Filtration of Port Hills Loess for Retaining Wall Situations

A thesis submitted in partial fulfilment of the requirements for the degree of Master of Science in Engineering Geology at the University of Canterbury by JUSTIN ANDREW LAVELLE HARRISON

I'm sorry buddy, but I think you have quite enough dirt!
Abstract

A series of laboratory tests have been conducted with Port Hills loess from the Ahuriri quarry. Tests have been conducted to obtain a theoretical basis for suggestions of good design practice for retaining wall filtration/drainage systems. The study has concentrated on investigation of geotextiles as filtration options, with granular filters being tested for comparison. Two of the tests (stage 1 test and gradient ratio test) are designed to provide comparative information on filter performance with specific soil types. The third test is a series of three laboratory scale retaining wall simulations. Information from these tests provides evidence of potential problems with commonly used retaining wall filtration/drainage systems. In conjunction with laboratory testing, field observations have been carried out to assess the current state of the practice, and identify additional areas of concern.

Results from both the gradient ratio and stage 1 tests indicate good performance of a range of needle punched nonwoven geotextiles for filtering the specific soil tested (Ahuriri quarry loess). Scanning Electron Microscope investigation of the soil and geotextile filter structures formed during testing indicate bridging to be the dominant filter network, with one example of a vault network for the needle punched range. Small amounts of clogging observed within the geotextile structure have not resulted in significant reductions in permeability. Gradient ratio values for a selection of the needle punched range support evidence from the stage 1 test that satisfactory filter performance is provided.

Gradient ratio values of greater than 3.0 are recorded for two heat bonded nonwoven geotextiles, indicating a concerning level of clogging. Observations of tested heat bonded samples under the SEM show a degree of “blocking” (a specific form of clogging). These observations are supported by lower permeability values in both tests, comparative to the needle punched range, suggesting the needle punched range is better suited to filtering this specific soil type.

Indications from the retaining wall simulations suggest a need for impermeable drain channels under drainage pipes to prevent erosion of loess resulting from water flow under the drain pipes. Also indicated by these tests is the presence of salts on the backfill material obtained from a local quarry. Although further investigation is recommended, the potential exists for corrosion of steel reinforcing where adequate water proofing is not placed. The need for adequate surface drainage (particularly during construction) in combination with good retaining wall design is emphasised.

As a culmination of the testing and field work, a number of suggestions are put forward as aspects of good design practice for filtration/drainage systems for use in retaining walls on the Port Hills. Selection of an appropriate geotextile for a specific filtration project should ideally follow an in-depth design process, and should not be controlled by budget constraints, as is sometimes the case. Numerical design criteria, based on site specific soil properties and geotextile properties, as well as compatibility tests (such as the gradient ratio test, or the stage 1 test) are available to aid in the design process.
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Chapter 1
Introduction
Chapter 1  INTRODUCTION

1.1. Background Statement

The rapid development of the Christchurch metropolitan area is placing an ever increasing demand on the Port Hills for residential purposes. The Port Hills provide prime real estate, with good views and clear air. As a result, development on the hills is primarily up-market residential. With the availability of money being less of a problem to this sector of the public, the development of more and more “marginal land” (as defined by Bell, 1996) is continuing.

Development projects on the Port Hills regularly require retaining walls to be placed into soil slopes. Problems associated with the loessial soils found on the Port Hills are well documented (e.g. Hosking 1962, Bell and Trangmar 1987, Goldwater, 1990, Jowett, 1995 and many others), and will also be outlined within this thesis. The Resource Management Act of 1991 requires the potential for several natural hazards to be investigated, and mitigated against if present at the subdivision level, with the Building Act 1991 focusing on potential hazards of individual sites (see appendix 1 for more detail). Some of these hazards (for example erosion in its various forms) are common problems in loess (see appendix 1 for more detail). Retaining walls are commonly used on the Port Hills in loessial soils, and may be part of an erosion mitigation plan for a particular site. Therefore, there needs to be well set out guidelines for the construction of retaining walls with this particular soil type to mitigate against these hazards.

Adequate filtration and drainage are necessary features of any retaining wall, (along with the usual structural considerations) to ensure long term stability and performance. A number of filtration media are readily available for use on the Port Hills, for example: Canterbury Plains river gravel, premix, and a variety of geotextiles (these terms will be defined over the course of this thesis). However, no one product will suit every situation, and therefore, investigations must be carried out to provide much needed guidelines. Since geotextiles are becoming more commonly used on the Port Hills, they have formed a focus of this project.

1.2. Thesis Structure and Objectives

The objective of this thesis is to provide suggestions for good design of retaining wall filtration/drainage systems for Port Hills situations, in an attempt to reduce erosion problems
and prolong functional performance times. This is approached by way of obtaining both theoretical bases, through stage 1 and standard tests (Chapters 4 and 6 respectively), and practical bases through field observations and the stage 2 test (Ch.5), for design practise. As it is hoped a number of the suggestions made will be adopted by practising professionals, emphasis is placed on finding readily applicable solutions. Testing has also attempted to focus on real-life situations, hence soil sample selection has been purely random.

As the majority of filters tested are geotextiles, Chapter 2 outlines the terminology of the broad range of products known as geosynthetics, of which geotextiles are a subset. Chapter 2 also looks at theoretical aspects of design philosophy and methodology. In conjunction with laboratory testing, field observations have been carried out to ascertain the current state of field practise. A number of case studies of field practise are presented in Chapter 3.

1.2.1. Stage 1 test
This is a simple laboratory test used to compare the performance of various filters for retaining loess. Water that passes through loess and then a filter (all contained in a pipe) will be collected to assess the amount of fines passing through the system, and for measurement of system permeability. Experiments are continued until equilibrium has been attained. On completion, the experiment is carefully dismantled, so the soil/geotextile interface may be inspected. Chapter 4 focuses on this experiment.

Objectives of the stage 1 test:
➢ To obtain a quantitative comparison between various filters by way of measuring permeability versus time, and fines passed over time.
➢ To gain some information regarding the structure of the soil/geotextile interface.

1.2.2. Stage 2 test
This is a more complicated laboratory test that simulates retaining wall designs, looking specifically at the filtration/drainage systems. Chapter 5 discusses this experiment in detail.

Objectives of the stage 2 test:
➢ Compare the effectiveness of three retaining wall filtration/drainage systems
Establishment of good practice guidelines for retaining wall construction.

Obtain some information regarding water and sediment paths through the system.

Obtain information regarding the influence of tunnel gullies on retaining wall filter/drainage systems.

1.2.3. Standard test

The standard ASTM D5101-90 soil/geotextile permeameter provides permeability and gradient ratio data. Both of these parameters are widely recognised as important aspects of theoretical design approaches. Information provided is also useful for comparison to the stage 1 tests. Chapter 6 focuses on this experiment.

Objectives of the standard test:

➢ To provide a quantitative, repeatable and widely recognised comparison between various filters.
➢ Provide a basis with which to compare the results of the stage 1 test.

1.2.4. General

As well as the specific objectives for each test, the project as a whole has additional objectives:

➢ Comparison of geotextile and granular filtration media performance.
➢ Recommendations for use of filters in Port Hills loess retaining wall situations.
➢ Comparison of standard test and stage 1 test results.
➢ Development of new testing procedures (stage 1, stage 2).
➢ Provide suggestions for good design practice of retaining wall filtration/drainage systems on the Port Hills.

1.3. Field Area

The Port Hills form the north western margin of the Lyttleton harbour, which is in turn a part of the area known as Banks Peninsula. Banks Peninsula is located on the eastern coast of the South Island of New Zealand, as shown in figure 1.1. The city of Christchurch is predominantly spread over the flat lying areas to the northwest of the Port Hills, although residential development of Christchurch on the Port Hills is ongoing.
Figure 1.1. Location map of the Banks Peninsula area.
1.3.1. Urban development
The population of Christchurch city increased by approximately 21,000 to 313,969 people between 1991 and 1996, a 7.2% increase (Christchurch City Council, 1996). Growth is expected to continue in the city, with a projected population of 354,700 by the year 2016 (C.C.C. 1996). Some of that expected growth will be accommodated by development on the Port Hills (C.C.C. 1996). Figure 1.2 shows the extent of a proposed new subdivision on the Port Hills that would see the construction of many retaining walls in loess.

![Figure 1.2 Proposed subdivision of Montgomery Spur](from “The Press” 1998)

1.3.2. Climate
A summary of Christchurch city’s climate (given in table 1.1) shows that the city has a relatively mild climate. It is recognised that the Port Hills is a separate micro-climate from the flat lying areas of the city directly north (CCC, 1996). Humidity is typically higher on the Port Hills and a greater seasonal variation in rainfall is also observed on the hills, with rainfall averages remaining similar between the hills and flat areas of the city (CCC, 1996).
Table 1.1 Climate data for Christchurch area (data from CCC, 1996).

<table>
<thead>
<tr>
<th>Climatic Feature</th>
<th>January</th>
<th>July</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature (mean daily)</td>
<td>Max.= 21°C Min.=11.6°C</td>
<td>Max.=10.3°C Min.=1.4°C</td>
</tr>
<tr>
<td>Relative Humidity (average)</td>
<td>3am=83% 3pm=57%</td>
<td>3am=88% 3pm=70%</td>
</tr>
<tr>
<td>Sunshine</td>
<td>2,040 hours</td>
<td></td>
</tr>
<tr>
<td>Rainfall</td>
<td>(≥1mm) 87days &amp; 655mm total</td>
<td></td>
</tr>
<tr>
<td>Frost</td>
<td>(min air temperature &lt; 0°C) 36 days</td>
<td></td>
</tr>
<tr>
<td>Wind</td>
<td>(≥63km/h) 56 days, (≥96km/h) 2.8 days</td>
<td></td>
</tr>
</tbody>
</table>

The dominating weather pattern (particularly in summer months) of warm, dry north westerly winds is attributed to Christchurch being on the lee side of an orographic rainfall system. Winds blowing over the Tasman sea to the west of New Zealand accumulate moisture, and on contact with the Southern Alps moisture condenses producing the high rainfall that is seen on the west coast of New Zealand. The resulting dry, warm air then passes over the Canterbury region. Figure 1.3 shows rainfall distribution over the Port Hills and surrounding regions.

Figure 1.3 Rainfall isohyets for the Port Hills and surrounding regions (From McGann, 1983)
1.4. Geology of the Port Hills

1.4.1. Regional Setting

Banks Peninsula is composed primarily of volcanic rocks, unconformably overlying a basement of Triassic sediments (Torlesse Terrane), with some Cretaceous – Tertiary rocks, and a covering of several soil types. During the deposition of the volcanics (12-6Ma), the peninsula was then an island lying offshore from the mainland (Weaver et al, 1985). Following volcanism, a combination of changing sea level and the progradation of gravel deposits eastward have caused ‘Banks Island’ to join with the mainland and become a peninsula. The rapid progradation of the gravels, that now form the Canterbury plains, can be greatly attributed to the high rate of uplift that has been continuing in the Southern Alps since the initiation of the Kaikoura Orogeny approximately 15Ma (Weaver et al, 1985). This is, in turn, widely accepted as being due to the formation of, and movement along, the Alpine Fault which constitutes the tectonic plate boundary between the Pacific and Indo-Australian plates.

1.4.2. Geological History of Banks Peninsula

The following should be considered a brief and general overview of the history of the Banks Peninsula area.

I) Torlesse

Rocks of the Torlesse Terrane (which range in age from Permian to Mid Cretaceous (Bradshaw et al, 1993)) are found in the Gebbies Pass area of Banks Peninsula, and are thought to be approximately 240Ma at this locality (Weaver et al, 1985). Primarily argillites and arenites can be seen at this locality. However, this Terrane does contain conglomerates, red cherts, basic volcanics (including pillow basalts), and some limestones (Bradshaw, August 1989) at locations other than Banks Peninsula. The Torlesse Terrane is thought to have formed as an accretionary complex in a convergent margin setting (Bradshaw et al, 1993).

II) Mount Somers Volcanics

Also to be found in the Gebbies Pass area, and in McQueens Valley, are a series of andesite flows and rhyolite domes. At approximately 80Ma old, these are the Mount Somers Volcanics, and are thought to connect under the Canterbury Plains gravels to the similar
volcanics seen in the Mount Somers area approximately 100km to the west (Sewell et al, 1992). The majority of this sequence of explosive eruptions has been eroded from the Banks Peninsula area (Weaver et al, 1985).

III) Eyre Group and Burnt Hill Group
The deposition of marine sandstone between 65Ma and 15Ma, indicates that the Banks Peninsula area was submerged during this time. This period of deposition is seen today as the Eyre Group and Burnt Hill Group. These Groups are typically siliceous and volcanically derived marine sandstones. A number of gaps in the geologic record during this time indicate periodic variations in sea level relative to this area (Sewell et al, 1992). Initial movements of the Kaikoura Orogeny raised the area above sea level permanently by 15Ma (Weaver et al, 1985).

IV) Governors Bay Volcanics
The Governors Bay volcanics referred to by Weaver et al (1985) have since been separated into the Allandale rhyolite, and the Governors Bay Andesite (Sewell et al, 1992). Originally assumed to be approximately 15Ma (e.g. Weaver et al, 1985), more recent Rb/Sr dating has set an age for these deposits at 10.8±0.1Ma (Barley et al, 1988). These eruptions represent a relatively small portion of the total volume of Banks Peninsula. Deposits from this time are still visible in Gebbies Pass, Governors Bay, and on Quail Island.

V) Lyttleton Volcano
About 11Ma, a new phase of volcanism commenced. This activity was hawaiian in nature with occasional more violent strombolian eruptions. Deposits produced include basaltic ‘aa’ type lava flows interbedded with some ash and laharc deposits (Brown and Weeber, 1992). According to Weaver et al (1985) the dominant lava type produced was hawaiite. However, a recent study by Neumayr (1998) indicates a greater abundance of basaltic lavas for the initial, and largest phase of Lyttleton volcanism. There were also some flows of mugearite and trachyte, (Sewell et al, 1992). Over a period of approximately 2Ma this series of volcanic episodes had built the Lyttleton volcano to a height of around 1500m (Weaver et al, 1985). An estimated minimum volume for the Lyttleton Volcanic group (as defined by Sewell, 1988), is approximately 350km³ (Sewell, 1988).
Another feature of the Lyttleton volcano, is the presence of radial dikes. The dikes are mainly hawaiite, basalt, mugearite and trachyte in order of abundance, and are thought to have been formed continuously throughout the formation of the volcano (Shelley, 1988). This view is in contrast to earlier opinions that suggested dike formation occurred late in the volcano's formation due to a doming effect caused by an upwelling of magma deep within the volcano (Weaver et al, 1985). Orientation data collected from these dikes suggests that the Lyttleton volcanic group is derived from two distinct centres (Shelley, 1987).

VI) Akaroa Volcano
To the southeast of the Lyttleton volcano, volcanic activity began at approximately 9.3Ma with the initial eruptions (called the Tikao trachytes) of what was to become the Akaroa volcano. This volcano is composed of alkaline basalt to trachyte lava flows, some tuff, agglomerate and parasitic cinder cones, and of two intrusives (Duvauchelle gabbro and Onawe syenite). The estimated total volume for the Akaroa volcano is 1200km$^3$, and with a projected height of 1800m above present sea level (Sewell, et al, 1992). Activity is thought to have ceased in this area at about 8.0 Ma.

VII) Mount Herbert Volcanics
The Mount Herbert volcanics are a series of hawaiite, mugearite and basalt intrusives and lava flows, contemporaneous with the onset of early Akaroa volcanism. They are located between the centres of the Lyttleton and Akaroa volcanoes, in the central area of Banks Peninsula. Activity is also thought to have ended around 8.0Ma.

VIII) Church Volcanics
The Church volcanics represent a very small portion of the volume of Banks Peninsula. They were deposited between 8.1-7.3 Ma and represent a transition between the Akaroa Volcanic Group and the later Diamond Harbour Volcanic Group (Sewell, et al, 1992).

IX) Diamond Harbour Volcanic Group
From around 7.0-5.8Ma, the deposition of the Diamond Harbour volcanics was occurring, and represents the final phase of volcanism seen on Banks Peninsula. The most obvious feature seen today of this group, is the 5km long northward dipping lava flows that stretch
from Mount Herbert to Diamond Harbour. Composition of this phase of volcanism is typically basinite, olivine basalt and olivine hawaiite (Sewell, et al, 1992).

X) Post Volcanic
As mentioned in the regional setting, aggradation of fluvial gravels was responsible for the Lyttleton and Akaroa volcanic centres being joined with the mainland. Deposition of gravel was accelerated by a combination of the rapid rise of the Southern Alps, and a series of glacial advances, each providing abundant sediment. During glacial periods, sea level was thought to have been approximately 150m lower than present (Sewell, et al, 1992), providing a lower base level and therefore increased energy for river systems to transport gravel. Grinding action of glacial ice produced abundant silt which was subsequently transported to the Banks Peninsula area by north westerly winds, and deposited as loess (see 1.5 for more detail). Interglacial periods were more typically characterised by reworking of existing fluvial deposits, and the deposition of marine sand, silt, clay and peat (Sewell, et al, 1992). The Otira Glaciation ceased about 14,000 years ago, and is thought to have been the last major period of glacial climate (Bell, pers. comm. 1998). Post-glacial deposition has been a combination of fluvial and marine processes.

1.5. Loess
1.5.1. General introduction to loess
Loess is described by a number of authors (e.g. Jowett, 1995; Higgins and Modeer, 1996) simply as an aeolian deposit composed primarily (60-85%) of silt-sized particles. Loess deposits can be found in many areas around the world including Europe, Asia, North America, Antarctica, and of course, New Zealand. The area of land that is covered by loess has been estimated at 11% of the earth’s total land area (Turnbull, 1965) and at 10% (deposits over 1m thick) over the South Island of New Zealand (Bruce, 1972).

