a retaining wall, remedial measures may need to be considered.

5.4 Future Implications for PHG Fill Operations in Nelson

5.4.1 Construction and monitoring

In general terms fill monitoring procedures in Nelson are of a very good standard; areas that require improvement are:

• The initial level of training received by new inexperienced staff prior to being given the responsibility of monitoring earthworks is inadequate. This is at least partly due to staff workload commitments. But implementation of a site works mentoring programme and series of presentations to graduate employees would be beneficial.

• Some minor adjustments will need to be made to the current Earthworks specification for PHG soils, specifically setting hard upper moisture limits for both cohesive and non-cohesive soil types.

• Use of the air voids criteria for assessing the effectiveness of compaction should not be used without supporting strength testing (eg. Scala penetrometer and field shear vane). The air voids criteria, although generally the most suitable method of assessment for PHG soils, is probably not as effective when used with fresh, poorly graded, sandy gravel soils. For these soil types the dry density or relative
density criteria should be used. Field trials will need to be conducted in order to set optimum performance criteria regarding compaction effort, layer thickness and possibly percent dry density versus SPT N values.

- Greater on site involvement is required at the beginning of earthworks. Providing guidance to the contractor on placement methodology along with the conduction of field trials, is crucial to developing good fill construction practices, and will help with meeting completion targets for a project.

5.4.2 Design Practices
A number of design-related issues have been identified as a result of the laboratory index testing, strength testing and field observations, these are:

- The propensity of silty and sandy soils to develop rills following high intensity rainfall events, requires specific attention be given to control surface water run-off. Reducing the maximum grade (fall) on the fill surface during construction, and providing controlled outlets away from unprotected batter faces, should be considered. The critical surface water, flow velocity needs to be calculated for all fills exceeding 10 m vertical height to help determine benching and surface drainage requirements.

- The marked increase in fines that occur in PHG soils as a result of the compaction process (indicated in the PSD plots in Section 2.3) has direct implications for the design of graded filters and design permeability for an earth structure. All design assumptions should be based on either the average or lower bound characteristics of the compacted rather than the natural soil.

- The plastic limits testing revealed the presence of high activity clay soils along the Port Hills Syncline, where small residential earth fills are intended to support
foundations above building retaining walls (e.g. basements and garages) consideration needs to be given in design to the relaxation of the soil into the wall backfill as a result of soil drying over time.

- Triaxial tests indicate that total stress analysis should probably not be considered for design for any structure exceeding 200 kPa/m² effective vertical stress, as increased settlements occur above this pressure and the long term stress state in the soil is not likely to remain constant. The same applies to fill design, in cases where similar load is placed on the fill foundation.

- The current lower bound $c' = 6$ and $\Phi' = 36$ parameters need to be revised downward to $c' = 3$ and $\Phi' = 33^\circ$ in light of the triaxial test results. The present design effective friction angle of $\Phi' 36$, which is the upper bound parameter of the coarsest soil presently in regular use.

5.5 Port Hills Gravel Fill Placement Recommendations

5.5.1 General Fill Material
The following procedures are recommended for the placing of general fill for use as a controlled compacted fill:

<table>
<thead>
<tr>
<th>The fill shall comply with the following criteria and shall be approved by the Engineer:</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\bullet$ Shall be a well graded soil*</td>
</tr>
<tr>
<td>$\bullet$ Shall not include high plasticity soils with a plastic limit exceeding 50% **</td>
</tr>
<tr>
<td>$\bullet$ No particles greater than 300 mm and no more than 2% exceeding 150 mm</td>
</tr>
<tr>
<td>$\bullet$ No organic matter</td>
</tr>
<tr>
<td>$\bullet$ If the insitu moisture content of fill material falls outside the optimum moisture content by ±10 it shall be deemed unsuitable.</td>
</tr>
</tbody>
</table>

Notes: * This can be relaxed if the Engineer determines grading to be not critical to the design. ** For specific purposes, the Engineer may relax this criterion.
5.5.2 Fill Placement

5.5.2.1 General

No fill shall be placed until the foundation over which it is to be placed has been inspected and approved by the Engineer.

Prior to compaction, all fill material shall be broken into fragments of less than 100 mm. The material shall be spread uniformly in layers of less than 200 mm thickness, and conditioned to appropriate average water content.

New fill shall not be spread over surfaces that have deteriorated from their specified condition, and where necessary, the old surface shall be scarified, conditioned and recompacted before placing new fill.

In the event of fill operations ceasing in any area on account of weather or for more than two days for any reason, the Contractor shall obtain the Engineer’s approval of the condition of the fill surface before recommencing filling operations.

The Contractor shall exclude all organic matter from fills, unless the Engineer has given prior written approval.

5.5.2.2 Equipment

The Contractor shall employ sufficient compaction equipment to achieve the specified compaction. The type of plant shall be confirmed by trials carried out to the satisfaction of the Engineer. No subsequent changes shall be introduced without the prior approval of the Engineer.

5.5.2.3 Control of Water Content

When soil is to be dried the Contractor shall disc the soil and allow it to dry uniformly to its full depth.

When the soil is to be wetted, this shall be done with sprinkling equipment ensuring uniform and controlled distribution of water in conjunction with blading and discing.
5.5.2.4 Compaction Requirements

The maximum relative dry density will be determined by the methods of NZS 4402 where these are appropriate.

The compaction requirements for fill material shall be as follows:

General Fill

(i) Cohesive material:
- Average field vane strength over 10 consecutive readings shall not be less than 140 kPa with no individual reading less than 120 kPa;
- The number of blows per 100 mm to drive the Scala penetrometer from a depth of 100 to 300 mm below the fill surface shall be not less than six;
- The average air voids over 10 consecutive tests shall not be greater than 6% with no individual reading over 8%.

(ii) Granular material:
- The number of blows per 100 mm to drive the Scala penetrometer from a depth of 100 to 300 mm below the fill surface shall be not less than ten;
- The average air voids over ten consecutive tests shall not be greater than 8% with no individual reading over 10%.*

Notes: *If poorly graded soils are used compacted fill shall achieve at least 95 % of heavy compaction maximum dry density or 80% relative density.*

The base of any excavation prepared for filling shall also be compacted to the relevant standard specified above for fill. If this surface fails the above criteria, or contains organic or other unsuitable material, undercutting to a depth specified by the Engineer shall be required.
5.5.2.5 Compaction Trials

Prior to the commencement of filling, the contractor shall, in conjunction with the Engineer select representative samples of each type of material to be used as fill and on instruction from the Engineer, arrange laboratory testing for each soil type as identified by the Engineer.

Before the filling operation begins, the Contractor shall demonstrate to the Engineer the suitability of the equipment to be used by spreading and compacting a minimum of three individual superimposed layers of soil (up to 200 mm thickness before compaction) in which tests to confirm the standard of compaction shall be conducted.

The required standards of compaction shall be as defined in Clause 7.0. During the compaction trials the Contractor may develop, in conjunction with the Engineer, ad hoc tests, which the Contractor may use itself as an approximate guide to the standard of compaction being achieved at any time.

The Engineer shall determine how many compaction trials shall be undertaken, reflecting the variety of material to be placed as fill.

Should, in the opinion of the Engineer, differing kinds of soil be uncovered during the course of subsequent work, further compaction trials shall be conducted at the direction of the Engineer.

5.5.3 Testing

The Engineer may carry out control tests of compaction at any time. The Contractor shall stop or divert machines as required by the Engineer to allow the tests to be carried out.

Where field tests indicate that the specified standard of compaction has not been achieved, the Engineer may order stopping of work and/or removal of the fill, dependant on the nature of the fill concerned.
At any time either prior to or during the fill construction, the Engineer may direct modifications to the compaction criteria, for the purpose of ensuring that the optimum compaction criteria for the particular materials and conditions being encountered or likely to be encountered are achieved.

As soon as the Engineer is satisfied those materials are consistent and work is being carried out in a systematic and consistent manner, the Engineer may reduce the frequency of testing as he considers being appropriate. If any test results indicate material has not been placed according to the specification, the area and depth of the fill layer affected is to be reworked under the direction of the Engineer. The reworked fill is to be retested and assessed as meeting the specification requirements before any further placement of new fill may resume.

5.5.4 Applicability
The fill placement guidelines recommended in Section 6.2 are intended as a general placement specification and it will require further amendments if it is to be included as part of a contract document. In particular site and material specific specifications should be added as should detailed testing information (ex. types of test and relevant standards etc.)
Table 6.1: Engineering Properties for Port Hills Gravel Formation soils.

<table>
<thead>
<tr>
<th>Sample Id</th>
<th>USCS Classification</th>
<th>Field K</th>
<th>Laboratory dry density (kN/m³)</th>
<th>Average Field dry density (kN/m³)</th>
<th>Maximum (φk)</th>
<th>Pmax Measured</th>
<th>Lc</th>
<th>Pf</th>
<th>Pf</th>
<th>Pf</th>
<th>Linear (Drainage)</th>
<th>D20</th>
<th>Gradings</th>
<th>Consolidated</th>
<th>Consolidated</th>
</tr>
</thead>
<tbody>
<tr>
<td>GM-N</td>
<td>ML-CL</td>
<td>1.81</td>
<td>16.6</td>
<td>1.88</td>
<td>14.6</td>
<td>14.5</td>
<td>2.87</td>
<td>50.56</td>
<td>41.45</td>
<td>7.11</td>
<td>33</td>
<td>0.037</td>
<td>3.0 kPa</td>
<td>2.0 kPa</td>
<td>2.0 kPa</td>
</tr>
<tr>
<td>BRU-V</td>
<td>GM</td>
<td>1.65</td>
<td>9.53</td>
<td>2.13</td>
<td>7.5</td>
<td>8.4 - 8.7</td>
<td>2.7</td>
<td>38.40</td>
<td>26.30</td>
<td>8.12</td>
<td>8</td>
<td>0.020</td>
<td>Wall Graded</td>
<td>32% est.</td>
<td>10 kPa</td>
</tr>
<tr>
<td>BRU-V-F</td>
<td>GM-GC</td>
<td>2.05</td>
<td>12.8</td>
<td>2.19</td>
<td>8.6</td>
<td>12.5 - 14.4</td>
<td>2.67</td>
<td>39.29</td>
<td>20.24</td>
<td>13.17</td>
<td>7</td>
<td>0.096</td>
<td>10% coarser</td>
<td>32% est.</td>
<td>3.0 kPa</td>
</tr>
<tr>
<td>GIB</td>
<td>GM</td>
<td>2.21</td>
<td>10.6</td>
<td>2.21</td>
<td>9.18</td>
<td>10.5</td>
<td>2.2</td>
<td>29.93</td>
<td>11.15</td>
<td>18.20</td>
<td>7</td>
<td>0.010</td>
<td>10% coarser</td>
<td>32% est.</td>
<td>32% est.</td>
</tr>
<tr>
<td>GIB</td>
<td>GS-GM</td>
<td>2.21</td>
<td>10.6</td>
<td>2.21</td>
<td>9.18</td>
<td>10.5</td>
<td>2.2</td>
<td>29.93</td>
<td>11.15</td>
<td>18.20</td>
<td>7</td>
<td>0.010</td>
<td>10% coarser</td>
<td>32% est.</td>
<td>32% est.</td>
</tr>
</tbody>
</table>

For GIB F: all test results were fail and borrow PSD curve indicated material was outside specification and borrow was reclassified. Max Poly attained was 2.065 at 7.9% M but alloids were 8.61. Est = estimate.