The typical mode of formation of loess (especially for North American and New Zealand) is deposition of wind borne silt, fine sand and clay size particles, derived from glacial grinding. This theory has support in the USA with a tendency for loess to be more fine grained with greater distance from the inferred source (Schultz and Frye, 1965). As a result of the aeolian deposition, loess is characterized by a loose structure of silt and fine sand with a variable amount of clay binder (Higgins and Modeer, 1996). This structure
results in dramatic losses in shear strength close to saturation. This behaviour is a major contributor in a number of recorded slope failures in loess. Perhaps the most dramatic slope failure in loess was in China in 1920, when around 100,000 people lost their lives in a series of landslides in loess finally triggered by a M8.5 (Richter magnitude) earthquake (Higgins and Modeer, 1996). Fortunately, failures in loess in New Zealand have not yet been so devastating as a result of a slightly less collapsible structure. However, New Zealand loess does cause a number of problems (as will be discussed in section 1.6).

1.5.2. Origin and Distribution
The loess that is found on Banks Peninsula originated primarily from the grinding of rocks in the Southern Alps (to the west of Banks Peninsula) during the last glaciation approximately 2Ma-14,000 years ago (Sewell, et al, 1992). Occurrence of sponge spicules in loess deposits up to 50km inland suggests some loess may have been derived from exposed continental shelf during the Pleistocene glaciations (Raeside, 1964), with the less frequent easterly and southerly winds providing the necessary transport for this material. Figure 1.3 shows the suggested origin of Banks Peninsula loess.

Figure 1.4 Suggested origin of Banks Peninsula loess (From Bell and Trangmar, 1987)
On Banks Peninsula, two facies of loess have been described by Griffiths (1973) and are named after their type localities:

1) Barrys Bay loess, and
2) Birdlings Flat loess

The main differences between these two are: the presence of calcareous material (including veins, concretions and minor cement) in the Birdlings Flat loess, and absence in Barrys Bay loess; the typically finer grain size of Barrys Bay loess; and the tendency for Barrys Bay loess to occur on upper slopes and summit areas only. The distribution of these two facies is shown in figure 1.5. This distribution shows that Birdlings Flat loess is predominant over the Port Hills area of Banks Peninsula.

Figure 1.5. Distribution of Barrys Bay and Birdlings Flat loess on Banks Peninsula (From Griffiths, 1973)
When comparing the observed distribution of loess on Banks Peninsula with the rainfall isohyets seen in figure 1.3 there appears to be support for the theory that occurrence of calcite may be a function of rainfall or evaporation, as has been observed in eastern Washington, USA (Higgins and Modeer, 1996). Areas of greater than 51 cm of precipitation per year in eastern Washington contain loess with no calcite (Barrys Bay equivalent), and areas of less than 38 cm annual precipitation contain loess with significant calcite (Birdlings Flat equivalent). Although a similar pattern can be roughly seen when comparing the distribution of loess with rainfall on Banks Peninsula, it appears that 80 cm annual precipitation may be a more appropriate boundary in this setting. It should be noted that this observed trend has not been investigated further than comparing figures 1.3 and 1.5 and thus is not firmly established, but may perhaps warrant further investigation.

1.5.3. Composition
The composition of loess is typically 50-60% quartz and 20-30% feldspar, which makes up the bulk of the silt and sand fractions (Jowett, 1995). The clay size fraction is made up of quartz and clay minerals including illite, interstratified illite/vermiculite, and minor vermiculite (Mackwell, 1986). A range of accessory minerals including muscovite, epidote, zircon, tourmaline, chlorite and hornblende have been observed, and their presence is dependent on the source area. Figure 1.6 shows the range of grain size distributions that were tested by Jowett (1995) at the Ahuriri quarry in Birdlings Flat loess. This is the site from which test samples have been collected for this study.

Figure 1.6 Grain size distribution envelope for Birdlings Flat loess at Ahuriri quarry (from Jowett, 1995)
1.5.4. Structure

Most exposures on the Port Hills show some degree of layering in loess. Typical structure of loess as observed on the Port Hills, has been described as having three main layers (Bell et al, 1986):

1) “S” = surface layers (0.5-1.5m deep, typical dry density = 1.54x10^3 kg/m^3)
2) “C” = compact layer, also referred to as fragipan (directly below S, typically 0.5m thick with dry densities as high as 1.88x10^3 kg/m^3, thought to form as a result of repeated expansion of weakly weathered illitic clay minerals under seasonally wet and dry conditions).
3) “P” = parent layer (directly below C, up to 10m thick, dry densities as low as 1.32x10^3 kg/m^3)

Figure 1.7 shows a generalised cross section of loess structure on the Port Hills.

Figure 1.7 Generalised cross section of Port Hills loess (adapted from Bell et al, 1986)

1.5.5. Typical Geotechnical Properties

Significant testing has been carried out on loess from Banks Peninsula, and is summarised in table 1.2. Recent work by Jowett (1995) on loess from the Ahuriri quarry (which is used throughout the testing program of this study) shows that this material is both erodible, and moderately dispersive. Both of these properties are problematic for filtration design, as they encourage mobilisation of soil toward the filter with water flow. In addition, the silt rich grain size distribution makes satisfying typical filtration design criteria difficult with commonly available geotextiles (see chapter 2 for more detail on design criteria).
Table 1.2 Summary of Geotechnical properties of Banks Peninsula loess.
(Modified from Jowett, 1995).

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porosity</td>
<td>30 - 40%</td>
<td>Birrel and Packard (1953)</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>0.4 - 0.7</td>
<td>Birrel and Packard (1953)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Miller (1971)</td>
</tr>
<tr>
<td>Atterberg Limits</td>
<td>LL: 18 - 33</td>
<td>Crampton (1985)</td>
</tr>
<tr>
<td></td>
<td>PI &lt; 12</td>
<td>Alley (1966)</td>
</tr>
<tr>
<td></td>
<td>Activity: 0.1 (C horizon)</td>
<td></td>
</tr>
<tr>
<td>Grain Size</td>
<td>Sand: 10%</td>
<td>Alley (1966)</td>
</tr>
<tr>
<td></td>
<td>Silt: 65 - 80%, Clay 11 - 25%</td>
<td>Crampton (1985)</td>
</tr>
<tr>
<td>Dry Density</td>
<td>B horizon average = 1.54 t/m$^3$</td>
<td>Evans (1977)</td>
</tr>
<tr>
<td></td>
<td>Cx'horizon average = 1.64 t/m$^3$</td>
<td>Crampton (1985)</td>
</tr>
<tr>
<td></td>
<td>(1.51 - 1.88 t/m$^3$ range)</td>
<td>Yetton (1986)</td>
</tr>
<tr>
<td></td>
<td>(1.32 - 1.7 t/m$^3$ range)</td>
<td></td>
</tr>
<tr>
<td>Linear Shrinkage</td>
<td>0 - 1%</td>
<td>Alley (1996)</td>
</tr>
<tr>
<td>Permeability</td>
<td>1.5 x $10^{-7}$ m/s (undisturbed)</td>
<td>Birrel and Packard (1953)</td>
</tr>
<tr>
<td></td>
<td>1x$10^{-7}$ m/s (in situ test)</td>
<td>Sanders (1986)</td>
</tr>
<tr>
<td></td>
<td>2x$10^{-8}$ m/s (remoulded)</td>
<td>Tehrani (1988)</td>
</tr>
<tr>
<td></td>
<td>1.41x$10^{-8}$ m/s (remoulded Ahuriri)</td>
<td>Jowett (1995)</td>
</tr>
<tr>
<td>Internal Angle of Friction</td>
<td>$\phi$=30 (Peak, direct shear: drained)</td>
<td>Goldwater (1990)</td>
</tr>
<tr>
<td>Cohesion</td>
<td>0 - 20kPa (Peak, direct shear: drained)</td>
<td>Goldwater (1990)</td>
</tr>
<tr>
<td>Total Dissolved Salts in Pore Water</td>
<td>1me/100g (A horizon) to 60 me/100g (C horizon)</td>
<td>Miller (1971)</td>
</tr>
<tr>
<td>Exchangeable Sodium %</td>
<td>0.9 in B horizon to 41 deep in C horizon</td>
<td>Hughes (1970)</td>
</tr>
<tr>
<td>Quantitative Pinhole Erodability Index</td>
<td>B horizon 0.2 - 0.5</td>
<td>Schafer and Trangmar (1978)</td>
</tr>
<tr>
<td></td>
<td>Cx horizon: 2.2 - 12.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C horizon: 10 - 19</td>
<td></td>
</tr>
<tr>
<td>Dispersion crumb class)</td>
<td>B horizon 2 - 4</td>
<td>Yetton (1986)</td>
</tr>
<tr>
<td></td>
<td>Cx horizon: 2 - 3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C horizon: 2 - 4</td>
<td></td>
</tr>
<tr>
<td>Uniaxial Expansion</td>
<td>13.8% Confined Uniaxial Expansion</td>
<td>Jowett (1995)</td>
</tr>
<tr>
<td>Linear Shrinkage</td>
<td>0.27%</td>
<td>Jowett (1995)</td>
</tr>
<tr>
<td>Optimum moisture</td>
<td>15%</td>
<td>Jowett (1995)</td>
</tr>
<tr>
<td>Slake Durability Index</td>
<td>0 Jar Slake Test</td>
<td>Jowett (1995)</td>
</tr>
<tr>
<td>Slaking Class</td>
<td>1 Jar Slake Test</td>
<td>Jowett (1995)</td>
</tr>
</tbody>
</table>
1.6. Slope Stability and Erosion Problems on the Port Hills

1.6.1. General

In addition to in situ loess, Bell and Trangmar (1987) identify four additional soil types on the Port Hills, which are a combination of loess and volcanic material. These soil types display a range of erosion processes that can create engineering problems. Six main erosion processes have been associated with these soil types, and are outlined in Table 1.3. The five soils seen on the Port Hills are:

I) in situ loess (primary airfall loess)

II) loess colluvium (reworked loess with <10% volcanic material)

III) volcanic colluvium (reworked weathered volcanic fragments <10% loess)

IV) mixed colluvium (10-90% loess with 10-90% volcanic material - reworked)

V) residual volcanic regolith

Table 1.3 Erosion processes on Banks Peninsula (From Bell et al 1986)

<table>
<thead>
<tr>
<th>SLOPE TYPE</th>
<th>SCIENTIFIC IDENTIFICATION</th>
<th>EROSION PROBLEM</th>
<th>ENGINEERING OPTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>In situ loess</td>
<td>1. In situ loess, loess colluvium</td>
<td>1. Structural damage due to gradient exceeding soil strength</td>
<td>1. Avoidance of likely failure by soil identification (e.g., airfall sorting)</td>
</tr>
<tr>
<td>Mixed soil</td>
<td>2. Structural damage due to gradient exceeding soil strength</td>
<td>2. Control techniques that minimize gradient disturbance</td>
<td></td>
</tr>
<tr>
<td>Losses of soil</td>
<td>3. Structural damage due to gradient exceeding soil strength</td>
<td>3. Control techniques that minimize gradient disturbance</td>
<td></td>
</tr>
</tbody>
</table>
| Volcanic colluvium | 4. Structural damage due to gradient exceeding soil strength | 4. Soil stabilization by soil modification (e.g., filling)
| 5. Structural damage due to gradient exceeding soil strength | 5. Soil stabilization by soil modification (e.g., filling) |

Table 1.3 Erosion processes on Banks Peninsula (From Bell et al 1986)
Of particular relevance are the processes of tunnel gully, sheet and rill erosion (as described in table 1.3). These will be discussed briefly in the following sections.

1.6.2. Tunnel Gully Erosion
The principal erosion process of concern in the design of most retaining walls on the Port Hills is that of tunnel gullying. Jowett, (1995) summarises the process of tunnel gully formation as follows:

"1) Depletion of vegetation cover and hot, dry conditions promotes soil desiccation. The resulting soil fissures extend to sub-surface soil layers, and subsequent seasonal wetting and drying causes the soil fissures to enlarge. The infiltration capacity of the topsoil is decreased by sun baking. As a result, the volume and velocity of surface run-off is increased.
2) The soil fissures allow infiltration of surface run-off. Subsoil void enlargement and interconnection caused by clay mineral dispersion and slaking mechanisms results in the formation of small tunnels.
3) Tunnel enlargement by erosion eventually leads to roof collapse."

Dispersion (commonly relating to high exchangeable sodium percentages) and slaking have been identified as the primary soil geotechnical properties influencing the formation of tunnel gullies (Bell and Trangmar, 1987). Figure 1.8 shows models for shallow and deep tunnel gully formation. Figure 1.9 shows mature tunnel gullies near the Ahuriri quarry.

Figure 1.8 Models for shallow and deep tunnel gully formation in situ loess and loess-colluvium regoliths. (From Bell & Trangmar, 1987)
1.6.3. Sheet and Rill Erosion.

These processes are particularly prevalent on in situ loess or loess-colluvium slopes where vegetation cover is inadequate, or absent (as is commonly the case during construction). Sheet erosion is associated with rapid surface runoff resulting from heavy rainfall, or in some rare cases, excessive domestic watering, and is exacerbated by increasing slope angle. Dry conditions result in desiccation, and when followed by heavy rainfall, allows soil particles to be easily dislodged by surface water flow or raindrop impact. Where sheet erosion occurs, approximately uniform volumes of soil are removed over the affected area.

Commonly, small-scale flow paths may develop during sheet erosion as a result of minor variations in soil profile, or areas of preferential erosion. Rill erosion occurs when these small flow paths connect, and provide larger scale flow paths along which water flow (and subsequent erosion) is concentrated. These rills may become up to half a meter deep over a single high energy rainfall event, in specific conditions, and thus can pose significant problems. Bell and Trangmar (1987) recommend maintenance of continuous vegetation cover, and suggest caution when topsoil is removed during development. Figure 1.10 shows erosion features at the Ahuriri quarry.
Figure 1.11. a. Extensive rill erosion at the Ahuriri quarry (sunglasses approx. 15 cm)

Figure 1.11. b. Sheet and deeply incised rill erosion at the Ahuriri quarry.
Chapter 2
Geosynthetics
Chapter 2: Introduction to Geosynthetics Terminology and Design

2.1. General Introduction

The term geosynthetics is a general one used to describe all those polymer-based products used in the construction of engineering structures or systems. Development of geosynthetics began shortly after a major disaster in the Netherlands in 1953 where floods killed about 2,000 people. In an attempt to construct protection works as soon as possible, alternatives to traditional methods and materials were developed (Van Santvoort, 1994).

Since the initial development of the field of geosynthetics, there has been a rapid development in the range of available products. With increasing diversity, continuing research and successful case histories the use of geosynthetics has seen a rapid rise in worldwide expenditure on these products. Figure 2.1 shows the growth in usage of geosynthetics between 1970 and 1995 for North America in US dollars, with the total for 1995 being $US1670 million (Koerner, 1997).

Figure 2.1 Rapid rise in North American expenditure on geosynthetics (From Koerner, 1997)
The large range of products, materials and uses covered by the term geosynthetics should be emphasised. Under the heading of geosynthetics comes a series of product groups, each of which has a large amount of variation in production techniques, materials and therefore applications. Table 2.1 is a brief list of some of the available product groups, their most desired function and typical applications in which they are used. The comparatively low usage by 1995 of geonets, geogrids, geosynthetic clay liners and geocomposites as seen in figure 2.1 is perhaps indicative of their relatively young age in terms of development, research and case-histories. As none of these groups have been investigated in this study, the reader is directed to Koerner (1994, 1997) for more detail.

Table 2.1. Common geosynthetics, their functions and applications.

<table>
<thead>
<tr>
<th>Product group</th>
<th>Most desired function</th>
<th>Typical application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geogrid</td>
<td>strength and separation</td>
<td>Roading, slope stabilisation</td>
</tr>
<tr>
<td>Geotextile</td>
<td>separation and filtration</td>
<td>Roading, retaining wall filter</td>
</tr>
<tr>
<td>Geo-pipe</td>
<td>Drainage</td>
<td>Strip drains in roading or retaining walls</td>
</tr>
<tr>
<td>Geonet</td>
<td>Drainage</td>
<td>Subsurface drainage of sports field</td>
</tr>
<tr>
<td>Geocell</td>
<td>Strength</td>
<td>Embankment foundation strengthening</td>
</tr>
<tr>
<td>Geomembrane</td>
<td>Impermeability</td>
<td>Prevention of leaching from landfills</td>
</tr>
<tr>
<td>Geocomposite</td>
<td>Variable</td>
<td>Wide ranging, eg drainage</td>
</tr>
</tbody>
</table>

Note: 1) Properties and applications discussed further in section 2.3.4, specifically for geotextiles.

All of the above have properties that can be useful in a wide range of situations and are therefore seen in a correspondingly large range of applications. The group referred to as geocomposites are an ever increasingly diverse range of combinations of the other geosynthetic products. Geocomposites are commonly designed and produced for the specific requirements of individual projects.

Within this study, the two geosynthetic product groups that have been used are geotextiles and geo-pipe. Since geo-pipe products have not been directly investigated, only general information will be given on geo-pipe, whilst geotextiles will be discussed in some detail.
2.2. Geo-pipe

2.2.1. Terminology

The term geo-pipe, is defined by Koerner (1994) as:

"Any plastic pipe used with foundation, soil, rock, earth, or any other subsurface related material as an integral part of a human-made project, structure, or system"

The products most commonly associated with the term geo-pipe are in the basic design of a traditional pipe, for example, Novaflo®. Figure 2.2 shows a typical application for Novaflo®.

Figure 2.2 Novaflo® removing excess water from permeable non-sandy soil (James Hardie Pipelines pamphlet, 1997)
A number of variations on this basic theme have been developed to better suit particular situations. An example is Megaflo® which has a number of specialised design features including: a geotextile wrap (and could therefore also be classified as a geocomposite) to reduce the potential of the drain clogging; and a range of heights to enable vertical as well as horizontal drainage (traditional pipes are only able to provide one or the other depending on installation). These features make this product ideal for strip drainage situations such as behind a retaining wall, and may reduce backfill requirements. Figure 2.3 shows Megaflo® being installed in a typical strip drain application.