Table 6.2: Engineering use table for Port Hills Gravel Formation soils.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Bulk fill</th>
<th>Base course</th>
<th>Impermeable core</th>
<th>Impermeable liner</th>
<th>Den Embankment</th>
<th>Free Draining Back Fill</th>
<th>Workability</th>
<th>Minimum Plant (mm)</th>
<th>Minimum no. of Passes</th>
<th>Key Control Parameters</th>
<th>Max' layer</th>
<th>Compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean Gravel (ML-CL)</td>
<td>2 to 3</td>
<td>5</td>
<td>2&quot;</td>
<td>2&quot;</td>
<td>5</td>
<td>5</td>
<td>Fair</td>
<td>6</td>
<td>6</td>
<td>Air voids / Strength</td>
<td>200 nm</td>
<td></td>
</tr>
<tr>
<td>Silty Gravel (GM-GC)</td>
<td>1</td>
<td>4</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>5</td>
<td>Very good</td>
<td>10</td>
<td>10</td>
<td>Air voids / Strength</td>
<td>200 nm</td>
<td></td>
</tr>
<tr>
<td>Clean Gravel (GM)</td>
<td>1</td>
<td>3</td>
<td>2 to 3</td>
<td>3</td>
<td>1</td>
<td>5</td>
<td>Good</td>
<td>13</td>
<td>13</td>
<td>Air voids / Strength</td>
<td>200 nm</td>
<td></td>
</tr>
<tr>
<td>Silty Sandy Gravel (GP-GM)</td>
<td>1</td>
<td>2</td>
<td>5</td>
<td>5</td>
<td>4</td>
<td>3</td>
<td>Fair</td>
<td>21</td>
<td>21</td>
<td>Dry density / Air voids</td>
<td>500 nm</td>
<td></td>
</tr>
<tr>
<td>Sandy Gravel (GP)</td>
<td>4</td>
<td>2</td>
<td>6</td>
<td>5</td>
<td>5</td>
<td>2</td>
<td>Poor</td>
<td>21</td>
<td>21</td>
<td>Determine by Compaction</td>
<td>Relative Density</td>
<td></td>
</tr>
</tbody>
</table>

Notes: (1) The fill suitability scale (numbers 1 to 5) is to be read as 1 being most suitable for purpose and 5 being unsuitable for purpose. (2) The soil shows considerable expansion characteristics so would require careful placement methodology if it were to be used. (3) For non-linear cohesive fills a steep toe 'tamping roller' is to be used.
CHAPTER 6 SUMMARY & CONCLUSIONS

6.1 PHG Soil Properties

6.1.1 Engineering Geology

PHG soils are a heterogeneous mix of clayey silty and sandy gravel. They are generally well graded, have high to moderate fines content and exhibit a range of engineering properties.

A continuum of four distinctive PHG soil types was identified; these soils are easy to visually identify in the field and comprise;

- Clayey - silt with gravel;
- Clayey - gravel;
- Silty - gravel;
- Sandy - gravel.

These soil types reflect the weathering processes they have been subjected to, and these differences were reflected in laboratory index tests, triaxial testing and field permeability testing. In general terms PHG soils are well graded, and have a low plasticity to high plasticity ($L_{pl} = 52$) fines portion. The sandy-gravel soil is non-plastic, is poorly graded, and has high permeability.

6.1.2 Index Properties

- The average effective soil strength parameters derived from index testing GMIV soils =$\left(\Phi^c \text{ of } 32^o \text{ to } 34^o\right)$ and GibAir $F = \Phi^c$ of $33^o$ to $38^o$ compares very well to the results obtained through direct triaxial compression ($30^o$ to $36^o$) testing and as such can be utilised for design purposes for PHG soils, provided lower bound values are used.
- Index testing also inferred low permeability for all silty and clayey soil types, the indexed range of permeability was of the order $> 10^{-2}$m/sec. This compared fairly
well to field permeability of $3 \times 10^{-7}$ and $3 \times 10^{-8}$ m/sec, derived from the barrel permeability tests.

- Index testing of soils should be carried out routinely at the start of any new stage of earthworks or changes of borrow area, to confirm field observation and design assumptions.

- Plasticity tests on PHG soils of granitic origin contain clay of high activity (3.8 to 7.5), while those clays of siltstone origin are normal (0.9). The effective particle size for most PHG soils lay within the silt size range and this becomes the dominant material controlling the strength of the soil under triaxial compression. This occurs because it is easier to shear through a fine soil with minimal asperities (especially once pore pressure induced dilation occurs) than it is to break through a coarse gravel or small cobble clast.

6.1.3 Compaction

- The New Zealand Standard Compaction Test provides soil densities and moisture contents that are mostly comparable to field results, provided the stone correction was applied to the test results. Without the stone correction it is hard to justify the use of standard compaction in any PHG soil with more than 30% gravel content. For silty and clayey gravels compacted with a 21 tonne roller values greater than 100% of standard compaction are achieved regularly.

- PHG soils display a wide range of field OMC and maximum dry density results. These results are in the order of:
  - OMC - < 4% to 16.6%
  - Maximum dry densities - 1.86 Mg/m$^3$ to > 2.19 Mg/m$^3$

This reflects the natural and compaction imposed variability of PHG soils.
• It was observed that where completely or highly weathered gravels are present in the fill material the fines content doubled after the soil had been compacted. As a result design assumptions for earth structures utilising PHG soils, need to design for the expected or known properties of the compacted soil.

• It has been determined that air voids should not be used as the key compaction standard for poorly graded sandy gravel soils as high air voids will almost always occur due to the coarse nature of the soil. For these poorly graded soils a dry density or relative density specification needs to be determined.

• A typical PHG fill material (silty or clayey gravel) compacted to the existing specification is likely to have good strength, low air voids, low compressibility and Low permeability (in the order of $3 \times 10^{-8}$m/sec).

• High plasticity gravelly silt soils should not generally be used for earthfill intending to support load-bearing structures. These soils exhibit high shrink/swell characteristics and may cause some bearing capacity failure.

• Differences in the compaction characteristics of PHG soils are primarily the result of chemical weathering and erosion of the soil, the original depositional environment (eg. river bed, oxbow lake, swamp) of the deposit, the break up of larger clasts during the compaction process and poor fill placement control.

6.1.4 Strength Testing
Triaxial testing indicates that lower bound effective stress parameters for a typical PHG soil are likely to be about 3\(^{\circ}\) less than previously assumed. As a result it is recommended
that at least one set of consolidated undrained triaxial tests should be obtained for the weakest material identified on site at the start of each new project.

The effective stress parameters for PHG soils are:

- Clayey silt – $\Phi' = 30^\circ$ and $c' = 7$ kPa
- Silty gravel with clay $\Phi' = 33^\circ$ and $c' = 0$ kPa
- Silty-sandy gravel – $\Phi' = 36^\circ$ and $c' = 26$ kPa

Testing showed that an increase in the gravel content of PHG soils leads to an increase, in both the shear strength and pore water pressures within the soil body. It is recommended that fill be mixed in the borrow area to distribute gravel more evenly through the fill, this will decrease the risk of layer separation and elevated pore water pressures within the fill platform.

6.1.5 Fill Testing

The current fill testing methodology, as in Nelson is robust and performance criteria for fill monitoring provide good control of fill strength. Further work is required to ascertain a suitable standard of compaction for poorly graded sandy gravel soils.

Recommendations for fill placement were made in Section 5.5.
APPENDIX A

Field investigation data

- Cut Face Log 1;

- Test pit excavation logs (TP1 and TP7/06);

- Bore Hole 4/3
**EXCAVATION LOG**

**PROJECT:** Green Meadows
**LOCATION:** Stages 6 & 7, Stove, Nelson
**JOIN NO:** 16533-004

**CO-ORDINATES:** Refer to Site Plan

**EXPOSURE TYPE:** TP
**HOLE STARTED:** 22-11-2006
**EQUIPMENT:** T & T Newman
**HOLE FINISHED:** 22-11-2006
**OPERATOR:** Rod Thompson
**EXCAVATION DIMENSIONS:** 900 x 1300 x 2600
**CHECKED BY:** Llyw
**LOGGED BY:** JMV
**DATE:** 1-12-2006

**EXCAVATION AND TESTS**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (m)</th>
<th>Classification Symbol</th>
<th>Soil Name, Plasticity, Colour, Texture</th>
<th>Exposed And Minor Components</th>
<th>Moisture Condition</th>
<th>Particle Size Characteristics, Colour, Secondary And Minor Components</th>
<th>Geological Structure</th>
<th>Origin Type, Mineral Composition, Detectors, Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>PP</td>
<td>0.5</td>
<td>OL</td>
<td>Silt, low plasticity, dark brown, organic, silt, occasional fine gravel</td>
<td>M</td>
<td>F</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>OL</td>
<td>Silt, low plasticity, black organic</td>
<td>M</td>
<td>F</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>OL</td>
<td>Clayey gravel, loosely packed, creamy grey &lt; yellow</td>
<td>M</td>
<td>F</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>OL</td>
<td>Subrounded, fine with occasional cobbles in a low plasticity silt-clay matrix</td>
<td>M</td>
<td>F</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>OL</td>
<td>Subrounded, rounded, fine cobbles in a low plasticity clay matrix</td>
<td>M</td>
<td>F</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.0</td>
<td>OL</td>
<td>Ergo clay, low plasticity orange brown weathered grey with subrounded fine gravel, chaotic structure into clay matrix, highly weathered, grey, brown, weathered, fine cobbles in a low plasticity clay matrix</td>
<td>M</td>
<td>F</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**SKETCH**

1. Topsoil (Silt?)
2. Buried Topsoil
3. Landslide Descent
4. Debris Flow
5. (Bucket logged)
6. E.O.H 2.5m
   (Hovel digging)
# Borehole Log

**Project:** Green Meadows  
**Location:** Site TN  
**Job No:** 16583.003  
**CO-ORDINATES Refer Site Plan**  
**R.L.**  
**Datum**

## Geological Engineering Description

<table>
<thead>
<tr>
<th>Soil Type, Minor Components, Plasticity or Particle Size, Colour</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLAY, highly plastic (when moist)</td>
</tr>
<tr>
<td>Silt, non-plastic, dark-grey, brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>Silt, non-plastic, dark-grey, brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>Silt, non-plastic, darker grey to brown, with some sub-rounded, fine to medium gravel.</td>
</tr>
<tr>
<td>Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>SILTY CLAY, low plasticity, orange-brown, with some sub-rounded, fine to medium gravel and some organic material. Above gravel is a CLAY, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
</tbody>
</table>