Figure 2.3 Installation of Megaflo® as a road-side strip drain (Geofabrics pamphlet, 1997)

2.2.2 Specifications of geo-pipe products used in this study

There have been two products used over the duration of this thesis that come under the geo-pipe classification: Megaflo®, and Novaflo®. The full range of available specifications for each product is given in the following figures.
Figure 2.4.a Megaflo® specifications (Geofabrics pamphlet, 1997)

<table>
<thead>
<tr>
<th>Panel</th>
<th>Panel Height (Nom)</th>
<th>ASTM D2122</th>
<th>170mm</th>
<th>315mm</th>
<th>450mm</th>
<th>900mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Panel Width</td>
<td>ASTM D2122</td>
<td>&gt;40mm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slot Size (Min)</td>
<td>ASTM D2122</td>
<td>2.0mm X 25mm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Compressive Strength</td>
<td>ASTM D2412 (Mod)</td>
<td>&gt;200 kPa</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Geotextile</th>
<th>CBR Burst</th>
<th>AS 3706.4.90</th>
<th>&gt;1800 N</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pore Size (09)</td>
<td>AS3706.7.90</td>
<td>&lt; 230 μm</td>
</tr>
<tr>
<td></td>
<td>Permeability</td>
<td>AS 3706.9.90</td>
<td>&gt;35 X 10^-4 m/s</td>
</tr>
</tbody>
</table>

*NOTE Specification subject to change at any time without notice.*

Figure 2.4.b Megaflo® flow chart (Geofabrics pamphlet, 1997)

Flow elevation maintained @ one third drain height to ensure water does not re-enter pavement.
Figure 2.5 Novaflo® specifications (Hardie iplex Technical manual, c1985)

a Dimensions and statistics

<table>
<thead>
<tr>
<th>Nominal diameter (De)</th>
<th>160mm</th>
<th>110mm</th>
<th>65mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual Outside mean diameter (Dm)</td>
<td>160.3mm</td>
<td>110.2mm</td>
<td>69mm</td>
</tr>
<tr>
<td>Actual Inside mean diameter (Di)</td>
<td>139mm</td>
<td>94mm</td>
<td>56mm</td>
</tr>
<tr>
<td>Mean Weight/m</td>
<td>835g</td>
<td>430g</td>
<td>225g</td>
</tr>
<tr>
<td>Length of pipe per coil*</td>
<td>45m</td>
<td>100m</td>
<td>150m</td>
</tr>
<tr>
<td>Inlet area/metre length ‡</td>
<td>6429mm²</td>
<td>7980mm²</td>
<td>7590mm²</td>
</tr>
<tr>
<td>% Inlet water area per continuous length of pipe</td>
<td>1.47%</td>
<td>2.7%</td>
<td>4.31%</td>
</tr>
</tbody>
</table>

* 65mm & 110mm Novaflo are also available in compact easy to handle 30m coils.
‡ Novacoil has nil inlet area for all sizes.

b Safe loads on Novaflo®

<table>
<thead>
<tr>
<th>De (mm)</th>
<th>W(kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>65</td>
<td>1.96</td>
</tr>
<tr>
<td>110</td>
<td>3.54</td>
</tr>
<tr>
<td>160</td>
<td>5.16</td>
</tr>
</tbody>
</table>

This gives an indication of the safe load magnitude involved for the above trench conditions.

N.B. Attempts to correlate these safe pipe loads with the test results from ASTM D2412-77 cannot be made because in that test, the pipe is unsupported laterally (no haunching).
Figure 2.5 cont. Novaflo® specifications (Hardie iplex Technical manual, c1985)

Graph of discharge from Novaflo® vs pipe gradient

Based on Manning's formula

\[ V = \frac{1}{\sqrt{n}} \frac{1}{R} \left( \frac{A}{P} \right)^{1/2} \]

where

- \( V \) = mean velocity (m/s)
- \( r \) = hydraulic mean radius
- \( A \) = area (internal)
- \( P \) = wetted perimeter
- \( Q \) = flow
- \( S \) = slope (m/m)
- \( n \) = Manning's roughness coefficient

*It should be noted that this formula as published in Ref 10.14 is incorrect and has been verified as published herein.*
2.3. Geotextiles

2.3.1 Terminology
The term geotextile is defined by Hoan (1987) as:

"a permeable synthetic membrane designed for use with geotechnical materials as an integral part of a man-made project, structure or system."

2.3.2 Background
Geotextiles have been in use since about 1957 (Van Santvoort, 1994). One of the early papers that appears in the literature on geotextiles is Barrett (1966). This paper describes the use of some of the first geotextiles in the late 1950’s. The primary application at this stage, was that of erosion control for sea walls and the like with the primary function of filtration. One of the first major tests for geotextiles was when Koerner used Bidim® nonwoven geotextiles in the Valcros dam, France in 1970. A study 6 years post installation of the dam showed the geotextiles were performing satisfactorily (Giroud et al, 1977). Since the installation of the Valcros dam, the development of geotextiles has seen them used in an ever increasing range of applications. The following should be regarded as only a brief account of the subject of geotextiles, as it is a huge field with a great many additional complexities.

2.3.3. Production Techniques
The production of geotextiles is in itself a very complicated and involved topic as is illustrated by figure 2.6. It should be noted that the “special geotextiles” referred to are not included in all definitions of geotextiles. As this complexity can be confusing, the following summary is an attempt at a basic introduction to geotextiles.
Figure 2.6: Production of Geotextiles (From Yarn to Fabric, 1994)
Although there are a range of different geotextile types, each with their own method of manufacture, the basic process is similar for all. It begins with a reservoir of molten polymer (for example polyester), which is extruded into various types of fibers. There are three basic fiber types that are used in a variety of ways to produce various geotextiles (terminology modified from Giroud and Carroll (summer 1983):

1) Filaments: continuous string - like fibers that are often drawn along their long axis shortly after being extruded to align the molecules and therefore increase strength.

2) Staple fibers: these are simply filaments that have been cut into lengths of between 2 and 10cm.

3) Slit films: are 1-3mm wide tape - like fibers produced by slitting extruded plastic film with blades. As with the other fibers, these are drawn along the long axis to align molecules and increase strength.

Once extruded, the material is arranged, and then bonded in a range of ways crucial to the performance characteristics of the geotextile. Geotextiles are classified by their various production methods. The three broad categories are: woven, knitted and nonwoven; these are outlined in the following subsections.

A) Woven:
These are produced by weaving a particular type of fiber or collection of fibers known as yarn. Yarn types include: monofilament (single filament), multifilament (several filaments aligned together - see figure 2.7a), spun (staple fibers interlaced and twisted together), slit film (single slit film fiber, perhaps the most common yarn - see figure 2.7b), and fibrillated (film that has been nicked and broken up into fibrous strands). The yarn type, and a number of different weaving techniques allow the pore size and total percent of open area to be varied. Although the weaving provides the primary bonding, in some cases heat bonding is used in addition (see 2.3.3.C.I). Woven geotextiles are used most widely for separation applications (eg roading).
Chapter 2: Geosynthetics

Filtration of Port Hills Loess for Retaining Wall Situations

Figure 2.7. (From Koerner, 1994)
a. Micrograph of a multifilament woven geotextile b. Micrograph of a slit film woven geotextile

B) Knitted:
As with traditional knitting, knitted geotextiles are bonded by yarns (as explained above) being interlocked into a series of loops to form a flat sheet. There are a variety of knitting techniques and yarns used. Knitted geotextiles are, however, rarely used, due to their tendency for excessive elongation under tension (Ingold and Miller, 1988). The advantage of the knitted geotextiles is the high strength gained from the knitting of the fibres. Figure 2.8 shows images of a knitted geotextile.

Figure 2.8. (From Ingold and Miller, 1988)
a Micrograph of the base of a knitted geotextile b Upper surface of a knitted geotextile
C) Nonwoven:
These are produced with continuous filaments or staple fibers. The filaments or fibers are typically placed in a random manner, although some specialised geotextiles require an oriented pattern. In either case, an even thickness is important to the properties of the geotextile. Once the filaments are placed in a sheet, the bonding process is then initiated. As mentioned earlier, bonding is crucial to the performance characteristics of the geotextile; three distinctly different bonding methods are commonly used for nonwovens:

I. Heat bonding:
This involves the laid out filament sheet being compressed under heat. This has the effect of partially melting the polymers, resulting in the cross over points becoming fixed together on cooling, and reducing the thickness of the sheet significantly. Typically, heat bonded non-woven geotextiles (such as the Terram® range) are thin and have some degree of rigidity. Figure 2.9.a&b show a heat bonded non-woven geotextile in plan and cross section respectively as viewed under a scanning electron microscope (SEM) during this study.

II. Chemical bonding:
This relies on chemicals to bond the filaments together. Bonding agents are most frequently synthetic resins, but can be glues, rubber, latex, or cellulose derivative. The bonding agent used depends on the required properties of the geotextile. No chemically bonded geotextiles have been tested in this study.
III. Needle punching:
This method uses thousands of needles of a known size to “punch” through the filament sheet and then withdraw. The effect is two fold; firstly it creates pores of a known size. Secondly, the needles are designed so that they will act to entwine the filaments together, giving the required bonding to achieve acceptable strength of the product. Figure 2.10 a&b shows SEM images of a plan view and cross section of a needle punched nonwoven geotextile used in this study.

![Figure 2.10. SEM images of a needle punched nonwoven geotextile (Bidim® A24)](image)

a Plan view  
b. Cross section

The reader is directed to the following references for additional information on geotextile production techniques, terminology, etc:

  A.A.Balkema, Rotterdam


  Butterworths Publishing, London

2.3.4. Applications
Geotextiles have been used in a wide range of applications. The main reason for this is the large number of useful properties a geotextile provides in a relatively cheap and easy to install package: permeable (cross-plane, and in-plane); flexible; strong; range of pore size distributions; relatively light weight; thin. In many cases one or two of these properties may be sufficient to justify the use of a geotextile, and the other properties may simply be of additional benefit. Koerner (1994) lists a total of 96 “major uses of geotextiles”, and this
does not include applications of impregnated geotextiles. Since it is impractical to discuss all of these applications, the following is a brief account of some of the most common applications for geotextiles in engineering today.

A) Separation in roading (paved and unpaved), railroads, embankments etc.

In these applications a geotextile is placed between two materials that have a natural tendency to mix under any significant normal stress. Therefore, the main considerations are of strength, and secondarily, permeability and retention (see later). Figure 2.11 shows the diverse range of situations where geotextiles are used for separation.

Figure 2.11  Example applications with geotextiles acting as separator (From Giroud, 1981)

<table>
<thead>
<tr>
<th>(a)</th>
<th>(b)</th>
<th>(c)</th>
<th>(d)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(e)</td>
<td>(f)</td>
<td>(g)</td>
<td>(h)</td>
</tr>
<tr>
<td>(i)</td>
<td>(j)</td>
<td>(k)</td>
<td>(l)</td>
</tr>
<tr>
<td>(m)</td>
<td>(n)</td>
<td>(o)</td>
<td>(p)</td>
</tr>
<tr>
<td>(q)</td>
<td>(r)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

a) embankment on a soft soil, b) sand cushion wrapped with geotextile, c) gabions on soft soil, d) embankment built under water, e) base of dam on a cracked rock, f) storage of granular material, g) retaining structure with wire mesh, h) sheet pile retaining wall, i) storage area, j) working area, k) parking lot, l) board road, m) unpaved road, n) railway track, o) beach, p) sport ground q) race course or track field r) sidewalk with concrete slabs.
B) Reinforcement for retaining walls, embankments, and unstable slopes, etc
These applications typically involve the construction or remediation of a slope or wall taking full advantage of the flexibility, strength and frictional properties provided by geotextiles. Again, this application is one of many variations, and continuing development.

Figure 2.12a shows the basic steps in the construction of a “geotextile wall”. Figure 2.12b shows the same type of wall almost at completion. Due to the sensitivity of most geotextiles to ultra violet (UV) radiation, these walls are typically covered with shotcrete, or bitumen (Koerner 1994).

**Figure 2.12.a Construction sequence of a “geotextile wall” (From Koerner, 1994)**

---

**Figure 2.12.b Almost complete geotextile wall (From Koerner, 1994)**
C) Filtration for retaining walls or strip drains etc

The requirements of filtration applications are typically more hydrologic than mechanical (as with A and B), since they require the geotextile to allow water to pass through virtually unimpeded, whilst keeping the amount of soil passing to a minimum. Filtration is a requirement in many areas as indicated by figure 2.13 below. Since filtration applications form the basis of this thesis, they will be discussed in detail throughout.

Figure 2.13 Various filtration applications for geotextiles. (modified from Koerner, 1994).
D) Summary

Many of the applications in which geotextiles are used, and the level of importance of particular geotextile properties in each application, are summarised well in figure 2.14. This figure also illustrates the range of factors that may be considered when designing with geotextiles.

Figure 2.14 Summary of geotextile applications and importance of geotextile properties. (From Ingold, 1994).

<table>
<thead>
<tr>
<th>GEOTEXTILE APPLICATION</th>
<th>FUNCTION</th>
<th>DURABILITY</th>
<th>STRUCTURE</th>
<th>HYDRAULIC</th>
<th>MECHANICAL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>REINFORCEMENT</td>
<td>DRAINAGE</td>
<td>FILTERATION</td>
<td>CHEMICAL</td>
<td>UV LIGHT</td>
</tr>
<tr>
<td>UNPAVED ROADS</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>PAVED ROADS</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>RAILROADS</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>EMBANKMENTS ON WEAK GROUND</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>STEEP SIDED EMBANKMENTS</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>WALLS</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>WALL DRAINS</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>TRENCH DRAINS</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>TOXIC CUT-OFF DRAINS</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>CANAL REVETMENTS</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>FABRIC FORMWORK</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>LINERS TO GEOMEMBRANES</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
</tbody>
</table>

* Generally composite structures, + may be composites.
2.3.5. Specifications of geotextiles used in this study

For the purposes of this thesis products tested have not been chosen by use of traditional criteria. The approach has been to test those products currently in use (as suggested by practising engineers, and from observations in the field), for filtration in retaining wall situations.

To this end, geotextiles from two product ranges have been used in this study: Terram® (heat bonded continuous fibre nonwoven – polypropylene/polyethylene mixture); and Bidim® (needle punched continuous fibre nonwoven - polyester). Product specifications as available are given in tables 2.2 and 2.3. The geotextiles that form the main focus of this study are: Bidim®: A14, A24, A34, Terram®: T1000, T1500

Minimal testing on the following has also been carried out: Bidim®: A12, A44, A64

See chapter 4 for a discussion of the testing programme.

Table 2.2 Terram® product specifications (Information from Terram® data pamphlet, 1996)

<table>
<thead>
<tr>
<th>TYPICAL VALUES</th>
<th>TEST METHOD</th>
<th>UNITS</th>
<th>T1000</th>
<th>T1500</th>
</tr>
</thead>
<tbody>
<tr>
<td>MECHANICAL PROPERTIES</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WIDE-STRIP TENSILE STRENGTH</td>
<td>BS6906 Part 1: 1987</td>
<td>kN/m</td>
<td>8.0</td>
<td>13.0</td>
</tr>
<tr>
<td>TRAPEZOIDAL TEAR RESISTANCE</td>
<td>ASTM D4533</td>
<td>N</td>
<td>250</td>
<td>275</td>
</tr>
<tr>
<td>CBR PUNCTURE RESISTANCE</td>
<td>BS6906 Part 4: 1989</td>
<td>N</td>
<td>1200</td>
<td>1900</td>
</tr>
<tr>
<td>GRAB TENSILE STRENGTH</td>
<td>ASTM D4632</td>
<td>N</td>
<td>550</td>
<td>1100</td>
</tr>
<tr>
<td>INDICATIVE HYDRAULIC PROPERTIES</td>
<td>TEST METHOD</td>
<td>UNITS</td>
<td>T1000</td>
<td>T1500</td>
</tr>
<tr>
<td>PORE SIZE (mean AOS)</td>
<td>BS6906 Part 2: 1989</td>
<td>O₉₀ - H,m</td>
<td>100</td>
<td>&lt;60</td>
</tr>
<tr>
<td>FLOW RATE UNDER 100 mm HEAD</td>
<td>BS6906 Part 3: 1989</td>
<td>l/m².s</td>
<td>40</td>
<td>15</td>
</tr>
<tr>
<td>PHYSICAL PROPERTIES</td>
<td>TEST METHOD</td>
<td>UNITS</td>
<td>T1000</td>
<td>T1500</td>
</tr>
<tr>
<td>MASS PER UNIT AREA</td>
<td>BS EN 965 1995</td>
<td>g/m²</td>
<td>135</td>
<td>200</td>
</tr>
<tr>
<td>ROLL WIDTH</td>
<td>[n/a]</td>
<td>m</td>
<td>4.5</td>
<td>4.5</td>
</tr>
<tr>
<td>ROLL LENGTH</td>
<td>[n/a]</td>
<td>m</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>TYPICAL VALUES</td>
<td>TEST METHOD</td>
<td>UNITS</td>
<td>A12</td>
<td>A14</td>
</tr>
<tr>
<td>------------------------------</td>
<td>-------------</td>
<td>-------</td>
<td>------</td>
<td>------</td>
</tr>
<tr>
<td>MECHANICAL PROPERTIES</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WIDE-STRIP TENSILE STRENGTH</td>
<td>AS3706.2-90</td>
<td>kN/m</td>
<td>8.9</td>
<td>10.2</td>
</tr>
<tr>
<td>TRAPEZOIDAL TEAR STRENGTH</td>
<td>AS3706.3-90</td>
<td>N</td>
<td>244</td>
<td>288</td>
</tr>
<tr>
<td>CBR BURST STRENGTH</td>
<td>AS3706.4-90</td>
<td>N</td>
<td>1654</td>
<td>1967</td>
</tr>
<tr>
<td>G RATING</td>
<td>Austroads</td>
<td>G</td>
<td>&gt;1290</td>
<td>&gt;1450</td>
</tr>
<tr>
<td>GRAB TENSILE STRENGTH</td>
<td>AS2001.2.3</td>
<td>N</td>
<td>492</td>
<td>609</td>
</tr>
<tr>
<td>INDICATIVE HYDRAULIC</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PROPERTIES</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PORE SIZE - DRY SIEVING</td>
<td>AS3706.7-90</td>
<td>O₉₅ - Hₘ</td>
<td>220</td>
<td>210</td>
</tr>
<tr>
<td>PERMITTIVITY</td>
<td>AS3706.9-90</td>
<td>S⁻¹</td>
<td>3.0</td>
<td>2.6</td>
</tr>
<tr>
<td>COEFFICIENT OF PERMEABILITY</td>
<td>AS3706.9-90</td>
<td>x10⁴m/s</td>
<td>50.0</td>
<td>50.0</td>
</tr>
<tr>
<td>FLOW RATE UNDER 100 mm HEAD</td>
<td>AS3706.9-90</td>
<td>l/m²/s</td>
<td>300</td>
<td>260</td>
</tr>
<tr>
<td>PLANAR FLOW RATE - J=1.0</td>
<td>ASTM D4716</td>
<td>l/hr/m.width</td>
<td>10.4</td>
<td>10.8</td>
</tr>
<tr>
<td>(under 100kPa normal</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>compressive stress)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PHYSICAL PROPERTIES</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TEST METHOD</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MASS PER UNIT AREA*</td>
<td>AS3706-1</td>
<td>g/m²</td>
<td>120</td>
<td>140</td>
</tr>
<tr>
<td>THICKNESS*</td>
<td>AS3706-1</td>
<td>mm</td>
<td>1.4</td>
<td>1.5</td>
</tr>
<tr>
<td>(pressure 2.0 kPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ROLL WIDTH</td>
<td>[n/a]</td>
<td>m</td>
<td>4</td>
<td>4 or 2</td>
</tr>
<tr>
<td>ROLL LENGTH</td>
<td>[n/a]</td>
<td>m</td>
<td>200</td>
<td>200 or 50</td>
</tr>
</tbody>
</table>

Table 2.3 Bidim® product specifications (Information from Geofabrics pamphlet, 1996)

*Data from Geofabrics “A” range technical data: issue 2 effective 1/3/91
2.4. Design Methodologies
This section looks at the general approaches to choosing a geotextile for a specific job, and at the advantages and disadvantages of each design process.

2.4.1 Introduction
In theory, a great many factors need to be considered as part of the design process. In practice, however, it may often be the case that the budget of the project does not allow the detailed investigation of all variables necessary to come to satisfy all theoretical requirements. Although the following can be more generally applied to geosynthetics as a whole, this discussion will focus on geotextiles as does this thesis. As a result of the conflicts between money, time and theory, Koerner (1994) identifies three main approaches to designing with geotextiles:

2.4.2. Design by cost and availability.
This approach involves the selection of the geotextile to be used based on the maximum allowable unit cost. That is, the budget is set for geotextiles and the available material with the “best properties” within that budget is used. This approach has obvious drawbacks, particularly with the selection of the “best properties” being based on little or no empirical information of the specific situation. The other most obvious problem with this is the setting of a budget (which may be done in many situations by non-engineers), thereby potentially eliminating more suitable options.

2.4.3. Design by specification
This design methodology involves the classification of the specific situation into general application categories. Required minimum properties are tabulated for each category, along with the specific test used to define each property (eg: grab tensile strength test method: ASTM D1682). Koerner (1994) notes that this approach is commonly used with public agencies. This may be a result of the need for large numbers of projects to be completed quickly and at reasonable cost with the “look-up table” method being faster and cheaper than more involved methodologies.

There are advantages of this method over design by cost and availability. Specifically, this approach requires minimum performance properties, rather than minimum
cost. However, the provision does exist where more than one geotextile is found acceptable to make the final choice by price and availability. Also, this method requires the specific identification of the main application which is usually indicative of the critical properties thereby providing the geotextile selector with better guidelines as to which are the “better property values”. It appears that the minimum specified properties have, in many instances, been set as a result of review of a number of case studies, thereby strengthening the justification for this design approach.

Drawbacks of a design by specification approach are less obvious than for design by cost and availability, but still significant. The primary concern is the general nature of the application categories. These do not allow for the large amount of variation that can exist within a particular application. This problem can be, and is occasionally addressed by the addition of further subdivisions within the broad categories, and the inclusion of new case-study data. Also, as many of the specification tables are drawn for specific areas of specific countries, variations within these areas may be limited.

Another potential problem with this method outlined by Koerner (1994) is that the specifications outline minimum requirements, whilst manufacturers tend to list “average-lot” values which are not representative of the minimum. To deal with this problem, the concept of a Minimum Average Roll Value (MARV) is being used by the American Association of State Highway and Transportation Officials (AASHTO) and others. A “MARV” value represents the 95% confidence interval as is shown below in figure 2.15.

Figure 2.15 The statistical MARV relationship relating to test values (From Koerner, 1994)

\[
\text{Mean: } \bar{X} = \frac{\sum X_i}{N_i} \quad \text{Standard deviation: } S = \left[ \frac{\sum (X_i - \bar{X})^2}{N - 1} \right]^{1/2} \\
\text{Where } \bar{X} = \text{the mean value} \quad X_i = \text{the measured value} \quad N = \text{the number of measurements}
\]
In the situation where the property is required to be a maximum (eg tensile strength), then the same concept can be applied by adding two standard deviations to obtain a MaxARV.

2.4.4. Design by function

This approach would be seen by most as the "purist approach". That is, this approach requires detailed investigation of required properties, testing to confirm properties, application of numerical design criteria (see section 2.4.2), establishment of a minimum acceptable factor of safety, comparison of tested properties to required properties (with use of the MARV concept) to establish an effective factor of safety for individual geotextiles, the application of compatibility tests where appropriate, and finally an iterative process to establish all available options. Obviously this methodology for design is superior to the earlier mentioned approaches, as it requires site specific information, is not limited by budget, and demands investigation by way of testing and application of numerical criteria to the specific site. A number of other authors have outlined similar systematic design procedures eg Corbet (1992) with a detailed 9 step procedure, and Mckenna (1995). In particular, these authors emphasise the need for accurate characterisation of the soil conditions the filter needs to cope with.

Counteracting the large positives of this design philosophy is the large cost in both monetary (for both testing and expenditure on materials), and time terms. It is therefore most likely that this approach is only strictly followed in projects with budgets capable of sustaining it. Also to be aware of, is that the methods applied in this approach must themselves be considered carefully. As will be discussed later, the subject of numerical criteria is still one for debate. Testing methods are continually being critically evaluated and refined, as is vital in a still relatively young branch of science. Faure (1996) details a series of tests used in an attempt to establish a European standard for wet sieving (used to determine "characteristic opening size" of tested geotextiles); this paper illustrates the potential for confusion without standardisation of testing. Rigo, et al, (1990) note also that many pore size specifications have been overestimated as a direct result of testing procedures. Barroso and Lopes (1998) similarly show that variations in test methods influence measurement of characteristic opening size. It should also be emphasised that comparison of values obtained by different testing methods is not advisable!
As a result of observations of practice on the Port Hills, I propose two further design methodologies that appear to be used frequently as outlined in the next two sections:

2.4.5. Design by experience
This approach involves the selection of a geotextile for a particular site, based on the past experience of the design engineer. Although budget may be a consideration for some cases, it is by no means the deciding factor with this approach. There are a number of significant advantages with this design methodology. In particular, both time and money are saved by the elimination of testing and application of numerical criteria. Depending on the length of experience of the design engineer, a design may be based on a large number of “successful” past case histories.

The reason I say “successful” is that the relatively short history of usage of geotextiles, particularly in New Zealand, means that in most cases less than half of a 50-year design life would have passed up to the present time since installation. This is one of the limitations of the design by experience approach, and is compounded in the situation where the design engineer has a limited number of case histories to call on.

Another advantage of this method, is that the design engineer is directly involved in the selection of the geotextile based on project specific information. This means that the primary functions required of the geotextile have been assessed by an “expert”.

Other disadvantages are potentially significant for this approach. Firstly, if the design engineer does a less than thorough investigation of the site due to similarities to past experiences, subtle variations of soil type, site geometry, etc, may be missed that may have a significant impact on the true requirements of the geotextile for that site. A problem as far as the performance of the geotextile is concerned, is the potential for over-specification. Without an extensive and tested experience base, there may be a tendency for the design engineer to specify a higher grade of geotextile than is necessary, thereby increasing the cost of the project. Another potential of “over-specification” is the selection of a geotextile with too small pore size, according to McKenna (1995), this may increase the potential for clogging. This is particularly important on the Port Hills with loessial soils having a grain size distribution that makes satisfying retention and clogging criteria (discussed later) problematic.
2.4.6. Design by chance

It has come to my attention that the practice of an engineer specifying “a geotextile”, or perhaps a specific geotextile “or equivalent” is still happening. This situation tends to arise on very small projects where budgets are very limited. An example may be a home-owner attempting to install a new retaining wall or repair an existing one, with very limited technical input. The result may be the home-owner ringing a local geotextile distributor and asking for the cheapest geotextile available. Clearly this approach is unsatisfactory in many ways:

- Technical input may be limited, and therefore selection is not based on properties of the geotextile or site specific soil
- As price is a dominant influence, unsatisfactory materials may be selected
- Without detailed plans, poor installation of the geotextile may render it’s properties less effective

2.5. Design Criteria

As mentioned in the previous section, there are a number of factors that must be considered for accurate design of a geotextile system. A number of these factors have been separated into a range of criteria to help subdivide the requirements of a geotextile for any project. Many of these criteria have been put into numerical form so that measurement and calculations can more accurately guide the design process. The following is a very brief list of the main criteria that might be considered when designing a retaining wall with a geotextile as the filter:

- Survivability: is it strong enough to survive the installation process
- Durability: will it last for the design life of the project
- Permeability: will it allow water to pass through virtually unimpeded
- Retention: will it stop the break down of the soil structure due to soil loss
- Clogging: will it keep sufficient open pore space to maintain permeability

As a number of researchers have investigated these criteria, there are a range of numerical calculations associated with each. Since the survivability and durability criteria have not been investigated as part of this study, only a brief mention of these will be given, while the others mentioned above will be covered in more detail.
2.5.1. Survivability
This criterion commonly requires investigation of several properties of a geotextile, to ensure it will remain intact and perform as expected following the installation procedure. Typical properties investigated are:

- Tensile strength
- CBR puncture resistance
- Tear resistance
- Burst strength

As these are measurable mechanical properties, testing and application of criteria can be done with confidence. That is providing testing methods follow standard procedures. A more in depth coverage of this topic can be found in: Richardson and Wyant, (1987), Koerner (1994), Van Santvoort (1994).

2.5.2. Durability
As with survivability, several properties of a geotextile are investigated in durability criteria, this time to ensure the geotextile will perform satisfactorily over the design life of the project:

- UV radiation resistance
- Biological resistance
- Chemical resistance

Another aspect that could be considered as a durability concern is clogging. It is vitally important that clogging does not take place, especially in filtration applications. As it can occur in a short time, and also because of its complexity, clogging is generally treated as a criterion in need of independent investigation and is discussed in section 2.5.5. Although most of the above properties can be tested relatively easily, it is difficult to accurately simulate these effects over a prolonged period (ie many years). To cope with this, there is a need to closely monitor the performance of long-running field projects in conjunction with accelerated laboratory experiments.
2.5.3. Permeability

Simply put a permeability criterion is a calculation to ensure fluid (usually water) will pass relatively uninhibited through the filter for the duration of the project’s design life. Giroud, (1982) presents a derivation of the permeability criteria:

\[ k_{\text{geotextile}} > 0.1k_{\text{soil}} \]  

\((k=\text{coefficient of permeability in m/s})\)

This criterion, like many found in the literature, is based on criteria for granular filters founded by Terzaghi in 1922. A great many of the permeability criteria that have been recommended follow a similar format:

<table>
<thead>
<tr>
<th>Author</th>
<th>Criteria</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Christophor and Holtz (1985)</td>
<td>[ k_{\text{fabric}} \geq 10k_{\text{soil}} ] [ k_{\text{fabric}} \geq k_{\text{soil}} ]</td>
<td>Critical/severe applications Less critical and with sands or gravels</td>
</tr>
<tr>
<td>Murray and McGown (1992)</td>
<td>[ k_{\text{geotextile}} &gt; 10k_{\text{soil}} ] [ k_{\text{geotextile}} &gt; 100k_{\text{soil}} ]</td>
<td>Wovens and thin nonwovens (\leq)2mm Thick nonwovens &gt;2mm</td>
</tr>
<tr>
<td>Giroud (1988)</td>
<td>[ k_{\text{geotextile}} &gt; i.k_{\text{soil}} ]</td>
<td>(i=\text{soil hydraulic gradient of application})</td>
</tr>
<tr>
<td>McKenna (1995)</td>
<td>[ k_{\text{geotextile}} &gt; 10k_{\text{soil}} ]</td>
<td>Equivalent to a factor of safety of 10</td>
</tr>
</tbody>
</table>

Although not specifically stated, most of the above include some amount of safety factor. The two main reasons for this are the reduction in pore size distribution due to confining stress, and the potential for clogging (see 2.5.5). Research by Vermeersch, et al (1997) shows that increases in confining pressure can significantly reduce pore size distributions, particularly with thick nonwoven geotextiles. Some authors (eg Koerner, 1994 and IFAI, 1992) suggest the consideration of permittivity rather than permeability. Since permittivity \((\Psi) = k/t\) (where \(\Psi = \text{permittivity, } k = \text{permeability normal to the plane of the geotextile and } t = \text{geotextile thickness at a specified normal pressure}\), the effect of normal pressure can be readily incorporated into a permeability criterion.
2.5.4. Retention

This criterion attempts to ensure that the passage of fines through the filter will be limited so as to not allow the break down of the soil structure, that is, allow as little as possible soil particles to pass through. As noted by Giroud (1982), this criterion is contradictory to the permeability criterion when the two are formulated too strictly. Therefore, balancing permeability with retention is one of the significant challenges in geosynthetics, and can be complicated further with the inclusion of clogging criteria. As a result of the above mentioned difficulties, as well as the many approaches around the world, the retention criteria show significant variation in the literature as shown in table 2.5.

Table 2.5. Selection of available retention criteria.
adapted from Koerner (1994) and Fischer, Christopher & Holtz (1990)

<table>
<thead>
<tr>
<th>Author</th>
<th>Criterion</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Task Force #25 (1991)</td>
<td>$O_{95}/d_{10} \leq 1$ $O_{95}/d_{20} \leq 1.8$</td>
<td>No limitations on geotextile type or soil type $50% \leq 0.074\text{mm}, O_{95} &lt; 0.59\text{mm}$ $50% &gt; 0.074\text{mm}, O_{95} &lt; 0.30\text{mm}$</td>
</tr>
<tr>
<td>Ogink (1975)</td>
<td>$O_{95}/d_{15} \leq 1$ $O_{95}/d_{30} \leq 1$</td>
<td>Wovens nonwovens</td>
</tr>
<tr>
<td>Sweetland (1977)</td>
<td>$O_{95}/d_{15} \leq 1$ $O_{95}/d_{30} \leq 1$</td>
<td>Nonwovens, soils with $C_{u} = 1.5$ nonwovens, soils with $C_{u} = 4.0$</td>
</tr>
<tr>
<td>Rankilor (1981)</td>
<td>$O_{95}/d_{15} \leq 1$</td>
<td>Nonwovens, soils with $0.25d_{15} \leq 0.25\text{mm}$</td>
</tr>
<tr>
<td>Schober &amp; Teindl (1979)</td>
<td>$O_{95}/d_{30} \leq 2.5-4.5$ $O_{95}/d_{30} \leq 4.5-7.5$</td>
<td>Woven &amp; thin nonwovens dep. on $C_n$ Thick nonwovens, dep. on $C_w$ silt &amp; sand soils</td>
</tr>
<tr>
<td>Millar, Ho &amp; Turnbull (1980)</td>
<td>$O_{95}/d_{30} \leq 1$</td>
<td>Wovens and nonwovens</td>
</tr>
<tr>
<td>Giroud (1982)</td>
<td>$O_{95}/d_{30} \leq 1$</td>
<td>Dependent on soil relative density and $C_n$</td>
</tr>
<tr>
<td>Carroll (1983)</td>
<td>$O_{95}/d_{20} \leq 1.2$ $O_{95}/d_{30} \leq 2.3$</td>
<td>Wovens and nonwovens</td>
</tr>
<tr>
<td>Christopher &amp; Holtz (1985)</td>
<td>$O_{95}/d_{15} \leq 1$ or $O_{95}/d_{30} \leq 0.5$</td>
<td>Dependent on soil type and $C_n$ Dynamic, pulsating, and cyclic flow, if soil can move beneath geotextile</td>
</tr>
<tr>
<td>Fischer, Christopher &amp; Holtz (1990)</td>
<td>$O_{95}/d_{15} \leq 1.8-7.0$ $\beta_1 \leq O_{95}/d_{15} \leq \beta_2$</td>
<td>Cohesionless soils dep. on $C_n$ As above &amp; where $\beta_1$ &amp; $\beta_2$ are bounding curves</td>
</tr>
<tr>
<td>$O_{95}/d_{30} \leq 0.06\text{mm}$</td>
<td>For cohesive fine silts</td>
<td></td>
</tr>
</tbody>
</table>

Notes: $C_u =$ coefficient of uniformity $= d_{u}/d_{10}$ $O_x =$ Opening size in $\mu m$ where $x$ is the percentage smaller than this value $d_i =$ soil grain size in $\mu m$ where $x$ is the percentage smaller than this value

Perhaps the most striking point about the above list of criteria, is the variation of parameters used in the calculations. As noted by Fischer, et al (1990), in order for criteria based on $O_{95}$ (or other non-standard values) to be adopted, these values which are not commonly specified
by manufacturers, would need to be established by way of existing tests, or may require the development of new tests. Although the pursuit of new approaches is valuable to the goal of eventually finding the "best" criteria, such pursuits may be impractical to implement with the current level of standardisation. It appears that the main standard parameter used in retention criteria and that is commonly specified by manufacturers is the $O_{95}$ value. It therefore seems logical that this value be adopted as the standard, with future investigations of retention criteria being based on, or at least converted to, the $O_{95}$ value.

Retention is assisted by the formation of structures at the soil/geotextile interface. Two such structures are identified by Mlynarek, et al (1990):

1) Bridging structure (or network). Also variously known as a natural reverse filter, and a filter cake, this structure is formed by the coarser soil particles retained by the geotextile, preventing the smaller particles from passing through the system. A gradational filter is thereby created within the soil, as a result of the interaction with the geotextile (see figure 2.16.).

2) Vault network. According to Mlynarek, et al (1990) the formation of vault networks "usually occurs in silty or sandy soils with appreciable clay content". These networks allow a geotextile with a given opening size to stop finer particles than expected. This is "believed to result from electrokinetic adsorptive forces between the organic lubricant and/or antistatic agent on the fibres and the soil particles" (Mlynarek, et al, 1990). See figure 2.17 for a schematic representation of the vault network.