## Soil Tests

<table>
<thead>
<tr>
<th>Soil Tests</th>
<th>Soil Type, Minor Components, Plasticity or Particle Size, Colour</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT/93mm</td>
<td>CLAY, low plasticity, orange-brown, with some sub-rounded, fine to medium gravel and some organic material. Above gravel is a CLAY, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>PP 300</td>
<td>Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>PP 415</td>
<td>Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>PP 525</td>
<td>Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
</tbody>
</table>

## Engineering Description

<table>
<thead>
<tr>
<th>Engineering Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLAY, low plasticity, orange-brown, with some sub-rounded, fine to medium gravel and some organic material. Above gravel is a CLAY, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
<tr>
<td>Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel.</td>
</tr>
</tbody>
</table>

## Borehole Log Sheet

**Borehole No:** BR41/3  
**Drill Type:** HQ, IT  
**Drill Method:** CORE  
**Drilled By:** C.W. drilling (Melcom)  
**Logged By:** L.W.  
**Hole Started:** 25-02-04  
**Hole Finished:**  

## Geological Unit, Generic Name, Origin, Mineral Composition

- Topsoil
- Columbium
- Columbium
- Columbium
- Topsoil

## Soil Tests

- SPT/93mm
- PP 300
- PP 415
- PP 525
- PR 860
- PR 125

## Engineering Description

- CLAY, low plasticity, orange-brown, with some sub-rounded, fine to medium gravel and some organic material. Above gravel is a CLAY, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel. |
- Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel. |
- Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel. |
- Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel. |
- Silt, non-plastic, dark-grey, grey or brown, with occasional fine to medium gravel. |
**Geological Log**

**Project:** Green Meadows  
**Location:** SWAG 4  
**Job No:** 16583-002

**Co-ordinates:** Refer site plan  
**RL:**  
**Datum:**

<table>
<thead>
<tr>
<th>Geological Unit</th>
<th>Description</th>
<th>Soil Description</th>
<th>Rock Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>President</td>
<td>Limestone</td>
<td>As Above with</td>
<td>Clayey Silt with</td>
</tr>
<tr>
<td></td>
<td></td>
<td>coarse gravel,</td>
<td>gravel, orange</td>
</tr>
<tr>
<td></td>
<td></td>
<td>orange, brown,</td>
<td>brown with minor</td>
</tr>
<tr>
<td></td>
<td></td>
<td>subdued clay</td>
<td>subangular fine</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Limestone content is higher than in low plasticity.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Tests:**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Sample</th>
<th>R.L (m)</th>
<th>Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.5</td>
<td>PP 100-250</td>
<td>65.5</td>
<td></td>
</tr>
<tr>
<td>65.5</td>
<td>CLG M</td>
<td>65</td>
<td></td>
</tr>
<tr>
<td>95</td>
<td>CLG M</td>
<td>95</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>CLG M</td>
<td>120</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>MLC G M</td>
<td>150</td>
<td></td>
</tr>
<tr>
<td>180</td>
<td>MLG M</td>
<td>180</td>
<td></td>
</tr>
<tr>
<td>210</td>
<td>MLG M</td>
<td>210</td>
<td></td>
</tr>
<tr>
<td>240</td>
<td>MLG M</td>
<td>240</td>
<td></td>
</tr>
<tr>
<td>270</td>
<td>CLG M</td>
<td>270</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>CLG M</td>
<td>300</td>
<td></td>
</tr>
<tr>
<td>330</td>
<td>CLG M</td>
<td>330</td>
<td></td>
</tr>
</tbody>
</table>

**Engineering Description:**

- **Soil Description:**
  - Soil type, minor components, plasticity or particle size, colour.
  - **Rock Description:**
    - Substance: Rock type, particle size, colour, minor components.
    - Features: Type, inclination, thickness, roughness, filling.
## BOREHOLE LOG

### PROJECT: Green Meadows

**LOCATION:** Stage IV

**DRILL TYPE:** HQ TT

**DRILL METHOD:** Core

**DRILL FLUID:** H₂O

**HOLE STARTED:** 25-02-04

**HOLE FINISHED:**

**DRILLED BY:** C W Drilling (Melcon)

**LOGGED BY:** Unnamed

**CHECKED:**

### GEOLOGICAL

**GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION:**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>SUITABLE GROS</th>
<th>RECOVERY</th>
<th>METHOD</th>
<th>GRADEING</th>
<th>TESTING</th>
<th>LOGS</th>
<th>WATER</th>
<th>ROCK CATEGORIES</th>
<th>SOIL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 11.5</td>
<td>2.2%</td>
<td>93%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>GRAVEL</td>
<td>As above but &gt; ratio of silt &amp; clay</td>
</tr>
<tr>
<td>11.5 - 19.0</td>
<td>PP 1400</td>
<td>11.5%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Siltstone</td>
<td>Silt non plastic, clays brown -</td>
</tr>
<tr>
<td>19.0 - 30.0</td>
<td>SP 1250</td>
<td>12.5%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>GRANUL</td>
<td>Silt gravel &gt; 80%</td>
</tr>
<tr>
<td>30.0 - 40.0</td>
<td>PP 3500</td>
<td>12.5%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CLAY/SAND</td>
<td>Silt 40% Silt, non plastic, orangy brown -</td>
</tr>
</tbody>
</table>

### ENVIRONMENTAL

**SOIL DESCRIPTION:**

- Soil type, mineral composition, plasticity or particle size, colour.
- Rock description:
  - Subtype: Rock type, particle size, colour, mineral components.
  - Degree: Type, fluctuation, thickness, roughness, filigree.
## BOREHOLE LOG

**PROJECT:** Green Meadows  
**LOCATION:** Single Ju  
**JOB No.:** 16553.003

**CO-ORDINATES**  
Refer Site Plan

**R.L.**  
**DATE:**

**DRILL TYPE:** HQ II  
**DRILL METHOD:** Core  
**DRILL FLUID:** H₂O  
**HOLE STARTED:** 26-02-01u  
**HOLE FINISHED:**  
**DRILLED BY:** C W Drilling  
**LOGGED BY:** J W T

### GEOLOGICAL

<table>
<thead>
<tr>
<th>Geological Unit, Generic Name, Origin, Mineral Composition</th>
<th>Engineering Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt Gravel Sequences</td>
<td>Silty gravel, blue grey with minor components, plasticity or particle size, colour.</td>
</tr>
<tr>
<td>Fluvial</td>
<td>Above grades into: Silty, low plasticity, grey, with minor, Ew, subangular medium gravel, s clay</td>
</tr>
<tr>
<td>Clay</td>
<td>Above coarse to sandy fine gravel, continuing to coarsen down hole to a medium gravel with KH, subrounded, subangular clasts.</td>
</tr>
</tbody>
</table>

### SOIL DESCRIPTION
- Soil type, minor components, plasticity or particle size, colour.

### ROCK DESCRIPTION
- Subtype: Rock type, particle size, colour, minor components.
- Massive: Type, inclination, thickness, rock type, filling.
**BOREHOLE LOG**

**PROJECT:** Green Meadows  
**LOCATION:** Stage In  
**JOB No:** 16583-003

**GEOLOGICAL**

<table>
<thead>
<tr>
<th>Geological Unit</th>
<th>Fluid Loss</th>
<th>Core Recovery</th>
<th>Testing</th>
<th>Samples</th>
<th>R.L (m)</th>
<th>Engineering Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Colloquium</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>20.5</td>
<td>CLAYY SANDY CLAY</td>
</tr>
<tr>
<td>(PHC - Devoured)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Potential</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>21.5</td>
<td>CLAYY Silty CLAY</td>
</tr>
<tr>
<td>Silt</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Colloquium</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>22.5</td>
<td>Silty-Grey CLAY</td>
</tr>
<tr>
<td>(PHC - Devoured)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Phc</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
SPT 4/3.
APPENDIX B

Laboratory Data

• Soil gradation - tables and charts;

• Atterburg limits - calculation sheets, Plasticity chart plot and tables;

• Determination of Linear shrinkage calculation sheets;

• Determination of the water content calculation sheets;

• Determination of the solid density of soil

• New Zealand Standard Compaction - tables;

• New Zealand Standard Compaction vs field compaction curves;

• Direct shear box - strain/time and stress/strain plots;

• Triaxial test results – strain/time and stress/strain plots.
Particle size distribution analysis test report.

Reference: 81069.010
Sample: PHG (Silty Gravel)
Sampled from: Borrow 1
Date: 18 February 2005
Test methodology: NZS 4402: 1986 Test 2.8.1 (≥ 0.063 mm),
Laser particle sizer (≤ 0.063 mm)
Tested by: John Westerson on 15 October 2005.
Jennifer Jackson on 05 July 2006.

**Summary**

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>Percent passing (%)</th>
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</thead>
<tbody>
<tr>
<td>100.000</td>
<td>87.80</td>
</tr>
<tr>
<td>75.000</td>
<td>80.60</td>
</tr>
<tr>
<td>63.000</td>
<td>77.80</td>
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<tr>
<td>37.500</td>
<td>71.10</td>
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<tr>
<td>19.000</td>
<td>62.60</td>
</tr>
<tr>
<td>6.300</td>
<td>45.90</td>
</tr>
<tr>
<td>2.000</td>
<td>38.60</td>
</tr>
<tr>
<td>0.600</td>
<td>29.60</td>
</tr>
<tr>
<td>0.425</td>
<td>27.60</td>
</tr>
<tr>
<td>0.212</td>
<td>22.00</td>
</tr>
<tr>
<td>0.063</td>
<td>17.30</td>
</tr>
<tr>
<td>0.020</td>
<td>9.60</td>
</tr>
<tr>
<td>0.006</td>
<td>4.10</td>
</tr>
<tr>
<td>0.002</td>
<td>1.50</td>
</tr>
<tr>
<td>0.001</td>
<td>0.50</td>
</tr>
</tbody>
</table>
Particle size distribution analysis test report.

Reference: 81069.010
Sample: PHG (Clayey Gravel)
Sampled from: Playing field fill platform
Date: 18 February 2005
Test methodology: NZS 4402: 1986 Test 2.8.1 (≥ 0.063 mm), Laser particle sizer (≤ 0.063 mm)
Tested by: John Westerson on 05 November 2005, Jennifer Jackson on 05 July 2006.

### SUMMARY

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>Percent passing (%)</th>
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<tbody>
<tr>
<td>100.000</td>
<td>92.50</td>
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<tr>
<td>75.000</td>
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<tr>
<td>63.000</td>
<td>92.50</td>
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<tr>
<td>37.500</td>
<td>88.10</td>
</tr>
<tr>
<td>19.000</td>
<td>82.30</td>
</tr>
<tr>
<td>6.300</td>
<td>64.60</td>
</tr>
<tr>
<td>2.000</td>
<td>54.30</td>
</tr>
<tr>
<td>0.600</td>
<td>46.70</td>
</tr>
<tr>
<td>0.425</td>
<td>44.10</td>
</tr>
<tr>
<td>0.212</td>
<td>40.30</td>
</tr>
<tr>
<td>0.063</td>
<td>31.70</td>
</tr>
<tr>
<td>0.020</td>
<td>20.50</td>
</tr>
<tr>
<td>0.006</td>
<td>9.90</td>
</tr>
<tr>
<td>0.002</td>
<td>4.00</td>
</tr>
<tr>
<td>0.001</td>
<td>1.40</td>
</tr>
</tbody>
</table>
Particle size distribution analysis test report.