Both of these structures may potentially be indicated in a test situation by stable system permeability, and only small amounts of material passing through the soil/geotextile system being investigated.

Figure 2.16 Bridge network (Mlynarek, et al, 1990)  Figure 2.17 Vault network. (Mlynarek, et al, 1990)
2.5.5. Clogging

The term clogging represents the combined effects of three distinct mechanisms, all of which have the potential effect of reducing system permeability (Giroud, 1994). The three mechanisms are (terminology adapted from Giroud, 1994 and Mlynarek et al, 1990):

1) Blinding: fine soil particles form a low permeability layer at or near the surface of the geotextile. See figure 2.18

2) Blocking: soil particles permanently obstruct entrance to fabric pores (common in filters with individual openings such as woven geotextiles). See figure 2.19

3) Clogging: soil particles become trapped by fibres within the structure of the geotextile as a result of non-homogeneity (common in thick nonwoven geotextiles). See figure 2.20

Henceforth the term “clogging” will relate to the combination of these three mechanisms unless expressly stated otherwise. The following figures are all from Mlynarek et al (1990).

Fig. 2.18 Blinding mechanism  Fig. 2.19 Blocking mechanism  Fig 2.20 Clogging mechanism

Clogging criteria attempt to ensure that system permeability does not drop below acceptable levels over the design life of the system. There appears to be a wide variety of vastly different approaches to the problem of clogging in geotextiles. The most common clogging criteria are based on similar parameters to the retention criteria mentioned above, and many researchers are now recommending a single criterion for both retention and clogging as shown in table 2.6.
Another prominent approach to design criteria for clogging, is the use of laboratory testing with soils from the project site with a range of geotextiles, also known as compatibility tests. One such test is the ASTM D5101-90 gradient ratio (GR) test that forms a major part of this thesis. Koerner (1994), Christopher and Holtz (1985) and McKenna (1995) suggest that a GR $\leq 3$ is desirable, as values above this are indicative of clogging. The particular advantage with this approach is the requirement for direct testing of the soil/geotextile system that will be in place in a specific project.

Another approach to clogging design is described by Koerner, (1994), and involves identification of problematic situations. Koerner (1994), lists 4 specific conditions that have significantly increased potential for excessive clogging. If any of these situations exist, then selection of “a relatively open geotextile” (Koerner, 1994) is recommended (typically: wovens with $\geq 8\%$ open area, or nonwovens with $\geq 50\%$ open area), provided downstream drainage measures are sufficient to accept and transport the larger amount of particulate matter that may result than in conventional systems (Koerner, 1994).

### Table 2.6 Combined clogging and retention criteria (modified from Austin, et al, 1997)

<table>
<thead>
<tr>
<th>Author</th>
<th>Criteria</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Holtz, et al (1995)</td>
<td>$O_{95} &lt; 265^{1}$</td>
<td>sandy soils$^{2}$</td>
</tr>
<tr>
<td></td>
<td>$168 &lt; O_{95} &lt; 280$</td>
<td>silty sand</td>
</tr>
<tr>
<td></td>
<td>$37 &lt; O_{95} &lt; 66$</td>
<td>woven, silt</td>
</tr>
<tr>
<td></td>
<td>$37 &lt; O_{95} &lt; 119$</td>
<td>nonwoven, silt</td>
</tr>
<tr>
<td></td>
<td>$O_{95} &lt; 239$</td>
<td>silty sand</td>
</tr>
<tr>
<td></td>
<td>$O_{95} &lt; 95$</td>
<td>silt</td>
</tr>
<tr>
<td>Mlynarek (1995)</td>
<td>$183 &lt; O_{95} &lt; 265$</td>
<td>sand</td>
</tr>
<tr>
<td></td>
<td>$112 &lt; O_{95} &lt; 171$</td>
<td>silty sand</td>
</tr>
<tr>
<td></td>
<td>$50 &lt; O_{95} &lt; 66$</td>
<td>silt</td>
</tr>
<tr>
<td>McKenna (1995)</td>
<td>$80 &lt; O_{95} &lt; 250$</td>
<td>in combination with porosity $&gt; 30%$ under 2kPa confining pressure</td>
</tr>
</tbody>
</table>

Note:  
1) Measurements in micrometers  
2) Where geotextile type not mentioned assume for woven or nonwoven
2.6 Synthesis

2.6.1. Discussion

One of the main problems associated with designing with geosynthetics, is the huge number of variables that can be considered. The sheer number of different available criteria for retention (for example) is indicative of the range of efforts to overcome these variables. It is vitally important that design engineers continue to question and develop the approach to designing with geosynthetics, and take care in assessing the relevance and validity of any criteria that they may apply.

For example, if a simple retention criterion is applied, and a geotextile selected on the basis of manufacturer's specified $O_{95}$ value, without accounting for high confining stresses that may exist in the project, problems may arise. Vermeersch, et al (1997) present research that shows the maximum pore size of some geotextiles can reduce by up to 40% at 50kPa normal stress (compared to 2kPa normal stress). This is clearly a significant point, and one which is not accounted for in most traditional criteria. Vermeersch, et al (1997) propose that for filters or separators under compression, a "pore size distribution under compression" (PSDC) should be considered instead of pore size values (eg $O_{95}$) that are currently used in filter design criteria.

This example is one of hundreds of variables that still need to be considered, tested and scrutinised, before designing with geosynthetics can become a precise science at the theoretical level. It is for this reason that the approach of geotextile selection assisted by testing site specific soils directly with possible geotextiles (such as in the GR test), is applauded, but again, further development of these test methods is also required.

Focus must also be directed toward design methodologies, with the approaches of "design by cost and availability" and "design by chance" in desperate need of being abolished. Although design criteria and testing methods may still be in development, the approaches of design by specification and in particular design by function are seen to be far more appropriate at this stage. Finally, the design by experience approach has a great many advantages, but should only be applied in conjunction with an experience of design by function, an extensive experience base, and a thorough site investigation.
As the field of geosynthetics has only been around for approximately 40 years, with continuing advancements, new products, and approaches, this branch of science could still be considered to be in its infancy.

2.6.2. Conclusions

- Geosynthetics are an ever increasing range of polymer based products used in human made systems. The two groups used within this study are geotextiles (primarily) and geopipe (minor usage).
- Geotextiles are classified by their method of production, which directly influences their properties. Within this study a number of nonwoven needle punched and heat bonded geotextiles have been tested and compared for filtration performance with Port Hills loess.
- Five approaches to design methodology are outlined, with design by function identified as the ideal. The approaches of design by chance and design by cost and availability, in particular, have been identified as cause for concern. Design by experience is seen as an acceptable compromise for non-critical projects, provided the experience base is broad, includes experience of design by function, and is applied in conjunction with thorough site investigations.
- A number of criteria exist for guidance of the design process, and in particular for the selection of an appropriate geotextile for a specific project.
- Numerical criteria for permeability, retention, and clogging exist and may aid the design process. This is provided the application of these criteria is informed with the knowledge that the criteria, along with testing methods used to obtained variables used in the calculations, are still under debate and that some are only applicable to very specific conditions.
Chapter 3
Field Practice
Chapter 3. Current Field Practice with Geotextiles.

3.1. Introduction

3.1.1. General
This chapter focuses on current practice of geotextile installation and retaining wall design with geotextiles in the Canterbury area, with emphasis on the Port Hills. It should be noted that three of the four case studies involve retaining wall situations in loess on the Port Hills, and so comments made from observations are in no way intended to encompass design and installation practice in other applications. However, case study number 4 does indicate installation practice in one other application.

It is important to mention at this point that thorough design is pointless if the installation procedure does not allow the materials to perform their designed tasks. Section 3.5. looks specifically at a case study where the poor installation of a geotextile would have significantly reduced it’s effectiveness. Assuming that the design had been correct, the function may still not have been adequate as a direct result of poor installation. It is therefore just as important to look at installation procedures as it is to consider the theoretical merits of a range of designs.

3.1.2. Objectives
The primary objective of this section is to establish the geometries and installation methods that should be used as examples of “good” practice for use in the stage 2 experiments. In order to achieve this objective, a number of initial objectives must be met, as listed below:
1) Investigate current design methods
2) Investigate current installation methods
3) Identify areas of concern in design and installation
4) Identify aspects of “good” design and installation practice

3.1.3. Introduction to Case Studies and Terminology
The following case studies are based on site visits of approximately one hour each, and therefore should not be considered as full and thorough investigations. However, these visits were sufficient to identify the obvious areas of concern, and to provide an adequate
Mulseal®: bitumen emulsion commonly used for water proofing (usually painted on).
Shelter seal: impermeable bitumen product sandwiched between two layers of polyethylene.
Novaflo®: (see section 2.2)
HDPE: High Density Polyethylene (geomembrane used for water “proofing”)
Siltation: non-technical term used to describe an accumulation of sediment
Silt: non-technical term used to describe sediment (commonly silt sized)

3.2. Case Study Number 1

3.2.1. General Description

A brief investigation was conducted over a 1 hour period at a construction site of a new house. Three walls were looked at, each of which was designed to retain loess as shown in figure 3.1. The first wall, forming one of the new house walls, was almost completed and had been mostly backfilled. The second wall was a garage wall and was in the early stages of construction. The final wall was a completed pole-frame wall that secured the roadway batter slope.

Figure 3.1 Site plan for case study 1
3.2.2. Reason for current works
Owners of the property had decided to subdivide their section, therefore this was a new construction.

3.2.3. Original site conditions
Prior to the current phase of construction, this land had been the back yard of an older house (down slope of this new house).

3.2.4. Current works
Retaining wall 1:
Retaining wall 1 was covered with Mulseal® and with ‘shelter seal’ as an addition (bottom 1m of the wall only) for water proofing. Foam matting approximately 10mm thick was laid directly over top of the water proofing layer in order to protect the under layer from damage during the placement of the backfill. In this case the backfill was tailings (typically rounded greywacke clasts 20-40mm diameter) with the occasional piece of concrete and rare clasts of loess. No concrete bedding was present for support of the Novaflo® drainage piping and in most places the top of the piping was approximately level with the floor slab join. The geotextile used was Bidim® 34, which had been secured at the top of the slope by nails. The geotextile was placed from the top of the slope to the bottom, but was neither layed across the base (under the Novaflo®), nor wrapped around the Novaflo® in any way. Figure 3.2.a shows a projected cross section of the finished wall (see page 55).

Retaining wall 2:
Retaining wall 2 had not yet been water proofed, backfilled or had the geotextile placed. However, the plans for these elements were the same as for wall 1. The cut loess face was approximately 2.5m high and appeared to have a small degree of overhang towards the top. Again there appeared to be no concrete bed for the drainage pipe. Excavation levels were similar to wall 1 (relative to the floor slab). Figure 3.2.b (page 55) shows a representation of the completed wall.
Points to note:
- Novaflo below the level of join between wall and floor slab
- Significant water proofing for wall

Figure 3.2.a. Cross section of wall 1.

Points to note:
- Novaflo below the level of join between wall and floor slab
- Significant water proofing for wall
- Overhangs on soil profile possibly result in gaps between geotextile and loess.

Figure 3.2.b. Cross section of wall 2.
Pole-frame wall:
This wall was approximately 1.5-2m high consisting of tanalised pinus radiata poles. As it had already been completed, detailed inspection was not possible. From what was visible, it appeared that the backfill of this wall was soil (predominantly loess, plus minor amounts of top soil). Drainage appeared to be restricted to seepage between poles. Figure 3.2.c shows this wall in cross section.

![Cross section of wall 3 (pole-frame)](image)

Points to note:
- No provision for drainage other than between poles.
- Soil backfill will retain moisture.

Figure 3.2.c. Cross section of wall 3 (pole-frame).

3.2.5. Observations and Interpretation.

Retaining wall 1:
The main points that stood out during the brief look at this wall were the lack of an impermeable bedding for the Novaflo®, and the presence of some loess clasts in the backfill. As there were only a few fist size clasts of loess in the backfill, this is not seen as a major problem, but a small one that could have been avoided easily. The lack of a concrete bed for the Novaflo® means that any water pooling at the base of the Novaflo® has potential to cause some erosion (due to the dispersive nature of loess), which may potentially lead to a progressive undermining of the wall.
Retaining wall 2:
The first observed concern was the geometry of the cut loess slope, specifically the overhanging nature. This geometry makes satisfactory placement of the geotextile difficult if the same method of securing the geotextile was used as for wall 1. That is, on placement of the backfill the geotextile may be strained unnecessarily which has certain implications (namely alteration of pore size distribution, and potential strength losses). Secondly, if it is decided to use more geotextile than the height of the face in an attempt to contour the face, the possibility exists for wrinkles (small folds) to form during backfilling. Wrinkles have been attributed to failures as a result of clogging (Giroud, 1982) and are therefore extremely undesirable. If the geotextile is fastened too tightly, there is also significant potential for gaps between the geotextile and the loess to form, thereby eliminating the filtering effects of the geotextile in these areas.

The loess slope has already demonstrated some instability by the presence of a small failure (fall failure resulting from stress release cracking associated with excavation and the overhanging nature of the face). The debris resulting from the failure (approx 0.5m³) had the potential to lay against the wall and trap water. However, the trench for the Novaflo® was dug after the failure, and so the loess that was against the wall had been removed. Potential for similar sized failures was evident and would need to be cleared fully before emplacing the drainage and filter system if they were to occur.

Pole-wall:
The roadway retaining pole-wall showed some evidence of erosion at its eastern end. This area lined up with a paleochannel, visible in a cut face just down slope (where glass embedded in the soil indicates activity within European settlement times - approx. 150 years), and with an area where foundations had to be taken to depth (approx 1600mm below surface). An active erosion gully further up slope is also aligned with this area of erosion in the wall. Measures are being put in place up slope to intercept debris and water before they have a chance to affect the current project. The active erosion of the road wall suggests that the measures currently in place are either not sufficiently designed, or are not being maintained correctly and are in need of further study.
3.2.6. Suggestions for Adequate Wall Completion

Retaining wall 1:
Removal of loess clasts from the backfill and placement of a concrete bed for the Novaflo® are the only recommendations that can be made with the level of investigation that was carried out. It is unclear how the decision to used Bidim® 34 was reached, other than recommendation from a senior design engineer.

Retaining wall 2:
A modification to the slope geometry would be relatively simple using a small digger, and would reduce the potential for problems discussed earlier. If this is not feasible, then removal of minor slope failure material should be thorough, as this material is likely to have an even higher tendency toward erosion. Again, a concrete bed for the Novaflo® is suggested.

Pole-wall:
As mentioned above, further investigation of the erosion gully up slope and a reassessment of the interception measures is required. As this is a less critical wall, the need for waterproofing and excellent drainage is reduced, thereby making this seemingly inferior wall design acceptable.

3.3. Case study number 2
3.3.1. General Description
This case study looked at a retaining wall in loess that was in need of remediation. The wall in question was the hill-side wall of a new extension to the basement of an existing property. During the hour long investigation of the site, the wall was in the process of being re-excavated for the installation of a drainage and filtration system. The drainage system from the wall leads along the house through a series of differing soil conditions to a sump as shown in figure 3.3.
3.3.2. Reason for Current Works
The problem was based around an extension made to the basement (back into the slope and under the main house) about 7 years ago. The basement had started to become very damp and visible cracking of the skirting board wood is evident due to swelling. Also, the carpets had become significantly dampened and were removed for drying during the investigation. The problem was concentrated towards the eastern end of the wall in question.

3.3.3. Original Site Conditions
The wall consists of masonry blocks that had not been infilled with concrete or reinforced in any other way. Further into the basement there was a wood frame wall with some insulation in the form of pink batts. There had been little or no water proofing (some HDPE), and the blocks were originally very close to the loess face, with some loose backfill (thought to be loess). The combination of these factors suggests that this wall was largely ineffective for either strength or water proofing the basement extension. Figure 3.4 shows a representation of a cross section of the original wall system.
3.3.4. Current Works

Excavation of the entire basement wall has been completed, widening the gap between the loess face and the masonry wall to at least 0.4m. The resulting loss of structural support for the house following this additional excavation was remedied by the installation of a concrete pile at the eastern end of the wall. The wall was to be covered with Mulseal® (three coats), and then with HDPE to be taped to the wall. This water proofing layer is to be placed from the base of the excavation (against the floor slab) to a height of approximately 3 bricks (roughly 50cm).

Subsequent to the water proofing, a concrete support channel for the Novaflo® is to be built, with a 5cm drop from the west end to the east end (approximately 6m). At the far end, the top of this channel is intended to be 50mm below the top of the floor slab. A Bidim® geotextile will be used (grade not yet specified) to line the loess face to a similar height as the water proofing (but not to wrap around the Novaflo®). After backfilling to that level, the geotextile will be folded over on top of the tailings and held down with a final thin
layer of tailings. The remaining height of the wall (approx 1.5m above backfill) is not expected to pose any problems, and be self supporting, it will therefore not be backfilled. Figure 3.5 below shows a representation of the proposed completed wall.

![Diagram of planned system](image)

The drainage system will be continued along the eastern wall of the house, along which there are three distinctly different sections (see site plan, figure 3.3).

1) Backfill/loess bank
2) Loess/loess
3) House wall/soil (more garden soil than loess)

In each of the above cases Bidim® is to be used against the eastern bank. The Novaflo® pipe from the main retaining wall will continue along a trench running the length of these three sections, which will be backfilled with tailings. In the case of 2 (above), the loess which is under the house will be covered by waterproofing material (HDPE), so that no water can migrate under the house. It is not expected that there will be any water present under the house due to the collection system along the retaining wall described above.
under the house due to the collection system along the retaining wall described above. Section 3 of the trench will also have HDPE on the house side. The water will be pooled at the end of the eastern wall of the house in a sump where some provision will be made for the settling of suspended material. The outlet for this sump will be storm water pipe positioned at a level slightly below the inlet of the Novaflo®.

3.3.5. Observations and Interpretation
As excavations had just been completed, there was little evidence of any other problems with the original wall system, other than the moisture in the basement extension. The only moisture source visible was from a leaking downpipe up slope of the excavated wall along the eastern wall of the house. The property owner (who had done all the current and previous works) suggested that this may have been the source of the moisture that had passed into the basement extension. However, as this is uncertain, the full works are to be carried out as a precautionary measure to protect against other moisture sources not evident during excavation.