Reference: 870095.010
Sample: PHG (Sandy Gravel)
Sampled from: Borrow 1
Date: 15 March 2006
Test methodology: NZS 4402: 1986 Test 2.8.1 (≥ 0.063 mm),
Tested by: John Westerson on 26 June 2006.

![Particle size distribution curve for GIBAIR B](image)

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>Percent passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100.00</td>
<td>100.00</td>
</tr>
<tr>
<td>75.00</td>
<td>76.70</td>
</tr>
<tr>
<td>63.00</td>
<td>76.70</td>
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<tr>
<td>37.50</td>
<td>54.50</td>
</tr>
<tr>
<td>19.00</td>
<td>34.70</td>
</tr>
<tr>
<td>6.30</td>
<td>15.30</td>
</tr>
<tr>
<td>2.00</td>
<td>9.40</td>
</tr>
<tr>
<td>0.60</td>
<td>6.60</td>
</tr>
<tr>
<td>0.425</td>
<td>6.10</td>
</tr>
<tr>
<td>0.212</td>
<td>5.00</td>
</tr>
<tr>
<td>0.063</td>
<td>3.30</td>
</tr>
<tr>
<td>0.020</td>
<td>0.00</td>
</tr>
<tr>
<td>0.006</td>
<td>0.00</td>
</tr>
<tr>
<td>0.002</td>
<td>0.00</td>
</tr>
<tr>
<td>0.001</td>
<td>0.00</td>
</tr>
</tbody>
</table>
Particle size distribution analysis test report.

Reference: 870095.010
Sample: PHG (Sandy Gravel)
Sampled from: Gibair fill platform
Date: 15 March 2006
Test methodology: NZS 4402: 1986 Test 2.8.1 (≥ 0.063 mm).
Laser particle sizer (≤ 0.063 mm).
Tested by: John Westerson on 27 June 2006.
Jennifer Jackson on 30 March 2007.

### Particle size distribution curve for GIBAIR F

![Particle size distribution curve](image)

### SUMMARY

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>Percent passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100.000</td>
<td>96.00</td>
</tr>
<tr>
<td>75.000</td>
<td>90.30</td>
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<tr>
<td>63.000</td>
<td>81.60</td>
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<tr>
<td>37.500</td>
<td>75.60</td>
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<tr>
<td>19.000</td>
<td>62.90</td>
</tr>
<tr>
<td>6.300</td>
<td>49.70</td>
</tr>
<tr>
<td>2.000</td>
<td>38.20</td>
</tr>
<tr>
<td>0.600</td>
<td>24.60</td>
</tr>
<tr>
<td>0.425</td>
<td>21.50</td>
</tr>
<tr>
<td>0.212</td>
<td>19.90</td>
</tr>
<tr>
<td>0.063</td>
<td>14.40</td>
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<tr>
<td>0.020</td>
<td>10.60</td>
</tr>
<tr>
<td>0.006</td>
<td>6.00</td>
</tr>
<tr>
<td>0.002</td>
<td>2.40</td>
</tr>
<tr>
<td>0.001</td>
<td>0.70</td>
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</table>
Particle size distribution analysis test report.

Reference: 16583.004
Sample: PHG (Silty Clay)
Sampled from: GM Stage IV
Date: 18 February 2005
Test methodology: NZS 4402: 1986 Test 2.8.1 (≤ 0.063 mm), Laser particle sizer (≤ 0.063 mm)

**SUMMARY**

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>Percent passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100.000</td>
<td>100.00</td>
</tr>
<tr>
<td>75.000</td>
<td>100.00</td>
</tr>
<tr>
<td>63.000</td>
<td>100.00</td>
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<tr>
<td>37.500</td>
<td>100.00</td>
</tr>
<tr>
<td>19.000</td>
<td>100.00</td>
</tr>
<tr>
<td>6.300</td>
<td>87.70</td>
</tr>
<tr>
<td>2.000</td>
<td>78.50</td>
</tr>
<tr>
<td>0.600</td>
<td>73.60</td>
</tr>
<tr>
<td>0.425</td>
<td>71.20</td>
</tr>
<tr>
<td>0.212</td>
<td>65.40</td>
</tr>
<tr>
<td>0.063</td>
<td>54.00</td>
</tr>
<tr>
<td>0.020</td>
<td>38.90</td>
</tr>
<tr>
<td>0.006</td>
<td>22.90</td>
</tr>
<tr>
<td>0.002</td>
<td>10.30</td>
</tr>
<tr>
<td>0.001</td>
<td>3.30</td>
</tr>
<tr>
<td>Soil Gradation Results for Port Hills Gravel Formation Soils.</td>
<td></td>
</tr>
</tbody>
</table>
Grain size distribution for Port Hills Gravel

### Weight retained

<table>
<thead>
<tr>
<th>Location</th>
<th>&lt; 63 um</th>
<th>&gt; 63 um</th>
<th>&gt; 212 um</th>
<th>&gt; 425 um</th>
<th>&gt; 600 um</th>
<th>&gt; 2 mm</th>
<th>&gt; 6.3 mm</th>
<th>&gt; 19 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bowl weight (gm)</td>
<td>276.1</td>
<td>159.3</td>
<td>242.5</td>
<td>na*</td>
<td>312.1</td>
<td>279.3</td>
<td>30.1</td>
<td>270.4</td>
</tr>
<tr>
<td>BRJV B</td>
<td>1814.0</td>
<td>757.1</td>
<td>737.9</td>
<td>176.8</td>
<td>1111.1</td>
<td>919.7</td>
<td>1518.6</td>
<td>1024.4</td>
</tr>
<tr>
<td>BRJV B</td>
<td>1537.9</td>
<td>417.8</td>
<td>495.4</td>
<td>176.8</td>
<td>799.0</td>
<td>640.4</td>
<td>1488.5</td>
<td>754.0</td>
</tr>
<tr>
<td>BRJV F</td>
<td>2365.2</td>
<td>729.5</td>
<td>492.8</td>
<td>171.5</td>
<td>812.2</td>
<td>961.4</td>
<td>1198.8</td>
<td>654.3</td>
</tr>
<tr>
<td>BRJV F</td>
<td>2089.1</td>
<td>570.2</td>
<td>250.3</td>
<td>171.5</td>
<td>500.1</td>
<td>682.1</td>
<td>1168.7</td>
<td>383.9</td>
</tr>
<tr>
<td>GM IV</td>
<td>2710.1</td>
<td>673.1</td>
<td>503.9</td>
<td>108.2</td>
<td>533.0</td>
<td>694.0</td>
<td>584.5</td>
<td>0.0</td>
</tr>
<tr>
<td>GM IV</td>
<td>2434.0</td>
<td>513.8</td>
<td>261.4</td>
<td>108.2</td>
<td>220.9</td>
<td>414.7</td>
<td>554.4</td>
<td>0.0</td>
</tr>
<tr>
<td>GIBAIR</td>
<td>1241.2</td>
<td>524.4</td>
<td>349.6</td>
<td>209.8</td>
<td>1223.7</td>
<td>1048.9</td>
<td>917.8</td>
<td>1118.8</td>
</tr>
<tr>
<td>GIBAIR (1)</td>
<td>965.1</td>
<td>365.1</td>
<td>107.1</td>
<td>209.8</td>
<td>911.6</td>
<td>769.6</td>
<td>887.7</td>
<td>848.4</td>
</tr>
</tbody>
</table>

Weight % passing 63um obtained by measurement.

<table>
<thead>
<tr>
<th>Location</th>
<th>63 um</th>
<th>212 um</th>
<th>425 um</th>
<th>600 um</th>
<th>2 mm</th>
<th>6.3 mm</th>
<th>19 mm</th>
<th>37.5 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>BRJV B. (Weight %R)</td>
<td>4.7</td>
<td>5.6</td>
<td>2.0</td>
<td>9.0</td>
<td>7.2</td>
<td>16.8</td>
<td>8.5</td>
<td>6.5</td>
</tr>
<tr>
<td>% passing sieve</td>
<td>17.3</td>
<td>22.0</td>
<td>27.6</td>
<td>29.6</td>
<td>38.6</td>
<td>45.9</td>
<td>62.6</td>
<td>71.1</td>
</tr>
<tr>
<td>BRJV F. (Weight %R)</td>
<td>8.6</td>
<td>3.8</td>
<td>2.6</td>
<td>7.6</td>
<td>10.3</td>
<td>17.7</td>
<td>5.8</td>
<td>4.4</td>
</tr>
<tr>
<td>% passing sieve</td>
<td>31.7</td>
<td>40.3</td>
<td>44.1</td>
<td>46.7</td>
<td>54.3</td>
<td>64.6</td>
<td>82.3</td>
<td>88.1</td>
</tr>
<tr>
<td>GM IV (Weight %R)</td>
<td>11.4</td>
<td>5.8</td>
<td>2.4</td>
<td>4.9</td>
<td>9.2</td>
<td>12.3</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>% passing sieve</td>
<td>54.0</td>
<td>65.4</td>
<td>71.2</td>
<td>73.6</td>
<td>78.5</td>
<td>87.7</td>
<td>100.0</td>
<td></td>
</tr>
<tr>
<td>GIBAIR F (Weight %R)</td>
<td>5.5</td>
<td>1.6</td>
<td>3.1</td>
<td>13.6</td>
<td>11.5</td>
<td>13.3</td>
<td>12.7</td>
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<tr>
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<td>14.4</td>
<td>19.9</td>
<td>21.5</td>
<td>24.6</td>
<td>38.2</td>
<td>49.7</td>
<td>62.9</td>
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<tr>
<td>GIBAIR B (% passing)</td>
<td>3.3</td>
<td>5.0</td>
<td>6.1</td>
<td>6.6</td>
<td>9.4</td>
<td>15.3</td>
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<tr>
<td>&gt; 37.5 mm</td>
<td>&gt; 63 mm</td>
<td>&gt; 75 mm</td>
<td>&gt; 100 mm</td>
<td>Total</td>
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<td>na**</td>
<td>na**</td>
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<td>638.6</td>
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<td>271.0</td>
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<tr>
<td>402.1</td>
<td>585.6</td>
<td>375.9</td>
<td>271.0</td>
<td>6698.8</td>
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<table>
<thead>
<tr>
<th>63 mm</th>
<th>75 mm</th>
<th>100 mm</th>
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<td>7.2</td>
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</tr>
<tr>
<td>77.6</td>
<td>80.6</td>
<td>87.8</td>
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<td>0.0</td>
<td>7.5</td>
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<tr>
<td>92.5</td>
<td>92.5</td>
<td>92.5</td>
</tr>
<tr>
<td>8.7</td>
<td>5.6</td>
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<tr>
<td>81.6</td>
<td>90.3</td>
<td>96.0</td>
</tr>
<tr>
<td>76.7</td>
<td>76.7</td>
<td>100.0</td>
</tr>
</tbody>
</table>
Job: Theobald
Location: Thames, Waikato, New Zealand
Depth(s): 2.3 m below original soil
Test details:
Test performed on fraction passing 425 μm sieve/whole soil.
History: Natural/air dried/oven dried/40°C/unknown
Soil equilibrated with water for 2.5 hr.
Liquid limit machine no. CAE014 601 8

<table>
<thead>
<tr>
<th>Test no.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of test †</td>
<td>( L_L )</td>
<td>( L_L^w )</td>
<td>( L_L )</td>
<td>( P_L )</td>
<td>( P_L )</td>
<td></td>
</tr>
<tr>
<td>No. of blows (liquid limit test)</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>Container no.</td>
<td>26</td>
<td>35</td>
<td>30</td>
<td>60</td>
<td>22</td>
<td>33</td>
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<tr>
<td>Mass of container and wet soil ( M_2 ) g</td>
<td>241</td>
<td>241</td>
<td>19</td>
<td>219</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>Mass of container and dried soil ( M_3 ) g</td>
<td>211</td>
<td>211</td>
<td>18.3</td>
<td>19.6</td>
<td>19.6</td>
<td>20.2</td>
</tr>
<tr>
<td>Mass of container ( M_1 ) g</td>
<td>182</td>
<td>132</td>
<td>13.5</td>
<td>13.5</td>
<td>13.5</td>
<td>13.5</td>
</tr>
<tr>
<td>Mass of water ( M_2 - M_3 ) g</td>
<td>5</td>
<td>5</td>
<td>1.7</td>
<td>1.4</td>
<td>1.8</td>
<td>2</td>
</tr>
<tr>
<td>Mass of dried soil ( M_3 - M_1 ) g</td>
<td>3.4</td>
<td>3.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
</tr>
<tr>
<td>Water content ( w = \frac{M_2 - M_3}{M_3 - M_1} \times 100 )</td>
<td>37.4</td>
<td>36.7</td>
<td>34.7</td>
<td>34.1</td>
<td>26</td>
<td>27.7</td>
</tr>
</tbody>
</table>

Water content ...... %

Liquid limit ...... (36 - 40)
Plastic limit ...... (26 - 30)
Plasticity index ...... 40

* Delete inappropriate words.

† Water content test to be marked \( w \); Liquid Limit, \( LL \); and Plastic Limit, \( PL \).
Form 2.2, 2.3, 2.4  
DETERMINATION OF THE LIQUID AND PLASTIC LIMITS, PLASTICITY INDEX AND WATER CONTENT  
(Tests 2.2, 2.3 and 2.4)

Job:  
Location:  
Depth(s):  
Test details:  
Test performed on fraction passing 425 μm sieve/whole soil  
History: Natural/air-dried/oven-dried/unknown  
Soil equilibrated with water for ... h  
Liquid limit machine no.  

<table>
<thead>
<tr>
<th>Test no.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Type of test †</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>No. of blows (liquid limit test)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Container no.</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Mass of container and wet soil</strong></td>
<td>$M_2$ g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Mass of container and dried soil</strong></td>
<td>$M_3$ g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Mass of container (empty)</strong></td>
<td>$M_1$ g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Mass of water</strong></td>
<td>$M_2 - M_3$ g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Mass of dried soil</strong></td>
<td>$M_3 - M_1$ g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Water content $w =$</strong></td>
<td>$\frac{M_2 - M_3}{M_1 - M_3} \times 100$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Water content ... %  
Liquid limit ... %  
Plastic limit ... %  
Plasticity index ... %  

* Delete inappropriate words.

† Water content test to be marked $w$; Liquid Limit, $LI$; and Plastic Limit, $PL$.
Form 2.2, 2.3, 2.4  
DETERMINATION OF THE LIQUID AND PLASTIC LIMITS, PLASTICITY INDEX AND WATER CONTENT  
(Tests 2.2, 2.3 and 2.4)

Job:  
Location:  
Depth(s):  

Test details:
- Test performed on fraction passing 425 μm sieve/whole soil
- History: Natural/air-dried/oven-dried/ unknown
- Soil equilibrated with water for 2..... h
- Liquid limit machine no.

<table>
<thead>
<tr>
<th>Test no.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of test (t)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. of blows (liquid limit test)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Container no.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass of container and wet soil, $M_2$ g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass of container and dried soil, $M_3$ g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass of container, $M_1$ g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass of water, $M_2 - M_3$ g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass of dried soil, $M_3 - M_1$ g</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Water content $w = \frac{M_2 - M_3}{M_3 - M_1} \times 100$%

Water content ...... %
Liquid limit ...... $LL$
Plastic limit ...... $PL$
Plasticity index $IL = \frac{LL - PL}{100}$

* Delete inappropriate words.
† Water content test to be marked $w$; Liquid Limit, $LL$; and Plastic Limit, $PL$.

Water content (%)  
Number of blows ⊦
Form 2.2, 2.3, 2.4

DETERMINATION OF THE LIQUID AND PLASTIC LIMITS, PLASTICITY INDEX AND WATER CONTENT
(Tests 2.2, 2.3 and 2.4)

Job: Thesis
Location: Green Wards, Nelson
Depth(s): 2

Test details:
- Test performed on fraction passing 425 μm sieve/whole soil
- History: Natural/air-dried/oven-dried/unknown
- Soil equilibrated with water for 24 h
- Liquid limit machine no. 0629

<table>
<thead>
<tr>
<th>Test no.</th>
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<th>4</th>
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<th>6</th>
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</thead>
<tbody>
<tr>
<td>Type of test</td>
<td>LL</td>
<td>LL</td>
<td>LL</td>
<td>LL</td>
<td>LL</td>
<td>LL</td>
</tr>
<tr>
<td>No. of blows (liquid limit test)</td>
<td>26</td>
<td>26</td>
<td>26</td>
<td>26</td>
<td>26</td>
<td>26</td>
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</tbody>
</table>

<table>
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<tr>
<th>Container no.</th>
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<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass of container and wet soil</td>
<td>20.5</td>
<td>20.5</td>
<td>22.2</td>
<td>26.4</td>
<td>21.7</td>
<td>22.1</td>
</tr>
<tr>
<td>Mass of container and dried soil</td>
<td>25.7</td>
<td>25.7</td>
<td>25.7</td>
<td>24.1</td>
<td>25.3</td>
<td>25.6</td>
</tr>
<tr>
<td>Mass of container</td>
<td>20.0</td>
<td>20.0</td>
<td>20.0</td>
<td>20.0</td>
<td>20.0</td>
<td>20.0</td>
</tr>
<tr>
<td>Mass of water</td>
<td>6.2</td>
<td>6.2</td>
<td>6.2</td>
<td>6.2</td>
<td>6.2</td>
<td>6.2</td>
</tr>
<tr>
<td>Mass of dried soil</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>Water content $w = \frac{M_2 - M_3}{M_4 - M_1} \times 100$</td>
<td>53.5</td>
<td>54.4</td>
<td>53.7</td>
<td>51.3</td>
<td>44.4</td>
<td>44.6</td>
</tr>
</tbody>
</table>

Water content ...... %
Liquid limit ...... $LL \approx \frac{w}{(0.5 - 0.6)}$
Plastic limit ...... $PL \approx \frac{w}{(0.1 - 0.15)}$
Plasticity index ...... $PI \approx \frac{w}{(0.1 - 0.15)}$

* Delete inappropriate words.
† Water content test to be marked $w$;
Liquid Limit, $LL$; and
Plastic Limit, $PL$.

4 — 2.3
Sometimes the soil flows so as to leave a gap between two areas of contact. The test shall continue until there is a length of continuous contact for 15 mm.

Some soils tend to slide on the surface of the cup instead of the soil flowing. If this occurs, the result shall be discarded and the test repeated until flowing does occur. If ater additional increments of water sliding still occurs, the test is not applicable and a note shall be made that the liquid limit could not be obtained.

One-point method. If flow curves are plotted on a log-log chart, it is found that their slopes are reasonably constant over a wide range of liquid limit. Thus if a single point on the flow curve close to the liquid limit is accurately determined, the standard slope may be used to calculate the liquid limit. This single point must lie between 20 and 30 blows and the water content must be determined in duplicate. Calculate the liquid limit from the following formula:

\[ LL = w \left( \frac{n}{25} \right)^{0.1} \]

where \( LL = \) liquid limit expressed to the nearest whole number
\( w = \) water content corresponding to \( n \) blows (%)

The main advantage of this method is speed, but it should not be undertaken unless a considerable number of liquid limit determinations have been carried out by the full standard method, and until some skill has been developed in handling soils and the apparatus.

Care shall be taken to see that the sample does not dry out rapidly between repeat tests as the number of blows for closure will increase gradually as the sample dries out. Some low plasticity materials may need to be tested in a humid room to prevent rapid drying.

**Dimensions**

<table>
<thead>
<tr>
<th>Letter</th>
<th>( A^* )</th>
<th>( B^* )</th>
<th>( C^* )</th>
<th>( D^* )</th>
<th>( E^* )</th>
<th>( F^* )</th>
<th>( G^* )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( mm )</td>
<td>±0.5</td>
<td>±0.4</td>
<td>±0.5</td>
<td>±0.5</td>
<td>±0.5</td>
<td>±0.5</td>
<td>±0.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Letter</th>
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<th>( J^* )</th>
<th>( K )</th>
<th>( L )</th>
<th>( M )</th>
<th>( N )</th>
<th>( P )</th>
<th>( Q )</th>
<th>( R )</th>
<th>( S )</th>
<th>( T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( mm )</td>
<td>±0.5</td>
<td>±0.5</td>
<td>50</td>
<td>150</td>
<td>130</td>
<td>27</td>
<td>28</td>
<td>6</td>
<td>22</td>
<td>19</td>
<td>45</td>
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</tbody>
</table>

Rubber base conforming to specification. See Note 2.

This design has been found satisfactory, but alternative designs may be employed provided that the essential requirements are fulfilled. (Essential dimensions are indicated by an asterisk.)