3.3.6. Suggestions for Adequate Wall Completion
The proposed works conform to what is currently considered “good practice” (see later discussion). This is provided that the drain is at a level below the floor slab to allow some siltation before the critical part of the wall is affected. It was also mentioned on site that the down pipe that had been providing moisture would need to be repaired. The only part of the design that was not established during this visit, was the grade of geotextile to be used on the various sections. Another aspect of this system is the reinforcement of the masonry wall. It is suggested that reinforcement would be a prudent measure, to provide some factor of safety during earthquake loading. Finally, continuing backfilling to the full height of the loess face will reduce the potential for later problems in the form of minor failures of the exposed loess (resulting in loess debris resting on the wall).
3.4. Case Study Number 3

3.4.1. General Description

This case study looks at a retaining wall set into loess in the Westmorland subdivision. The investigated wall formed one of the walls for the garage, as well as forming the boundary of the driveway, and clothes-line areas. During the brief investigation the wall was in the process of having the filtration and drainage system refitted due to the failure of the original system in the form of abundant silt and water passing through the wall. Figure 3.6 below shows a sketch of the site layout:

Figure 3.6 Site plan for case study3.

3.4.2. Reason for Current Works

Original wall geometry and construction was such that water and silt was passing through the wall at the southern end through vertical joints of the wall blocks. The distance from the wall (up to 1m) and the abundance of silt suggests either appreciable pressure, or flow rate. Novaflo® drainage piping had also been performing below expected levels due to clogging, resulting in part from damage during installation (according to a worker on the site).
3.4.3. Original Site Conditions

The 2m high wall consists of large concrete blocks as the main wall, with brick facing on the eastern side. Some water resistant Mulseal® had been painted on the entire height of the wall, but recently exposed sites show that this may not have been applied thickly enough. Along the garage section of the wall, additional shelter seal has been placed over the Mulseal® to add to the water retention properties. A layer of polystyrene was used over the length of the wall to protect the wall and its seal against the placement of the backfill. The backfill appeared to consist of tailings plus soil (a mixture of top-soil and loess) to approximately half the height of the wall. This was covered above and behind (i.e. against the soil slope) by an unidentified woven filter. Above the filter cloth was soil, with some tailings as a backfill. Drainage was from north to south via 110mm diameter Novaflo® piping. Figures 3.7 a & b show typical cross sections of the original system along two sections of the wall.
3.4.4. Current Works

The current scheme was to remove the old backfill and replace with pure tailings to the full height of the wall, and use of a heat bonded, nonwoven geotextile (grade unknown) from the base of the slope to the top, with no wrap under or around the Novaflo®. Novaflo® was to sit on freshly exposed loess (which implies a potential for undermining of the wall). Figure 3.8 shows a representation of the planned works in cross section.
3.4.5. Observations and Interpretation

Ponding of water was seen at the southern end of the wall (approximately along the same section that the silt had been passing through the wall earlier). This indicates that the drainage from north to south is insufficient as a direct result of an insufficient and consistent gradient. Pooling in this area may also be indicative of siltation in the Novaflo® (which had already been placed) down gradient, but this could not be confirmed. If this is the case, then this suggests that the levels will need to be reformed. The southern section of the main wall (the only part not yet backfilled) shows the base of the Novaflo®, approximately equal with the level of the floor slab on the other side of the wall. This suggests excavation was inadequate, since water may be pooled some way up the wall before adequate head will be attained for the Novaflo® to begin draining the site. This same level of head will result in some amount of flow through the vertical joints of the wall.

The Mulseal®, mentioned earlier, appears to be thin along this section, with none being visible along some of the joins in the blocks. Also, there are some vertical joins with
little or no fill material left. The combination of these two points and the level of the Novaflo® means that, should the wall be completed as is, there is a high likelihood that water and/or silt will come through the same cracks that let silt through in the past.

This section of the wall also shows evidence that the original backfill had some loess in it, since residual loess material can still be seen attached to the wall almost down to the level of the present Novaflo®. It seems somewhat fortunate that only these few problems occurred with the original wall since there seems to have been the potential for leakage up the entire height of the wall.

Also observed approximately 1.5m from the south end of the wall was a subsurface erosion cavity. The cavity was 0.5m in diameter at the exposed face, and extended back into the slope beyond visibility (>1m) at a diameter of approximately 20cm. Water was seen discharging from the cavity at a very slow trickle (estimated at around 100ml/hour). The cavity opening was 1-1.5m above the level of the Novaflo®. Significant erosion of the soil directly beneath the opening was visible. The presence of this cavity raises questions over the planned placement of the geotextile, as a number of experienced practitioners suggest placing a geotextile directly over an open tunnel gully or cavity is of little value. However, this point requires confirmation through long term testing and detailed investigation of a number of case histories.

Also of particular concern was an observation made by one of the workers at the site in relation to the Novaflo® drainage pipe. I was told that the piping had actually been crushed over a significant proportion of the length of the main wall. This would have had a profound impact on the performance of the system. As well as significantly reducing the available space for silt to accumulate within the piping, the space for water to flow in the pipe would have been dramatically reduced, thereby making this drainage measure ineffective. It is unsure exactly why the pipe was crushed. The two most likely contributing factors are: reduced strength of piping due to prolonged UV exposure; and “rough” installation procedures (for example, dramatic loading on small areas as a result of a load of backfill being dumped directly onto one area). Other possibilities to consider are: defective manufacture of this individual sample; inadequate design; failure of testing to provide accurate strength characteristics of the piping material; degradation of the pipe in place resulting in sufficient strength loss for failure. It is impossible to establish which was (or
were) the causative factor(s) at this stage, however, the firm conclusion can be made that close attention must be paid to the survivability and durability of the piping.

The final concerning observation (also relayed by the on-site worker), was the proposed placement of a garden directly on top of the tailings backfill. Although such a feature would be aesthetically more appealing, it would have a number of potential impacts on the performance of the wall’s drainage system in particular. The soil that would be used in such a garden could easily migrate through the coarse backfill, and add to the siltation of the drain pipe. This process would be aided by watering of the garden, which would itself provide additional demand on the drainage system. Organic matter from either living plants, or compost could also migrate down and may reduce the performance of the geotextile filter by biological clogging. Extended root systems have been shown to be capable of penetrating geotextiles (eg Giroud et al, 1977), which could significantly alter the performance of the geotextile locally.

3.4.6. Suggestions for Adequate Wall Completion

Ideally, the entire wall should be re-excavated (this may include the east-west section) and the levels adequately checked to ensure that the Novaflo® is below the level of the foot wall. Also, a concrete channel should be made for the Novaflo® to rest on, to avoid the potential for undermining. As a less satisfactory option, the filter cloth should be made to reach the wall, that is the Novaflo® should be placed on top of the filter cloth. In the section where the silt had been allowed to pass through the wall, resealing should be carried out, either with Mulseal® or cement filler into the gaps, preferably both. Another option may be to use shelter seal - as used along the garage section.

If excavation is repeated, the Mulseal® should be inspected, and will most likely need a refresher coating. Replacement of the tailings should be careful, since no polystyrene is available, in order to protect the wall, and most importantly the water proofing.

The section where the tunnel gully is present (approximately 1.5m from the south end of the wall) has some potential for problems, and may justify redesigning to include some of the features illustrated on figure 3.9. It is also suggested that the remaining length of the wall would need inspection if re-excavated to identify any other erosion cavities.
A final recommendation is that the proposed garden is not put in place as suggested. If a garden must be placed, a “rock garden” containing shallow rooting plants that require little watering and minimal soil will place less stress on the wall’s drainage system. If the owners consider this unacceptable, then a filtration system below the garden would have to be designed.

![Figure 3.9. Cross section of suggested system for southern section.](image-url)
3.5. Case Study Number 4

3.5.1. General Description

This case study focuses on the practice of installing geotextiles. This example was one where large tanks (2x 50,000litre & 1x20,000 litre) were being installed in a pit excavated into Canterbury plains river gravels. Figure 3.10 below shows the approximate layout of the site.

Figure 3.10 approximate layout of the site.

3.5.2. Reason for Current Works

This was a new site requiring storage of large volumes of liquid, hence the 130,000 litre total capacity.

3.5.3. Original Site Conditions

The pit measured approximately 20m long x 10m wide x 5m deep, and was excavated out of relatively dry Canterbury Plains river gravel. Observations of the flow patterns in the gravels suggested flow would be across the width of the pit if the water table were higher; these observations being consistent with a flow direction toward the coast. Local knowledge suggests it is likely that the water table could reach at least part of the pit during the wet winter months.
3.5.4. Current Works
Once the tanks were installed, the pit was to be backfilled with 14mm greywacke chip. Both the floor and the walls of the pit were to be lined with a heat bonded nonwoven geotextile for the primary purpose of separation, but also for filtration in the event of a rise in the water table.

3.5.5. Observations and Interpretation
Geotextile was placed by way of lowering from the top of the pit to the bottom directly from the roll, and held in place at the bottom by a small amount of the backfill material. Once a single length had been secured, a small amount extra was allowed at the top for fastening; the length was then cut, and the roll moved along. The geotextile lengths were initially overlapped by approximately 30cm (no sewing was done). Various methods were attempted for fastening each length at the top of the pit, including nailing, staking, and weighting down with stones, bricks or being held by people. These methods were inadequate in a number of ways, for example, the nailing or staking methods require a large strain to be placed on the geotextile before any extra length needed is made available, and since this point represents the failure of the nail or stake, the entire remaining length may slip. Other problems with fastening are mentioned later.

A number of problems also occurred during the backfilling process. As the weight of the backfill was placed, the geotextile tended to be pulled down into the pit. This was predominantly due to the geotextile matching the profile of the slope, as opposed to simply hanging down directly. However, as a result of the pulling, the fastening came loose in many instances, resulting in some folds in the geotextile being buried by the backfill falling from the digger bucket, thereby reducing the effectiveness of the geotextile as a separator and filter. An additional consequence of the fastening coming loose was the slipping of the geotextile lengths relative to each other. This had the effect in some cases of significantly reducing the original overlap or even creating gaps between lengths of geotextile. Clearly this is undesirable.

Another problem during the installation was the tearing of the geotextile. This occurred in two ways: firstly as a result of stakes or nails used to fasten the geotextile at the top of the pit tearing as the geotextile was pulled into the pit by the backfill loading, and secondly as a result of the digger bucket scraping (and tearing) the geotextile whilst placing a
load of backfill. The desired properties of the geotextile are severely affected by tearing, with filtration being lost in the damaged areas, and the strength required for adequate separation being diminished in a significant amount of surrounding area.

Figure 3.11. Cross section of features noted at case study 4

3.5.6. Suggestions for Better Practice
A number of the problems seen in this example were due to a lack of resources, including time, manpower and correct equipment. If more time had been available, then less short-cuts would have been taken, this is in particular reference to the tearing of the geotextile by the
have prevented the separation of lengths. A number of the problems would also have been reduced or avoided if the geotextile had been sewn together at the overlaps of each length. This would have eliminated separation of the lengths, and possibly made fastening more efficient.

The final major aspect that needs to be addressed, is the method of fastening. This is not a simply fixed problem, since the fastening needs to be firm enough to prevent folding in the geotextile (as a result of slippage), gentle enough to prevent tearing and flexible enough to allow some movement of the geotextile when conforming to the slope profile. The ideal machine for such a job is a human, as people are able to supply variable tension as needed. In this case, by the time people were employed for fastening, the majority of the damage had been done. Also, owing to the small amount of spare geotextile at the top of the pit, the people (including the author) were placed very precariously on the edge of the pit. Unfortunately, manpower was another of the resources that was lacking during this installation process as there were only three people to secure approximately 15 lengths. In this case, other options needed to be employed, for example pinned elastic (strong) attached to the geotextile to give a gradual lowering (see figure 3.12 for schematic representation), or something similar. For each of these possibilities (but in particular if people were used), there would need to be more spare geotextile left at the top of the pit to aid in the fastening. This excess material could easily be wrapped over the top of the pit once filled adequately.

Figure 3.12 Geotextile fastening device.
3.6. Retaining Wall Design Practice

This section focuses on the filtration/drainage aspects of retaining wall design. Specific discussion of geotextile selection or installation, will be covered in section 3.7.

3.6.1. Commentary from Case Studies and General Discussion

The above case studies have typically shown a site under construction or remediation at the time of investigation. Unfortunately this does not allow in-depth commentary on design methodologies for the filtration/drainage system in retaining walls on the Port Hills. However, discussion with a number of workers, some practicing engineers, and material suppliers tends to suggest that the main approach to design is that of design by experience, and unfortunately, design by chance is still being used in some cases.

All of the above retaining wall case studies outline problems of past, and in some cases, present retaining wall designs. These problems are summarised in table 3.1 below (note that “case No.” refers to before and/or after “current” phase of construction at time of investigation).

Table 3.1. Summary of problems observed with retaining wall design

<table>
<thead>
<tr>
<th>PROBLEM</th>
<th>POTENTIAL EFFECTS</th>
<th>CASE No:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Backfill</td>
<td>Siltation of drain, water retention</td>
<td>2,3</td>
</tr>
<tr>
<td>Poor drain levels</td>
<td>Water/silt ponding against wall</td>
<td>2,3</td>
</tr>
<tr>
<td>Crushed Novaflo® or no drain pipe</td>
<td>Water/silt ponding against wall</td>
<td>2,3</td>
</tr>
<tr>
<td>No drain channel</td>
<td>Erosion leading to undermining</td>
<td>1,3</td>
</tr>
<tr>
<td>Poor water proofing</td>
<td>Seepage through wall</td>
<td>2,3</td>
</tr>
<tr>
<td>Garden over backfill</td>
<td>Increased silt and water in system, potential for biological damage</td>
<td>3 (possibility)</td>
</tr>
<tr>
<td>Poor brick work</td>
<td>Easier flow through wall</td>
<td>3 (at least)</td>
</tr>
<tr>
<td>Active erosion</td>
<td>Increased silt, up-slope erosion</td>
<td>1,3</td>
</tr>
</tbody>
</table>

It is clear that a number of the problems identified have the potential for increased siltation of the drain, and retention/ponding of water against the retaining wall. The problem of siltation in the drain pipe is a significant one, as it tends to accentuate ponding of water that may already be in effect. If the siltation is localised (eg at the middle of a wall), this may well have the effect of creating a “mini dam” within the drain, blocking water up gradient
from draining past this point. This increases the area of the wall exposed to water over an extended time period. If water is ponded against the wall, seepage may become a problem even if water proofing measures are in place, and may be rapid if they are not in place or are inadequate. As seen in case study 3 (and emphasised by a number of workers and engineers), the vertical joins of the brickwork are the most susceptible to seepage. In case study 2 it was seen that water damage can occur through a number of wall layers, without necessarily having to flow through block joins. It should also be noted that in severe cases of impeded drainage, significant water pressure may build up, so as to become a structural concern to the wall’s integrity.

The need for concrete drain channels for the drain pipe to rest in has been discussed earlier, but should be reinforced. Observations of stage 2 tests (discussed later) tend to support the notion that water can and does readily flow under the drain pipe in the early stages of system development. If this flow was to occur directly over loess, continued erosion is virtually inevitable. This erosion would certainly lead to incorrect levels for the drain, increased sediment for drainage systems outside the wall, and may eventually lead to progressive undermining of the wall and perhaps structure behind the wall.

The issue of placing a garden directly on top of the backfill of a wall has been discussed in case study 3. It should be sufficient to say this is not considered advisable without careful consideration of the garden type, or the addition of a filtration system for the garden. Drainage capacity of the system underlying the garden (i.e., the retaining wall drainage system) may need to be increased depending on the expected level of watering for the garden.

Although surface drainage measures have only been discussed in case No. 1, such measures should be seen as an important aspect of filtration and drainage design. If surface water is intercepted and removed from the site prior to infiltrating the soil it will not place any demands on a down-slope retaining wall system. In addition, the occurrence of sheet and rill erosion on exposed areas during construction may create void spaces on the exposed loess face. Such void spaces make good contact between loess and geotextiles difficult to achieve during installation. Also, during early phases of operation, if sheet and/or rill erosion occurs directly up slope of a retaining wall, there is a high possibility of clay accumulation on the geotextile which will significantly inhibit it’s performance.
improving are outlined in the form of observed problems in table 3.1. Design approaches other than those covered by the case studies are in use, particularly for specialised situations. Some of these approaches, along with a summary of what has been identified as “good practice” are outlined in section 3.6.2 below.

3.6.2. Other Typical Designs Currently in use
Discussion with design engineers (particularly Marton Sinclair of Eliot Sinclair and Partners Ltd, Christchurch), and other very brief and informal site visits have shown a small number of other typical designs used on the Port Hills. It should be noted that there are very likely a large number of other approaches that have not been discussed, simply because the time was not available to visit more sites.

An example of an alternative design is presented simply in figure 3.13

Figure 3.13 Pole-wall design [Notes: 1) commonly used along roadsides and so therefore not considered critical, hence lower level of protection. 2) River gravel backfill allows for compaction and therefore can be load bearing]

Points to note:
- Drainage channeled along wall length to municipal drainage.
- River gravel backfill may be compacted to be load bearing.
- Effectiveness of river gravel as a filter to be established

3.6.3. “Good Design Practice”
Clearly good practice should emphasise the elimination of the problems identified to date. The following figure (4.14) outlines and labels the points considered to be key considerations for “good design practice”. The reader is also directed to figures 3.9 and 3.13 directly above as examples of “good practice” in different situations. It should be re-emphasised at this point that what is currently thought to be “good design practice” is yet to be tested by laboratory methods, or long term, intensive monitoring of field sites, and should not be considered the final word on good design practice as this should be an area of ongoing investigation and development.

Other points to note:
- Backfill must be free draining and provide uniform pressure on geotextile.
- At least one coat of emulsion seal to the full wall height (3 coats for bottom 1m).
- Vertical joins in wall must be well sealed, as these are common seepage points.
- Drain channel under novaflo reduces risk of erosion of excavated loess.