**Fig. 2.2.1**

LIQUID LIMIT APPARATUS

4 — 2.2
Measurements: Mould Dimensions: Length (L) = 150.25 mm Internal width (A) = 25 mm

Sample oven dried @ 110°C

L = 1 - \frac{L_1}{L_0} = 100 - 6.45 = 93.55%

Mould Dimensions: L_0 = 150 mm W_0 = 25 mm

Sample oven dried @ 110°C

L_1 = 1 - \frac{L_1}{L_0} = 100 - 7.65 = 92.35%
SAMPLE SIZE: 30

SAMPLE Dimensions - Dimensions:

Length (L) = 120.5 mm

Width (W) = 25 mm

SAMPLE Oven Design @ 110°C:

L = 1 - \frac{L_0}{L_0} = 111.2 mm
W = 25 mm

Length (L_0) = 120.5 mm
Width (W) = 25 mm

Volume (V) = \frac{L \times W \times T}{1000} = 22.7 ml
2.6.6.2 State whether the material used in the test was whole soil, or fraction passing a 425 μm test sieve.

2.6.6.3 State the history of the sample, for example natural state, air-dried, oven-dried or unknown.

2.6.6.4 State that the result was obtained in accordance with this Standard Test Method.

NOTES ON TEST 2.6

1. The mould may conveniently be made from 3 mm thick brass tube of 25 mm internal diameter cut to form a semi-cylindrical trough. The ends of 6 mm flat brass are brazed on normal to the longitudinal axis.

2. When the sample is tested in the liquid limit device, the groove should close with between 15 and 25 blows.

3. If a specimen cracks or curls badly, so that measurement is difficult, repeat the test at a slower drying rate.

All dimensions in millimetres.

These dimensions have been found to be satisfactory, but other sized moulds may be used provided the length is between 100 and 150 mm.

Ends of trough to be cut square and end blocks fixed on square to trough axis.

Material to be non-corrosive metal.

Fig. 2.6 LINEAR SHRINKAGE MOULD

2 — 2.6
Test Method Used: 4428 2.11.21 786 Test 2-2

Whole Soil.

Sample ID: 00135

\[ W_1 - \text{container} = 276.1 \text{ g} \]
\[ W_2 - \text{wet soil} = 2276.5 \text{ g} \]
\[ W_3 - \text{dry soil} = 1131.5 \text{ g, for } 24 \text{ hours} \]

\[ W = \frac{W_2 - W_3}{W_3 - W_1} \times 100 \]
\[ = 14.2\% \]

Sample ID: BB364B

\[ W_1 - \text{container} = 276.1 \text{ g} \]
\[ W_2 - \text{wet soil} = 2276.2 \text{ g} \]
\[ W_3 - \text{dry soil} = 3051.2 \text{ g, after 24 hours} \]

\[ W = \frac{W_2 - W_3}{W_3 - W_1} \times 100 \]
\[ = 7.5\% \]

Sample ID: BB365F

\[ W_1 - \text{container} = 276.1 \text{ g} \]
\[ W_2 - \text{wet soil} = 2276.5 \text{ g} \]
\[ W_3 - \text{dry soil} = 2985.9 \text{ g, after 24 hours} \]

\[ W = \frac{W_2 - W_3}{W_3 - W_1} \times 100 \]
\[ = 13.7\% \]
Test Method Used: NBS LLF2.024. TEST 212

Sample No. & Grade: 5

M1: Container 276.1 g
M2: Wet: 2276 g
M3: Dry: 2086 g

\[
\text{Moisture} = \frac{M2 - M1}{M3 - M1} \times 100
\]

= 6.7%

Sample No. & Grade: 6

M1: Container 276.1 g
M2: Wet: 2276.5 g
M3: Dry: 2086.5 g

\[
\text{Moisture} = \frac{M2 - M1}{M3 - M1} \times 100
\]

= 10.5%
Laboratory Reference: 2006/0365/2

DETERMINATION OF THE SOLID DENSITY OF FINE AND COARSE AGGREGATE

ATTENTION:

CLIENT: Tonkin & Taylor Ltd
CLIENT REFERENCE: Ps1-2006
PRODUCT: PHG Fill
CONDITION AS RECEIVED: Natural. Sealed plastic bag
SAMPLED FROM: Gib Air Fill Platform, Unknown
TEST METHODS USED: NZS4402:1986 Test 2.7.2 & NZS4407:1991 Test 3.7.2
SAMPLED BY: 1 Westerson on 14/3/2006
RECEIVED ON: 15/3/2006
TESTED BY: Alan Prescott on 16/3/06

| Fraction passing 19.0 mm Sieve | 74% |
| Solid Density | 2.70 |
| Fraction Retained on 19.0 mm Sieve | 26% |
| Solid Density | 2.58 |

AVERAGE SOLID DENSITY | 2.67 t/m³

Report Issued By: Alan Prescott on 16/3/06

Report Checked By: [Sign]

Approved Signatory: [Sign]

Interim Report
DETERMINATION OF THE SOLID DENSITY OF SOIL

John Westerson
Tonkin & Taylor Ltd
870095-110

PHG
Damp.
Gib Air, Failed pad
Not Specified, Tested as Received
NZS 4402:1986 Test 2.7.2 & NZS 4407:1991 Test 3.7.2
J Westerson on 29/5/2006
29/5/2006
Alan Prescott on 31/5/2006

<table>
<thead>
<tr>
<th>Fraction passing 19.0 mm Sieve</th>
<th>65.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid Density</td>
<td>2.75 t/m³</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fraction Retained on 19.0 mm Sieve</th>
<th>35.0</th>
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</thead>
<tbody>
<tr>
<td>Solid Density</td>
<td>2.60 t/m³</td>
</tr>
</tbody>
</table>

| AVERAGE SOLID DENSITY | 2.70 t/m³ |

Report Issued By: Alan Prescott on 31/5/2006
Report Checked By: Interim Report
Approved Signatory:
BRJV B (2.0 m below original ground surface)

NZS 4402: 1986 Test 4.1.1. Soil portion passing 20 mm standard sieve,

Sampled 18/02/2005
SOLID DENSITY | 2.71

<table>
<thead>
<tr>
<th>WATER</th>
<th>10% Air Voids Line</th>
<th>5% Air Voids Line</th>
<th>0% Air Voids Line</th>
<th>Results Lab</th>
<th>Results Field</th>
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<tbody>
<tr>
<td>5</td>
<td>2.15</td>
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<td>12.4</td>
<td>1.83</td>
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<td>13.9</td>
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<td>1.87</td>
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<td>14.5</td>
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<td>1.95</td>
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<td>1.92</td>
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<td>16.6</td>
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<tr>
<td></td>
<td>2.44</td>
<td>2.57</td>
<td>2.71</td>
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<td></td>
</tr>
</tbody>
</table>

BRJV F

NZS 4402: 1986 Test 4.1.1. Soil passing 19 mm B.S. sieve, Test 2.7.2

Sampled 18/02/2005
GibAir F

NZS 4402: 1986 Test 4.1.1. Soil passing 19 mm British standard sieve, Test 4.7.2

Sampled 29.05.2006
Solid density  2.68

<table>
<thead>
<tr>
<th>Water %</th>
<th>10% Air Voids Line</th>
<th>5% Air Voids Line</th>
<th>0% Air Voids Line</th>
<th>Results Lab</th>
<th>Results Field</th>
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</thead>
<tbody>
<tr>
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<td>2.13</td>
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<td>12.5</td>
<td>1.81</td>
<td>1.91</td>
<td>2.01</td>
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<tr>
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<td>1.77</td>
<td>1.87</td>
<td>1.97</td>
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<tr>
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<td>1.85</td>
<td>1.95</td>
<td>1.73</td>
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<tr>
<td>14.9</td>
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<td>1.82</td>
<td>1.92</td>
<td>1.81</td>
<td>1.81</td>
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<td>15.6</td>
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<td>1.89</td>
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<td>1.77</td>
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<tr>
<td>16.6</td>
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<td>1.76</td>
<td>1.85</td>
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<td>1.44</td>
<td>1.52</td>
<td>1.60</td>
<td>1.60</td>
<td>1.60</td>
</tr>
</tbody>
</table>

GM IV (top of the weathering profile).

NZS 4402: 1986 Test 4.1.1. Soil passing 19 mm British standard seive, Test 4.7.2

Sampled 18.02.2005
Shear Box Test results for sample BRJVF

Tested on 11 November 2006
By: John Westerson

\[ \sigma_n = 144 \text{ kPa} \]
\[ M\% = 13.7\% \]
\[ \rho_{\text{wet}} = 1.75 \text{ t/m}^3 \]
\[ \rho_{\text{dry}} = 1.54 \text{ t/m}^3 \]

Stone corrected
\[ M\% = 10.21\% \]
\[ \rho_{\text{dry}} = 1.71 \text{ t/m}^3 \]

\[ T_u \text{ peak} = 247.6 \]
\[ T_u \text{ residual} = 51 \]
\[ S_r \text{ at failure} = 48.8\% \]
Shear Box Test results for sample BRJVF

Tested on 11 November 2006
By: John Westerson

$\sigma_n = 144$ kPa

$M\% = 10.7\%$
$\rho_{\text{wet}} = 1.51$ t/m$^3$
$\rho_{\text{dry}} = 1.38$ t/m$^3$

Stone corrected
$M\% = 7.41\%$
$\rho_{\text{dry}} = 1.56$ t/m$^3$

$T_u \text{ peak} = 83.4$ kPa

$T_u \text{ residual} = 52$ kPa

$S_r \text{ at failure} = 29.39 \%$
Shear Box Test results for sample BRJVF

Tested on 11 November 2006
By: John Westerson

\[ \sigma_n = 144 \text{ kPa} \]

M\% = 9.8\%

\[ \rho_{\text{wet}} = 1.51 \text{ t/m}^3 \]

\[ \rho_{\text{dry}} = 1.38 \text{ t/m}^3 \]

Stone corrected

M\% = 7.41\%

\[ \rho_{\text{dry}} = 1.56 \text{ t/m}^3 \]

\[ T_{u \text{ peak}} = 107 \text{ kPa} \]

\[ T_{u \text{ residual}} = 58 \text{ kPa} \]

\[ S_r \text{ at failure} = 26.4 \% \]
Test #1. Sample is BRU F. Total on 144 kPa.

\[ F = \frac{374.54}{1500} = 249.6 \text{ kPa} \]

\[ F = \frac{771.7}{1500} = 51 \text{ kPa} \]

\( W = \) Devarda Sample.

\[ V = 10 \text{ cm} \times 10 \text{ cm} \times 6 \text{ cm} = 600 \text{ cm}^3 \]

\[ \frac{W}{V} = \frac{1117}{600} = 1.86 \]

\( \text{Crushed} = 242.2 \)

\[ B_0 = \frac{3.75}{600} = 1.75 \text{ kPa} \]

\[ B_3 = \frac{100 \times 1.75}{100 + 13.7} = 1.54 \text{ kPa/m}^3 \]

Store Corrections:

\( x = \) Retained on 1\( mm \) sieve = 25.4

\[ B_3 \] for retained portion = 2.58 kPa/m^3

\[ \text{corrected } W = \left( \frac{25.4}{100} \right) \times 13.7 = 3.41 \%
\]

\[ \text{corrected } B = \left( \frac{1.54 - 2.58}{2.58} \right) \times 1 - \left( \frac{25.4}{100} \right) = 1.71 \]
Test No 2  

Sample 1 3  4  5  6  7  8  9  10  11  12  13  14  15  16  17  18  19  20  Total 8n = 144 kPa

\[
\text{Tensile} \quad F = \frac{1260.3 \text{ kN}}{15 \text{ cm}^2} = 84 \text{ kPa}
\]

\[
\text{Tensorial} \quad F = \frac{739.32 \text{ kN}}{15 \text{ cm}^2} = 52 \text{ kPa}
\]

\[W = \text{Density} \times \text{Sample Volume}\]

\[N = 10 \text{ cm} \times 10 \text{ cm} \times 5 \text{ cm} = 5000 \text{ cm}^3\]

\[W_0 = 1006 \quad W_1 = 987.2 \quad W_3 = 281 \quad W_4 = 10.7\]

\[W = \frac{755}{500} = 1.51 \text{ kN/m}^3\]

\[p_0 = \frac{100 \times 500}{1000} = 1.5 \text{ kN/m}^3\]

\[p_s = \frac{1000 \times 500}{1000} = 10 \text{ kN/m}^3\]

Shore Conversion

\[K = \text{Shore to Shore} = 75.5\]

\[P_s = \text{Shore to Shore} = 2.85 \text{ kN/m}^3\]

Using Formula from Sheet 2

\[\text{Corrected } M/k = 7.51\]

\[\text{Corrected } p_0 = 1.56 \text{ kN/m}^3\]
Sample: 35.5 kN/m


t = \frac{144\text{ kN}}{150\text{ cm}^3} = 107 \text{ kPa}

t = \frac{35.5\text{ kN}}{150\text{ cm}^3} = 57.8 \text{ kPa}

Machine: Dynamic Sample

V = 10\text{ cm} \times 10\text{ cm} \times 5\text{ cm} = 500\text{ cm}^3

W_0 = 29.9\%

W_1 = 22.4

\text{Soil Type} = 

P_s = \frac{75.6}{500} = 1.51 (\text{ kN/m}^2)

\text{Po} = \frac{105 - 1.51}{105 - 0.4} = 1.3\% (\text{ kN/m}^2)

Stone Connession

v. Retained on Main Scew = 75.6

P_s = \text{in retained position} = 2.0\% (\text{ kN/m}^2)

Using Canadan Form Sheet 2

\text{Connected } = 7.41

\text{Connected } P_s = 1.51 (\text{ kN/m}^2)
Shear Box Test results for sample GIBAIRF (12.2%) 

Tested on 11 November 2006 
By: John Westerson 

$\sigma_n = 144 \text{ kPa}$

$M\% = 12.2\%$
$\rho_{wet} = 1.90 \text{ t/m}^3$
$\rho_{dry} = 1.69 \text{ t/m}^3$

Stone corrected 
$M\% = 10.8\%$
$\rho_{dry} = 1.76 \text{ t/m}^3$

$T_u\text{ peak } = 78.6 \text{ kPa}$

$T_u\text{ residual } = 46 \text{ kPa}$

$S_r\text{ at failure } = 53.77 \%$
Shear Box Test results for sample GIBAIRF (5.4%)

Tested on 11 November 2006
By: John Westerson

\( \sigma_n = 144 \text{ kPa} \)

\( M\% = 5.4\% \)
\( \rho_{\text{wet}} = 1.60 \text{ t/m}^3 \)
\( \rho_{\text{dry}} = 1.52 \text{ t/m}^3 \)

Stone corrected
\( M\% = 4.76 \% \)
\( \rho_{\text{dry}} = 1.60 \text{ t/m}^3 \)

\( T_{u \text{ peak}} = 51 \text{ kPa} \)

\( T_{u \text{ residual}} = 18 \text{ kPa} \)

\( S_r \text{ at failure} = 29.4 \% \)
Shear Box Test results for sample GIBAIRF (5%)

Tested on 11 November 2006
By: John Westerson

$\sigma_n = 144$ kPa

$M\% = 5\%$
$\rho_{\text{wet}} = 1.57$ t/m$^3$
$\rho_{\text{dry}} = 1.49$ t/m$^3$

Stone corrected
$M\% = 4.41\%$
$\rho_{\text{dry}} = 1.57$ t/m$^3$

$T_u \text{ peak} = 51$ kPa

$T_u \text{ residual} = 22$ kPa

$S_r \text{ at failure} = 16.4\%$
Tonkin & Taylor

Office: South Lab
Computed: 14/11
Checked:

Job No: 2006
Project: Undrained Shear Strength - Shear Box vs. Wt. Testing
File: Calc. Sheet

Sample 25 - Group E

Tc = 4

\[ \frac{L}{A} = \frac{261.5 \text{ kN}}{15 \text{ cm}^2} = 17.4 \text{ kPa} \]

\[ \frac{L}{A} = \frac{233.5 \text{ kN}}{15 \text{ cm}^2} = 15.5 \text{ kPa} \]

\( \gamma \), Density Coarse

\[ \gamma = \frac{100 - 150 - 50}{200} = 50 \text{ kN/m}^2 \]

No. = 1030
Vs. = 695.85

\[ \frac{V_s}{\gamma} = \frac{695.85}{1.852} = 373 \frac{\text{m}}{\text{kN}} \]

Wc = 2.42

\[ \gamma_c = \frac{373}{200} = 1.87 \text{ kN/m}^2 \]

\[ \gamma_d = \frac{100 \times 1.52}{100 + 5} = 1.496 \text{ kN/m}^2 \]

Soil saturation

% Retained on 10mm Sieve = 87.8%

No. of retained particles = 2456 kN/m³

Using formula from Sheet 1

Corrected \( \gamma \) = 4.41

Corrected \( \gamma_d \) = 1.52 kN/m³
Test No: 5

Sample ID: 013 A & F

Geotechnical

Tapered

\[
\frac{L}{A} = \frac{113,451}{1500} = 75.6 \text{ kPa}
\]

Tapered

\[
\frac{L}{A} = \frac{630}{1500} = 0.42 \text{ kPa}
\]

M\% = Density Sample

\[
\frac{W_o}{V} = \frac{1202}{10 \times 10 \times 50} = \frac{950}{500} = 1.90 \text{ kN/m}^3
\]

\[
\frac{W_o}{V} = \frac{1244}{10 \times 10 \times 50} = \frac{950}{500} = 1.90 \text{ kN/m}^3
\]

M\% = 12.2

\[
\frac{W_o}{V} = \frac{1252}{10 \times 10 \times 50} = \frac{950}{500} = 1.90 \text{ kN/m}^3
\]

\[
\frac{W_o}{V} = \frac{1252}{10 \times 10 \times 50} = \frac{950}{500} = 1.90 \text{ kN/m}^3
\]

\[
P_a = \frac{950}{500} = 1.90 \text{ kN/m}^3
\]

\[
P_b = \frac{100 \times 1.9}{100 + 12.2} = 1.69 \text{ kN/m}^3
\]

Stone Correction

\[
\gamma_{	ext{retained on Screen 1}} = 28.1
\]

\[
P_a \text{ for retained portion} = 2.52 \text{ kN/m}^3
\]

Using Formulae from Sheet 2

Corrected M\% = 10.75

Corrected \( P_a \) = 1.74 \text{ kN/m}^3
Test No.: 6  
Sample ID.: 0.3 cm.

For peak: \( F = \frac{361.05 \text{ N}}{15 \text{ cm}^2} = 51 \text{ kPa} \)

For residual: \( F = \frac{263.4 \text{ kPa}}{15 \text{ cm}^2} = 18 \text{ kPa} \)

Material: 8 Den-sity Sample

\( V = 10 \text{ cm} \times 10 \text{ cm} - 5 \text{ cm} \)

\( = 500 \text{ cm}^3 \)

\( \\frac{m_n}{m} = 104.5 \)

\( \\frac{m}{m} = 103 \)

\( m = 242 \)

\( m_2 = 80.3 \text{ kN} \)

\( m_3 = 1.00 \text{ kN} \)

\( m_2 = 359.6 \text{ kN} \)

\( m_3 = 43.4 \text{ kN} \)

\( m_2 = 5.4 \% \)

\( b_2 = 2.03 / \text{m}^3 \)

\( b_3 = 1.68 / \text{m}^3 \)

Stone concrete:

\( V \) Retained on 2mm sieve = 2.4 kN

\( V \) for retained portion = 2.54 kN

Using Formula from Sheet 1

Corrected \( m_2 = 5.7 \%

Corrected \( V = 1.60 \text{ cm}^3 \)
Figure.

\[ B = 2^\circ \]
\[ C = 48^\circ \]

\[ \sin \phi = \tan B = 0.035 \]
\[ = 2^\circ \]
\[ C_u = C + 0.5 \tan \phi \]
\[ = 66.5^\circ = 66 \text{ kPa} \]

\[ \sin \phi = 0.194 \]
\[ \phi = 11.2^\circ = 11^\circ \]
\[ C_u = C + 0.5 \tan \phi \]
\[ = 86 \text{ kPa} \]
BRJVF Confining pressure 400 kPa

Strain rate: 0.27 mm/min

Sample soil passing 20mm sieve
Sample compacted by NZ Standard 2.5 kg rammer at 62 blows/layer
M% = 15.5
W0 = 9179.5g
W1= 7923.2 Whole sample oven dried at 110°C
M% = 13.7
P$_{dry}$ = 1.67 Mg/m$^3$
P$_{bulk}$ = 1.90 Mg/m$^3$

Lo= 273 mm
L$_1$ = 230 mm
Strain = 43mm after 132 min's

LL = 35 - 39
Pl =13 - 17Low plasticity clay (31.7% fines)

USCS classification: CL-GC
Sample ID: 8K3N

Sample Details: Original bulk sample sealed @ 19mm standard tense.
- Field moisture content = 13.3%
- Air dried following sieving to 1.675mm
- Oven 16.6%
- P8 of < 2Mm Fraction = 2.31k/m³
- % of whole soil = 82.2%
- Sample compacted @ OMC in a 100mm x 300mm split mould using a standard 2.5 kg rammer @ 62 blow/layers

Tunnel Test: Multistage, 50, 100, 200, 400 kPa containing pressure
- No pore pressure measurement
- Red line = load 0 - 25 kPa/page width
- Small speed = 800 mm/hr

Vertical Strain Measurements:

<table>
<thead>
<tr>
<th>Sample</th>
<th>L = 295mm</th>
<th>D = 150mm</th>
<th>d = 15mm</th>
</tr>
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<tbody>
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<td>Standard</td>
<td>Trials</td>
<td>No.</td>
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<tr>
<td>L₀</td>
<td>28.9</td>
<td>0.0</td>
<td>-</td>
</tr>
<tr>
<td>L₁</td>
<td>24.0</td>
<td>1.0</td>
<td>50</td>
</tr>
<tr>
<td>L₂</td>
<td>28.6</td>
<td>1.0</td>
<td>100</td>
</tr>
<tr>
<td>L₃</td>
<td>28.5</td>
<td>1.0</td>
<td>200</td>
</tr>
<tr>
<td>L₄</td>
<td>28.3</td>
<td>1.0</td>
<td>362</td>
</tr>
</tbody>
</table>

\( W₀ \) contained = 3162.40 kg
\( W₀ \) total = 2611.30 kg
\( W₀ \) used = 2227.50 kg
\( M% \) = 11.7%

\( V = \frac{T²}{A} = 1.392 \)

\( \sigma = \frac{(S₀)}{P₀} (1 + M%)^{-1} = 0.65 \)

\( \sigma = \frac{(M% \times S₀) \times 100}{V} = 60.97 \)

\( 0 = 0.0176 m² \)

\( 0 = 0.0782 m \)
SAMPLE ID: BRU15

Sample History:
- Original bulk sample scooped at 10 min mean standard sieve
- Field moisture content: 13.1%
- Air dried following scooping to 10.6%
- Oven 16.6%
- P<sub>0</sub> of < 20mm fraction: 7.2t/\text{m}^3
- % of whole soil: 52.2%
- Sample compacted @ 2kPa in a 10cmm x 300mm split mould using a standard 2.5kg rammer @ 62 blows/layer (6 layers).