Figure 3.14. Cross section including elements of "good design".
3.7. Geotextile Design and Installation Practice

3.7.1. Commentary from Case Studies

In case studies 1 and 2, the engineers on site expressed a lack of knowledge as to why a particular geotextile had been specified. This suggests that there are few commonly known guidelines for the selection of geotextiles. No comment can be made as to the specific selection process for case studies 3 or 4 as an engineer was not present on site during the investigation. This in itself should be seen as a problem, as installation was also taking place at this time. The presence of an engineer is not necessarily a necessity, provided plans are detailed, and workers on site have sufficient skill levels to provide satisfactory installation.

The case studies outlined a number of installation concerns, along with uncertain design methodologies that are summarised in table 3.2 below.

Table 3.2 Identified problems with geotextile design and installation practice.

<table>
<thead>
<tr>
<th>PROBLEM</th>
<th>POTENTIAL EFFECTS</th>
<th>CASE No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overhanging slope</td>
<td>Gaps between loess/geotextile,</td>
<td>1,4</td>
</tr>
<tr>
<td></td>
<td>strained geotextile</td>
<td></td>
</tr>
<tr>
<td>Poor fastening</td>
<td>Tearing, wrinkles, strain</td>
<td>1,4</td>
</tr>
<tr>
<td>No sewn overlap</td>
<td>Gaps, difficult handling</td>
<td>1,3,4</td>
</tr>
<tr>
<td>Rough installation</td>
<td>Tearing, wrinkles, strain, gaps</td>
<td>4</td>
</tr>
<tr>
<td>Unclear design criteria</td>
<td>Inappropriate geotextile installed</td>
<td>1,2,3,4</td>
</tr>
<tr>
<td></td>
<td>(possible clogging or piping)</td>
<td></td>
</tr>
<tr>
<td>Design by cost</td>
<td>&quot;</td>
<td>Uncertain</td>
</tr>
<tr>
<td>Design by chance</td>
<td>&quot;</td>
<td>Uncertain</td>
</tr>
</tbody>
</table>

Although a number of the above problems seem to be common (unfortunately), most are easily remedied. For example, elimination of overhangs and other slope geometry problems can be remedied by emphasising to the excavation contractor the need for a uniform and as smooth as possible slope. The problem of fastening the geotextile does need to be looked at in more detail, and perhaps trials conducted on a range of fastening options. As for sewing geotextile overlaps, it is thought that this is not done because of the addition to time and cost. Machines designed for sewing geotextile overlaps in the field are available, but even these require the time to do the job, which is often considered not available. As a number of jobs requiring geotextiles in Canterbury are relatively small, it may perhaps be a worth-while
exercise to investigate the viability of “pre-sewn” geotextile rolls that can be rolled out (down-slope) and then unfolded (along slope).

The problem of rough installation is one that should not be overlooked purely as the fault of the contractor. If the importance of each aspect of installation is not explained to the contractors by the design engineer, they may be unaware that they are causing a problem. Even when this is explained, it can be forgotten (or in some cases simply ignored), and so it is important that the design engineer be present during installation to ensure good installation practice is followed.

The problem of design methodology has been discussed at length in chapter 2.

3.7.2. “Good design and installation practice”
As in section 3.6.3, good practice should be seen as the elimination of problems, and continuing scrutiny of system performance leading to even better design and installation. As mentioned in chapter 2, the ideal for design is a design by function approach. However, considering typical budget constraints of most retaining walls on the Port Hills, such an approach is unfeasible.

The common approach of design by experience is seen as an acceptable compromise, provided the experience base is large, investigations are thorough and experience has involved some design by function cases. This approach has been detailed in chapter 2.

As for installation, it appears that a lack of knowledge as to the function of the geotextile may be the cause of a number of errors seen in the field. If contractors are not explicitly shown what is needed, and explained why, poor installation will continue. The main points of good installation practice are:

1) Uniform close contact between geotextile and loess slope achieved by:
   a) uniform, smooth slope geometry (no overhangs)
   b) use of appropriate backfill
2) Adequate overlaps between lengths achieved by:
   a) sewing overlaps and
   b) careful placement of backfill
3) Approximately uniform stress on geotextile achieved by:
   a) effective fastening at the top of the slope
   b) careful placement of backfill
4) NO TEARING achieved by:
   a) good fastening at the top of the slope
   b) careful placement of backfill
   c) use of appropriate backfill
3.8. Synthesis

3.8.1. Discussion
It is clear that design practice with geotextiles is not firmly established even within some of the engineering community, and it is commonly the experience of longer established practitioners that is defining current usage. This was dramatically illustrated to me when a practising design engineer at a job site asked me what grade of needle punched geotextile I would recommend. Since this was toward the start of my project I declined recommending anything! Richardson and Wyant (1985) emphasise the importance of having a qualified engineer present during installation to ensure correct practice. They then go further to suggest that the term “qualified” should not be taken for granted, since according to them, “many geotechnical engineers are not familiar with geotextiles and their design or installation”. Although that was some 13 years ago, the relatively recent arrival of geosynthetics in New Zealand means that a great many geotechnical engineers in this country do have very limited experience with these materials.

3.8.2. Conclusions
The four case studies presented have indicated a number of concerning aspects of retaining wall design, and geotextile installation practice. The following is a brief summary of the concerns identified with regard to the installation of materials (particularly geotextiles). These concerns should be taken into consideration, and the problems eliminated as part of good practice.
Installation concerns:

- Poor Contact between geotextile and loess: This results from uneven shaping of the cut slope, the formation of rills prior to geotextile placement, and there are indications that common backfills used may be too coarse to provide good contact.

- Tearing of geotextile: This has been seen where installation has been rushed, with geotextiles being torn by digger buckets and fastening devices resulting in loss of filtration and separation function in those areas.

- Lack of overlap between geotextile lengths: Commonly as a result of poor fastening combined with rough placement of backfill material and inadequate initial overlaps. This problem may be eliminated by sewing of the overlaps.

- Crushed drain pipes: A crushed drain may be the result of degradation of the pipe prior to placement (suggesting materials should be placed when new). Or, crushing may be the result of rough placement of backfill, again suggesting the need for care during backfill placement.

Also, the following have been identified as aspects that should be considered as part of good design practice when designing a retaining wall on the Port Hills at this stage (pending laboratory testing, and long term monitoring of field performance).

- Backfill: must provide uniform pressure on geotextile and loess face, and must also be as free draining as possible. The elimination of use of backfills with abundant fines (ie soils) is recommended as these retain significant amounts of moisture, and potentially provide additional silt that may cause clogging of the drain pipe.

- Excavation levels: Excavation should be such that the base of the drain pipe is well below the join between the floor slab and the first blocks of the retaining wall to allow some degree of siltation to occur in the system before the critical joins are exposed to water or sediment. Also, drain levels should be of a uniform gradient to eliminate the possibility of water ponding, and promote drainage.

- Drain channel: The inclusion of a concrete channel for the drain pipe to rest on is seen as a prudent measure to avoid the possibility of erosion of the loess that may occur when the drain pipe is placed directly on to excavated loess.
> Water proofing: thorough water proofing is a vital aspect of any retaining wall where water seepage would be unacceptable (i.e., most house walls).

> Design and filter selection: the specific design and selection of filter should be arrived at by means of a design by function approach, or a design by experience approach (with the provisions presented in chapter 2).

The above should not be considered the final word on good design practice as this should be an area of ongoing investigation and development, and is drawn from a limited case history.
Chapter 4
The Stage Fest
Chapter 4  The Stage 1 Test

4.1. Introduction to Laboratory Testing Programme

4.1.1. Test methods used
As mentioned in chapter 1, three main tests have been used to investigate the performance of a range of materials for the purpose of filtering Port Hills loess behind retaining walls. These tests have been named as follows (relevant chapter given in brackets):
Stage 1 test (Ch.4)    Stage 2 test (Ch.5)    ASTM D 5101-90 Standard test (Ch.6)
A discussion of the methodology, analysis, and results from each test is given in the indicated chapters.

The stage 1 and standard tests both operate on similar principles, with an amount of recompacted loess being held by a filter in direct contact with the loess. In both tests water passes vertically down through the system. Due to a number of technical, financial and logistical concerns a smaller number of filter options have been tested using the standard test than with the stage 1 test. Although both tests do not directly simulate a retaining wall situation, they do provide detailed information on filtration performance with loess and this, in turn, provides a theoretical basis for practical design.

The stage 2 test is a developmental test that simulates retaining wall situations. This test provides a large amount of information on the entire filtration/drainage system of a retaining wall. This may prove very valuable in identifying problems thought to be due to weaknesses in filtration design, but may actually be attributable to other aspects of the system. Unfortunately, only three situations have been tested with this method, but the potential exists for continuing study with this method in subsequent projects.

4.1.2. Filter options used
4.1.2.1. Geotextiles
Selection of geotextiles for testing was based on current usage on the Port Hills. However, some mention should be given as to why no knitted or woven geotextiles were selected. The main reason for not using knitted geotextiles was their tendency for excessive elongation (as mentioned in chapter 2). This property has the additional effect of making pore space geometry difficult to predict under field conditions and subject to localised variations making the overall performance of knitted geotextiles erratic and difficult to predict. Rod
McKenna (formerly technical manager of Geosynthetic Testing Services, in Australia) suggested that a monofilament woven geotextile be tested in this study. However, through discussion with local engineers and observations in the field, I have not yet seen a woven geotextile being used successfully on the Port Hills for filtration. In addition, Giroud (1994) states:

"soil particles may obstruct geotextile filter openings...in the case of filters with individual openings such as woven geotextiles. This mechanism is sometimes referred to as “blocking” of the filter”.

The lack of use on the Port Hills, and this tendency for blocking, means that no wovens have been tested.

A list of the geotextiles that have been focussed on, along with their specifications, is given in section 2.3.5. Essentially, two of the Terram® range of thermally bonded nonwoven geotextiles, and 3 of the Bidim® range of needle punched nonwoven geotextiles form the core of materials to be tested. These geotextiles represent the most commonly used geotextiles for filtration on the Port Hills in retaining wall situations.

4.1.2.2. Granular filters
An important aspect of this thesis has been to find readily applicable solutions to practical field problems. Therefore the granular filters tested have been chosen in part for their ready availability. This is particularly the case for the filter “cyclone sand” which has been tested in place of an artificially created granular filter, designed with traditional criteria. Also tested for their ready availability and common usage are: 7mm premix and “cap-75 river gravel”. It is more common that river gravel used as a filter/backfill in the field has no maximum grain size imposed, however, due to the scale of these tests, particle sizes greater than 75mm (intermediate diameter) are considered impractical. Grain size distributions for these three filters can be found in appendix A3.4 (samples i, ii, and iii).

4.1.2.3. Soils tested
A bulk sample of Birdlings Flat loess was collected from the Ahuriri quarry for testing in both the stage 1 and ASTM standard test. It should be noted that, although this loess is of the same facies as that found in residential areas of the Port Hills, there are significant variations within this soil type. Additional large samples were taken from this same site for testing in the stage 2 tests. The advantage of this collection method was the consistency of
soil properties throughout testing that may not have been possible with sampling from construction sites on the Port Hills. Appendix A3.4. gives the grain size distributions for all tested samples, and shows minimal variation. The unfortunate disadvantage of this method was that samples collected were only slightly dispersive (crumb classification 2 in the modified crumb test, (Yetton, 1986) – see appendix 9).

Stage 1 Test

4.2. Introduction to stage 1 test
The stage 1 test is a developmental test that is designed to be as simple and cheap as possible. Once established, a test such as this will provide a quick and simple means of comparing the performance of filters with site-specific soils. The general concept of the test came from some informal work done by Mark Yetton (of Geotech Consulting, Christchurch), and discussion with Marton Sinclair (of Eliot Sinclair and Partners Ltd). From this point, the idea was further developed through a series of pilot tests and continuing discussions with a number of people.

4.2.1. Filters tested
Table 4.1 lists the filters investigated in the stage 1 test, and gives an indication of the level of current usage of each. Geotextile samples were provided by a local distributor, whilst granular filter samples were obtained from the Fulton Hogan quarry (Pound Road, Christchurch).
Table 4.1 Filters used in the stage 1 test.

<table>
<thead>
<tr>
<th>Filter name</th>
<th>Designation</th>
<th>Level of current usage</th>
</tr>
</thead>
<tbody>
<tr>
<td>A12</td>
<td>Bidim® geotextile</td>
<td>rare</td>
</tr>
<tr>
<td>A14</td>
<td>Bidim® geotextile</td>
<td>common</td>
</tr>
<tr>
<td>A24</td>
<td>Bidim® geotextile</td>
<td>very common</td>
</tr>
<tr>
<td>A34</td>
<td>Bidim® geotextile</td>
<td>common</td>
</tr>
<tr>
<td>A44</td>
<td>Bidim® geotextile</td>
<td>rare</td>
</tr>
<tr>
<td>A64</td>
<td>Bidim® geotextile</td>
<td>very rare</td>
</tr>
<tr>
<td>T1000</td>
<td>Terram® geotextile</td>
<td>very common</td>
</tr>
<tr>
<td>T1500</td>
<td>Terram® geotextile</td>
<td>common</td>
</tr>
<tr>
<td>Cap75</td>
<td>River gravel (granular(^4))</td>
<td>common(^2)</td>
</tr>
<tr>
<td>7mm Premix</td>
<td>Concrete aggregate (granular(^4))</td>
<td>common(^3)</td>
</tr>
<tr>
<td>Cyclone sand</td>
<td>Sand mix (granular(^3))</td>
<td>unknown (suggests very rare)</td>
</tr>
</tbody>
</table>

Notes:  
1) Current usage for retaining wall situations on the Port Hills  
2) Maximum intermediate diameter of 75mm used due to scale of testing; river gravels used in practise have no maximum grain size imposed.  
3) Current usage is typically as a secondary filter in conjunction with geotextiles in sites containing developing tunnel gullies.  
4) Grain size distributions shown in appendix A.3.4.

4.2.2. Basic description of the test

The test is simply recompacted loess at the bottom of a 1.2m vertical section of sewer pipe, held in place by a geotextile taped to the bottom of the pipe (schematic diagram shown in figure 4.1). Where granular filters have been trialled, a support system has been emplaced to hold the material in contact with the loess at the bottom of the sewer pipe (schematic diagram shown in figure 4.2, section 4.4.1). The loess is subjected to a head varying between 1 and 0.8m (approx.) of tap water. The head is varied by way of allowing water to leave the system through the geotextile at the bottom of the tube, periodic refills simulate rapid increase in head that could be associated with rainstorms. All tests in this series are conducted using recompacted loess from the Ahuriri quarry to a thickness of 100mm.
Figure 4.1 Schematic diagram of Stage 1 test with geotextile filter.

The 1m head is seen as the largest practicable in the lab, however, heads of up to 5m can be experienced in some exceptional circumstances (Yetton, pers. comm., 1997). More typically, however, geotextiles in retaining walls will not have to deal with heads in excess of 0.5m in most situations with loess on the Port Hills (Sinclair, pers. comm., 1997). The head of 1m is therefore considered to be on the high side of normal and represent a difficult case.

4.2.3. Objectives

The primary objectives of this test are:

➢ To obtain a quantitative comparison between various filters by way of measuring permeability versus time and fines passed over time.
➢ To develop a cheap test method for comparison of filter performance with site specific soils.
➢ To investigate the nature of the soil/geotextile interface.
The parameters of permeability over time and quantity of soil passed over time are seen to be the most relevant for retaining wall applications. Before this test may be used for testing site specific soils, the test method must be established as an acceptable method for comparing filter performance with a standard soil type (such as is used throughout this testing programme). Also, results of the stage 1 test must be compared to results obtained from the ASTM standard test, to provide the necessary basis for evaluation of the stage 1 test method.

Investigation of the soil/geotextile interface properties has been done by way of observation under a scanning electron microscope (SEM). As the performance of a soil/geotextile system is controlled by the structures that form between these two materials, observations of this critical interface may indicate potential causes of phenomena observed over the test duration. The reader is reminded of sections 2.5.4 and 2.5.5. for terminology relating to filtration performance (eg clogging, blocking, bridge network, and vault network).

4.3. Discussion of test Methodology

The methodology of this test is outlined in detail in appendix A2. This section will outline the advantages and disadvantages of various aspects of this testing procedure.

Placing the soil into the test apparatus wet (moisture content of approximately 16%) was effective in preventing the collapse of the soil structure that was observed in the standard test during saturation (see chapter 6). This amount of moisture also assisted in obtaining compaction levels similar to in situ densities (1675 kg/m³ ±3%). The primary intention of compaction was to reduce void space to an acceptable level. As noted by Rod McKenna (pers. comm, 1998), total elimination of space for air bubbles (which may influence test performance) is impracticable in most tests. Visual estimation of the dry soil sample after testing suggested less than 1% of the total soil volume was occupied by observable size air pockets.

Placement of the filter over the recompacted loess has been identified as an aspect of the test in need of improvement. The centre of an approximately 40x40cm geotextile sample is placed in contact with the loess, and the remaining area is folded down and taped to the tube. With the higher grades of needle punched geotextiles and the heat bonded geotextiles, small voids are created where the geotextile is folded to be taped down to the tube. The occurrence of void spaces between soils and filters is generally regarded as an undesirable situation, and has been indicated as a contributing factor for some clogging failures (Giroud,
1982). Also, if tension is too great during this folding process, excess strain may be placed on the geotextile, potentially changing its properties. Finally, when placed in the test position, the pliable nature of the geotextiles allows a small amount of movement in the downward direction as a result of loading from the water and soil above. A suggested remedy to this feature of the test is the addition of a soil support mesh (as used in the ASTM test, and stage 1 granular filter tests). However, small amounts of movement may potentially occur in the field, particularly on excavated faces where the sudden loss of support and the vibration of machinery may encourage some form of soil movement.

The potential exists for disruption of the soil if the water filling process is not followed rigorously. If the soil is disrupted as a result of too fast water flow during filling, segregation of the disturbed soil will result, and cause the formation of a low permeability layer at the top of the soil sample. The procedure outlined in appendix A.2.7 was effective in avoiding this problem. Another potential problem is the possibility of piping along the sewer pipe walls. Although there was no evidence of this occurring in any of the tests, the inclusion of piping barriers in subsequent tests is recommended.