Triaxial Test:
- Single stage monotonic load @ Pa conveying pressure
- No pore water measurement
- Red line = load 0.25 KN/page width
- Overall speed = 300 mm/hr
- Start time: 15:100 hrs
- Finish time: 15:40 hrs.
- Total 130 mm...

\[
\begin{align*}
W_0 & = 9514 \text{g} \\
W_d & = 336.4 \text{g} \\
W_i & = 9139.5 \text{g} \\
\text{WM} & = 7923 \text{g} \\
\text{Mf} & = 13.7
\end{align*}
\]

\[
\begin{align*}
% & = \frac{W_i - W_d}{W_d} \\
V & = \pi \frac{d^2}{4} \times l = 4.82 \text{ m} \\
\varepsilon & = \frac{\left(\frac{W_i}{W_d}\right) - 1}{\frac{W_i}{W_d}} = 0.62 \\
\pi \frac{d^2}{4} & = 0.0126 \text{ m}^2 \\
\frac{d}{l} & = 2.33 \\
\ell & = 2.30 \text{ m} \\
\text{Stress rate} & = 0.31 \text{ mm/min average}
\end{align*}
\]

Failure Mode:
GibAir F  Confining pressure 200 kPa

Strain rate: 0.27 mm/min

Sample soil passing 20mm sieve
Sample compacted by NZ Standard 2.5 kg rammer at 62 blows/layer

\[ W_0 = 9636.5 \text{g} \]
\[ W_{1x} = 8520 \quad \text{Whole sample oven dried at 110^\circ C} \]
\[ M\% = 11.6 \]
\[ P_{\text{dry}} = 1.92 \text{ Mg/m}^3 \]
\[ P_{\text{bulk}} = 2.14 \text{ Mg/m}^3 \]

\[ L_0 = 255 \text{ mm} \]
\[ L_1 = 250 \text{ mm} \]
\[ \text{Strain} = 5\text{mm after 20 min's} \]

\[ L_L = 29-33 \]
\[ P_L = 16 - 20 \quad \text{Low plasticity silt} \quad (14.4\% \text{ fines}) \]

USCS classification: GM
Sample ID: 15

Sample Details:
- Original Bulk sample sealed at 10 atm standard room
- Field moisture content: 10.5%
- Field content following compaction: 14.1%
- OM: 10.4%
- L % 25 ≤ 20 mm fraction: 1.75 kN/m³
- % of whole soil: 34.7%
- Sample compacted at OMC in a 150 mm x 500 mm split mould using a standard 5.5 kg rammer at 62 blows/layers.

Testing:
- Minor test at 50, 100, 200, 400 kPa continuing pressure
- No live pressure measurement
- Creep test: load = 25 kN/m², length
- Speed = 300 mm/h
- Undrained shown measurements.

<table>
<thead>
<tr>
<th>Sample</th>
<th>L</th>
<th>200%</th>
<th>d</th>
<th>150%</th>
<th>1</th>
<th>100%</th>
<th>90%</th>
<th>80%</th>
<th>70%</th>
<th>60%</th>
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<td>L0</td>
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<td>113.2</td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L3</td>
<td>28.8</td>
<td>0.0</td>
<td>65.1</td>
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<tr>
<td>L4</td>
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<td>65.1</td>
<td>90.5</td>
<td>113.2</td>
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<td></td>
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</tbody>
</table>

\[ R = \frac{2262.4}{124.0} = 1.83 \text{ kN/m}^3 \]
\[ B = \frac{100 \times 2.3}{100 \times 2.3} = 1.68 \text{ kN/m}^3 \]

\[ e = \left( \frac{S_g}{S_o} \right) \left( 1 + M \right)^{1/2} = 1.84 \]

\[ V = \pi r^2 h = 124.2 \]

\[ S_r = \left( \frac{M \times S_o}{v} \right) \times 100 = 113.3 \]
## Degree of saturation for unconsolidated undrained shear box and triaxial testing

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<thead>
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<th>Sample Shear box</th>
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<th>Bulk density</th>
<th>w</th>
<th>e</th>
<th>Sr</th>
<th>Comments</th>
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</thead>
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<td>GibAir F</td>
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<td>1.9</td>
<td>0.122</td>
<td>0.62</td>
<td>53.77</td>
<td>At failure</td>
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<tr>
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<td>1.75</td>
<td>0.137</td>
<td>0.76</td>
<td>48.80</td>
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<tr>
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<td>At failure</td>
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<td>BRJV F</td>
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<td>At Failure multi stage</td>
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<td>0.29</td>
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</table>
STAGE 3 GRAPHS

CONSOLIDATION

VOLUME CHANGE (mL)

SQUARE-ROOT TIME

STRESS-STRAIN CURVE

Uncorrected Deviator Stress (kPa)

Vertical Strain (%)
STAGE 1 GRAPHS

CONSOLIDATION

VOLUME CHANGE (mL)

SQUARE-ROOT TIME

STRESS-STRAIN CURVE

Unconfined Deviator Stress (kPa)

Stress Ratio

Vertical Strain (%)
STAGE 2 GRAPHS

CONSOLIDATION

VOLUME CHANGE (mL)

SQUARE-ROOT TIME

STRESS-STRAIN CURVE

Uncorrected Deviator Stress (kPa)

Vertical Strain (%)

Deviator Stress --- Stress Ratio

Entered by: [Signature]
Date: 9/11/07
Checked by: [Signature]
Date: 9/11/07
APPENDIX C

Fill Testing Data

- Fill test spread sheets – BRJV, GibAir & Green Meadows;

- Air void/Moisture charts;

- Barrel permeability test data and calculation sheets – (PT1 – PT3).
<table>
<thead>
<tr>
<th>Column 1</th>
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<td>Data 33</td>
<td>Data 34</td>
<td>Data 35</td>
<td>Data 36</td>
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**Notes:**
- Column headers are placeholders.
- Data entries are also placeholders for actual values.
- This table is part of a larger document, possibly a technical report or a research paper.
Model = Calculation Ref.

\[ K = \frac{H_1 - H_2}{t_1 - t_2} \times \frac{1}{C_{mf}} \]

\[ \frac{H}{R} = \frac{H_1 + H_2}{2R} \]

\[ \frac{C}{H} = \text{From} \]

Lake First Construction Contract;

Appendix A:

- In-situ Drum Method
  of Permeability Testing

Reference for Calculation

- Section D.2.4
  - Equation 4

- Appendix A
  - Figure 2
\[ k = \frac{h_1 - h_2}{t_1 - t_2} \times \frac{1}{c, t} \]

In sleds with sleds:

\[ k_1 = 282.0 \text{ mm} \quad t_1 = 111 \text{ mm} \]
\[ k_2 = 282.0 \text{ mm} \quad t_2 = 282.0 \text{ mm} \]

\[ R = \frac{880}{2} = 290.0 \text{ mm} \]

\[ \frac{h_1 + h_2}{2} = 700 \text{ mm} \]

\[ \frac{h}{R} = \frac{300}{200} = 1.5 \]

\[ c_t = 5.5 \]

\[ k = \frac{311 - 6560}{4820 - 2820} \times \frac{1}{0.5} \]

\[ = \frac{22/1000}{1950 \times 60} \times \frac{1}{5.5} \]

\[ = 2.245 \times 10^{-3} \text{ mm/mm} \]

\[ = 2.245 \times 10^{-6} \text{ m/m} \]

\[ = 3.409 \times 10^{-8} \text{ m/sec} \]

Acceptable < 1.0 x 10^{-7} m/s
\[ k = \frac{H_1 - H_2}{k_1 - k_2} \times \frac{1}{L_{1/2}} \]

In steady state section

\[ L_{1/2} = \frac{580}{2} = 290 \text{ mm} \]

\[ H_1 + H_2 = 490 \text{ mm} \]

\[ \frac{H_1 + H_2}{2} = \frac{490}{2} = 245 \text{ mm} \]

\[ H = \frac{480}{240} = 1.6 \]

\[ G = 4.1 \]

\[ k = \frac{222}{272.1 \times 60} \times \frac{1}{4.1} \]

\[ = -0.00123 \times 0.2434 \]

\[ = 3 \times 10^{-4} \text{ mm/sec} \]

\[ = 3 \times 10^{-7} \text{ m/sec} \]

\[ \theta < 1 \times 10^{-7} \text{ m/sec} \]
In stead. Val. Section

\[ K = \frac{H_1 - H_2}{L_1 - L_2} = \frac{1}{C_{311}} \]

\[ K = \frac{H_1 - H_2}{L_1 - L_2} = \frac{1}{C_{311}} \]

\[ R = \frac{500}{2} = 250 \text{ mm} \]

\[ Q = \frac{H_1 + H_2}{2} = 629 \text{ mm} \]

\[ \frac{Q}{R} = \frac{629}{250} = 2.519 \]

\[ \frac{C_1}{C} = 5.03 \]

\[ K = \frac{220}{450 \times 60} \times \frac{1}{5.03} \]

\[ = 1.4 \times 10^{-4} \times 0.192 \]

\[ = 4.7 \times 10^{-9} \text{ mm} \text{ / sec} \]

\[ = 7.0 \times 10^{-8} \text{ m} \text{ / sec} \]
### Falling Head measurements for Gib Air Dam Barrel Permeability Tests

#### Gibair Dam Barrel Perm Test 1  
**Clayey Gravel**

<table>
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<th>Log time</th>
<th>Head</th>
<th>Depth to water</th>
<th>Head loss</th>
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</thead>
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<tr>
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#### Gibair Dam Barrel Perm Test 2  
**Silty Gravel**

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**Silty clayey Gravel**

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APPENDIX D

- Soil Mechanics formulae table;
- Scala & Stockwell bearing capacity charts;
- Moore's Field soil strength tables for cohesive & non-cohesive soils
References


NZS 4402, 1986. Methods of testing soils for civil engineering purposes.