The continuous collection of material passing through the system is seen as a distinct advantage of this test. It allows calculation of permeability at regular intervals. Also, as water passes directly from the test to the collection beaker, a good representation of the total amount of non-liquid material passing through the system over time may be obtained.

It is recognised that the test set up does not accurately represent field conditions behind a retaining wall. This is as a result of the method being based on a permeameter concept, for the purposes of comparability with the ASTM D5101 test, whilst trying to incorporate some aspects of field conditions (eg density and variable head associated with rainfall events). The method is also intended to be as cheap and simple as possible, so as to encourage testing with site specific soils on low budget projects, as is the intended end result, should the test prove to be a reliable indicator of filter performance.

4.4. Methods of Analysis

4.4.1. Permeability Over Time

There were two primary options for measuring permeability. The first was to accurately measure the amount of water collected in the 2 litre collection beaker (by weighing). The second option was to measure the change in water level, and calculate the difference in volume. Both methods required that the system be closed from external losses of moisture.
This was achieved cheaply by covering all open parts of the system (eg top of the sewer pipe and between bottom of sewer pipe and collection beaker) with clear cling wrap.

Although both beaker weight and water level measurements were taken, results are presented from the change in water level. The main reasoning for this decision was as a result of the test set up of the granular filters tested, shown in figure 4.2 below.

Figure 4.2 test set-up of granular filter with stage 1 test.

As can be seen in the diagram above, there is potential for significant quantities of water to pass through the filtration system, and not be measured by collection, particularly in the early stages of the test when water will accumulate on and around the tailings. However, measuring the change in water level would account for all water passing through the system.

Results of permeability versus time are presented in section 4.5.
4.4.2. Mass Passed Over Time and Grain Size Analysis

A number of options were considered to investigate the distribution of sizes of material passing through the system. The main obstacle for this was the very small amount of material involved (typically 0.1-0.4g per week). Conventional grain size analysis methods such as pipette or hydrometer analyses would not provide results with such small samples (Associate Professor Doug Lewis, pers. comm, 1998). Alternatives considered included various forms of microscope analysis, automatic settling tubes, and light transmission, all of which were found to be unworkable for various reasons.

The concept of a grain size analysis using various grades of filter paper in similar fashion to sieves was developed from a suggestion by Cathy Knight (engineering geology technician at the Department of Geological Sciences) and some other testing work. The procedure is outlined in appendix 4, along with sample results. At this stage this method can only be considered in the early development stages, but has shown promise in analysing some samples from the stage 1 test.

For the majority of samples that have not been analysed by the filter paper method, only total weight of material passed has been recorded. This has been achieved by pouring the material collected in the 2 litre beakers into a preweighed beaker and rinsing the 2L beakers with distilled water into the preweighed beaker also. The water is then evaporated off in a 50°C oven, and reweighed, the difference being the total weight of non-liquid material passed through the system over the collection period.

An additional aspect of investigating the nature of the material passing through the system, other than weight, is observation. As the results section will outline, a significant portion of the material passing through the system was dissolved calcite. Observation is therefore necessary to estimate the proportion of sediment to crystalline material after drying.

Results and discussion of mass passed over time analyses are presented in section 4.6.

4.4.3. Investigation of soil/geotextile interface

A number of options were trialled to investigate the structure of the soil at this critical area of the system. Every option requires that the test be completed and drained of the majority of the water. A first attempt at investigating soil properties and structure was to remove the geotextile, visually inspect the soil’s large scale structures and take soil samples (mostly saturated at this stage), as shown in figure 4.3.
Grain size analysis of each of these samples revealed no significant variation between samples. This suggested that either no structure had formed throughout the soil, or that the structure present was too subtle to be detected on this rather gross scale. Therefore, this method was not repeated.

Several attempts were made at injecting various bonding agents into the structure, in order to preserve the soil/geotextile system intact in order to obtain thin sections. Unfortunately, the very fine grain size of the loess required the resin to be heated significantly to increase its viscosity to a satisfactory level in order to penetrate the small pore spaces. This level of heating was significant enough to potentially cause damage to clay particles, and possibly the fibres of the geotextile (Rob Spiers, thin section technician, pers. comm, 1998). Also, as the resin moved into the void spaces, there was a tendency for alteration of structure. Similar methods have been used with success in the past (eg Mlynarek et al, 1990), however, attempts to contact members of staff at the Ecole Polytechnique (Montreal, Canada) where this method was used, regarding this and other matters have been unsuccessful. Discussions with Rob Spiers indicated such work would require a significant number of trials and experience, along with specialised bonding agents before adequate results could be obtained. Therefore, this method has not been used, but it may be useful to invest some time in development for future studies.
Subsequent investigation of the soil/geotextile interface was by way of direct observation of samples under the SEM. Raw samples were mounted onto SEM stubs, thoroughly dried, and then coated with gold (coating is thin enough so as not to influence observations). This method provided excellent visual images of structures present in the samples. The procedure used for extracting these samples is detailed in appendix A.2.10. It is freely admitted that this process is less than ideal, particularly for investigation of the geotextile, with the cutting process very likely to disturb grains held on the surface, or within the structure. However, this technique was considered the most efficient with the resources available. It is suggested that a subsequent study may be able to focus more on this aspect, and investigate better methods of sample extraction and preparation.

Results of investigations of the soil/geotextile interface are presented in section 4.7.

4.5. Results: Permeability Over Time

4.5.1. Introduction

Graphs of permeability (in m/s) versus time (in days) for each individual filter are shown at a scale most appropriate for that filter (these plots will also include variation in head over time). Also plotted on all of these graphs is a reference base line at $1.41 \times 10^{-8}$ m/s, which is the permeability of a recompacted sample of loess from Ahuriri quarry with similar characteristics as the samples used in stage 1 and standard tests (obtained by Jowett, 1995) and should be regarded as an approximation of base soil permeability. For the purposes of comparison, a classification of performance is suggested in table 4.2 below. It should be noted that this classification has been set arbitrarily to aid comparisons between filters, and should in no way be applied for design, rating or promotional purposes.

Table 4.2 Classification of Permeability performance for the stage I test (for comparison only)

<table>
<thead>
<tr>
<th>Classification</th>
<th>Relation to base level permeability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>$\geq 10$ times greater</td>
</tr>
<tr>
<td>Good</td>
<td>4 to $10$ times greater</td>
</tr>
<tr>
<td>Fair</td>
<td>2 to $4$ times greater</td>
</tr>
<tr>
<td>Poor</td>
<td>1 to $2$ times greater</td>
</tr>
</tbody>
</table>

The above classification suggests that filters with the higher permeability have a higher degree of safety against clogging, that is, they can sustain some reduction in permeability due to clogging, before their permeability performance reaches levels of concern. Hence, the higher the permeability, the "better" the rating according to this classification. All classifications of permeability performance given in the following sections are based on this classification.
4.5.2. Bidim® A12 [refer to figure 4.4, page 95]
The range of permeability for A12 shown in figure 4.4 of 8.24E-08 to 1.31E-07 compares favourably to the base level permeability, with the maximum being close to an order of magnitude greater than the base level. The relatively consistent nature of the permeability suggests equilibrium is obtained very rapidly. A good correlation between water level and permeability is evident, with permeability being increased subsequent to water fills consistently. There is a suggestion of a slightly increasing permeability trend from approximately 25 days on. No conclusion can be drawn as to the mechanism behind this trend at this stage. One possible explanation for this trend may be increasing efficiency of the soil/geotextile interface system. A second possibility is the development of flow paths through the soil as a result of cyclic increases in water level. However, considering the system permeability is already close to an order of magnitude greater than the base permeability of the loess, this is unlikely to be the primary control. Overall the performance of Bidim® A12 with respect to permeability can be classified as “good” to “excellent”.

4.5.3. Bidim® A14 [refer to figure 4.5, page 95]
The range of permeability is 3.47E-07m/s to 8.27E-08m/s, with a relatively consistent permeability around 1.50E-07m/s from 50 days onward. This is an order of magnitude greater than the base level permeability of a similar sample of loess. The relatively sharp peak noticed at 42 days may be as a result of disturbance of the system due to bumping (as unfortunately occurs in a communal laboratory). However, it is interesting to note that permeability has almost doubled after this sharp peak at 42 days with a series of consistent readings around 7.5E-08m/s prior. Perhaps this is indicative of an initial filter structure breaking down in favour of a more efficient one. Unfortunately this could only be confirmed through detailed analysis of the soil/geotextile system before and after this time.

No direct correlation between water level and permeability is evident in this data set. This may be largely due to the high frequency of filling needed due to the high permeability. The performance of Bidim® A14 with respect to permeability can be classified as “excellent” at the final equilibrium value of 1.50E-07m/s.
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Figure 4.4
Permeability and Water Level vs Time for Bore A12 in the Stage 1 Test

Figure 4.5
Permeability and Water Level vs Time for Bore A14 in the Stage 1 Test
4.5.4. Bidim® A24 [refer to figure 4.6. page 97]
Results of permeability versus time are plotted on figure 4.6. The range of permeability for A24 is $2.78 \times 10^{-7}$ m/s to $5.12 \times 10^{-8}$ m/s, this minimum value still being approximately four times greater than the base level. An initial period of high permeability suggests instability during the formation of the soil/geotextile filter system. Equilibrium appears to have been attained after 14 days. There is a very weak increasing trend from approximately day 35 onward. Suggestions as to the possible mechanism for this weak trend are as for A12. Also weak is the correlation between increased water level and increased permeability. Overall performance of Bidim® A24 with respect to permeability can be classified as “good”.

4.5.5. Bidim® A34 [refer to figure 4.7. page 97]
Permeability is in the range $2.2 \times 10^{-7}$ m/s to $4.91 \times 10^{-8}$ m/s, and is still well above the base level (see figure 4.7). A relatively stable equilibrium permeability around $5.7 \times 10^{-8}$ m/s has been established after approximately 25 days. There is little direct correlation evident between water level and permeability. The performance of Bidim® A34 with respect to permeability at equilibrium is classified as “good”.

4.5.6. Bidim® 44 [refer to figure 4.8. page 98]
Rather than the general trend of an initial period of high permeability followed by a reduction to an equilibrium value seen in A14, A24 and A34, this test showed an initial period of moderate permeability (approximately $8 \times 10^{-7}$ m/s), followed by an increase to very high, and quite variable permeabilities up to a maximum of $2.6 \times 10^{-6}$ m/s. A suggestion that this may be caused by piping along the side of the test pipe wall is not supported by observations of material passing through which was predominantly sediment free (with the exception of samples at 12 days and 17 days). If piping were occurring, it is expected that abundant sediment would be passing continuously.

As this is one of the samples that was subject to the problem of small voids being created during test set up, it is proposed that the observed behaviour is associated with movement of the soil into these void spaces. It may be that movement mobilises the sediment, allowing fine particles to pass through, and encouraging the remaining sediment to create a more efficient bridging filter structure, resulting in the observed high permeabilities. The initial movement seems to have been triggered by the first water refill at 12 days. Subsequent to this, correlation between head and permeability is difficult to establish. The performance of A44 with respect to permeability is classified as “excellent”.

Figure 4.6

Permeability and Water Level vs Time for Sidm A24 in the Stage 1 Test.

Figure 4.7

Permeability and Water Level vs Time for Sidm A34 in the Stage 1 Test.
Figure 4.8

Permeability and water level vs time for Bidim A44

Figure 4.9

Permeability and Water Level vs Time for Bidim A44 in the Stage 1 Test

Other sample information:

- Other sample information
- Other sample information

Figure 4.9
4.5.7. Bidim® 64 [refer to figure 4.9, page 98]
The range of permeability, seen in figure 4.9, is 5.37E-07 m/s to 1.72E-07. This range is unexpectedly high given that equilibrium permeabilities from A14 to A24 to A34 have been decreasing. This, combined with the overall shape of the graph, suggests that perhaps an equilibrium has not yet been obtained even after 40 days. This may be a result of similar processes as suggested for Bidim® A44, with soil movement into void spaces disrupting system performance. However, with the permeability being so consistently high, this is not seen as a bad situation with respect to permeability at least. A strong correlation of permeability to water level can be from 39 days onward, but is less evident earlier. It is unclear if the reduction in permeability from this time is exclusively as a result of changing water level, or if it is an indication that the system is beginning to head toward equilibrium as movement of soil within the test ceases. The performance of Bidim® A64 according to this data is classified as “excellent”. The lack of a clear equilibrium reinforces the need for longer term testing with this test method in the future. This need is emphasised by research done by Lawson (1990) that found a tendency for the system permeability of very thick needle punched geotextiles (4mm thick) with West German loess, to decrease dramatically after approximately 400 days. This is in spite of the fact that apparently stable conditions appeared to have been obtained after approximately 100 days.

4.5.8. Terram® 1000 [refer to figure 4.10, page 100]
The range of permeability for Terram® 1000 is 1.6E-07 m/s to 2.2E-08 m/s, with an equilibrium level of about 2.5E-08 m/s being obtained by about 14 days as seen in figure 4.10. Due to the low system permeability, water levels have not needed to be filled often enough to establish any correlation between water level and permeability. The overall performance of Terram® 1000 with respect to permeability is classified as “fair” to “poor”. This is of significant concern given the current high level of usage of this filter on the Port Hills. Long term characteristics have not been investigated, and so the conservative approach of recommending products with higher permeabilities must be used at this stage.

4.5.9. Terram® 1500 [refer to figure 4.11, page 100]
The permeability range of Terram® 1500 is 2.9E-07 m/s to 3.9E-08 m/s. Equilibrium seems to be mostly established after 21 days, and averages about 4.8E-08 m/s. This classifies the performance of Terram® 1500 with respect to permeability, as “fair”. There appears to be poor correlation between water level and permeability.
4.5.10. Cap-75 River Gravel
No data has been obtained for this granular filter from the stage 1 test. The reason for this is that the two attempts made to set this test up, both resulted in what could be classified as a catastrophic failure. In the first effort, the river gravel was not compacted at all (as is commonly the case in the field). During the water filling process, a rattling noise was noted, later identified as sand and gravel particles moving within the tube. Shortly after, water was noticed draining into the collection beaker at a rate far greater than normal, and with abundant sediment. It was concluded that movement of the sand and gravel within the tube had disturbed the loess sufficiently for the water already filled to flow with relative ease through the system, eroding the loess in the process. In the second attempt, the gravel was lightly compacted, but the same outcome resulted on filling. These failures strongly indicate river gravel's unsuitability for filtering loess, particularly when not adequately compacted.

4.5.11. 7mm Premix [refer to figure 4.12. page 102]
The range of permeability for 7mm premix in the stage 1 test is 1.0E-07m/s to 2.3E-08m/s, with an equilibrium value of approximately 2.4E-08m/s being mostly established after 40 days. There are a number of concerning points about this data set, especially the close proximity to the base level. Also of concern is the general shape of the curve which very well approximates a function approaching an asymptote, in this case represented by the base level permeability. This effectively means that the permeability of the system is continuing to approach that of the base level. Performance of the system at the time of test completion is classified as “poor”.

4.5.12. Cyclone Sand [refer to figure 4.13. page 102]
Figure 4.13 shows the permeability versus time plot for the cyclone sand granular filter. The permeability range is 1.13E-07m/s to a concerning 9.34E-09m/s, with an approximate equilibrium value of 1.0E-08m/s, established around 30 days from test commencement. The measurement of permeabilities below the base level is a clear indicator of filter clogging, and suggests the addition of a further subdivision in the permeability comparison classification given in section 4.5.1; that of “very poor” (less than base level permeability). As with Terram® 1000, correlation of water level with permeability was not possible, as refilling was not frequent enough.
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Figure 4.12

Permeability and Water Level vs Time for Anh Pyramid in the Stage 1 Test

Figure 4.13

Permeability and Water Level vs Time for Cyclone Sand in the Stage 1 Test

Other sample information:
- Dry density = 1791 kg/m³
- Initial moisture = 16.5%
4.6. Results: Mass Passed Over Time

4.6.1. Introduction
Plots of mass passed (g) versus time (days) are presented for each of the filters tested. As well as commentary on the parameter of mass passed, comparisons to the permeability results will be made where appropriate. This is done in order to better understand the interrelationship of these parameters and infer possible explanations for observed phenomena.

It should be noted that a minimum of 30% of all measured mass passed for geotextile tests is attributable to calcite passing into the collection beaker in suspension. Unfortunately, due to lack of time, it was not possible to better quantify the exact amount of calcite for individual samples, or to identify the process by which the calcite passes through the system. It is unclear whether this occurrence is a result of the test method. Further research is suggested to establish whether or not calcite may pass through a filtration system in a retaining wall situation on the Port Hills, and what effects this may have on the long term performance of the retaining wall and subsequent drainage systems. It is emphasised that the following graphs and discussions of mass passed include all material remaining after evaporation of the material collected from the stage 1 tests.

4.6.2. Bidim® A12 [refer to figure 4.14, page 105]
The initially high amount of mass passing through the system is inferred to be the result of fine material passing during the formation of the natural filter system between the soil and geotextile. The amount of material passing reduces to a low background level of under 0.1g per week as the filter becomes established.

4.6.3. Bidim® A14 [refer to figure 4.15, page 105]
A similar trend to that seen for A12 is noted. One significant point of interest is the small increase noticed at 42 days. This coincides with an increase in permeability at 42 days, and a general improvement in permeability subsequent to that 42 days. It was suggested in the discussion of permeability results for A14 that the increase at 42 days may be the result of a disturbance of the test apparatus. It was also suggested in the discussion of permeability results for A44 and A64, that the high permeabilities observed for those tests may possibly
be the result of soil mobilisation assisting in the formation of a bridging filter network. This suggestion is supported by the A14 data, by the increase of mass passed at 42 days (representing mobilisation of fine particles, leaving coarser particles behind to form a bridge filter network), coinciding with an overall increase of the A14 system permeability at, and after 42 days.

4.6.4. Bidim® A24 [refer to figure 4.16, page 105]  
Again, the general trend for A24 is similar to both A12 and A14. The rapid drop in mass passed suggests the soil/geotextile filter is mostly established within the first 5-10 days. As with A14, a small increase is noted, this time at 41 days. However, although A24 system permeability is seen to be on a generally increasing trend at around 41 days, the same dramatic increase in system permeability is not noted.

4.6.5. Bidim® A34 [refer to figure 4.17, page 105]  
The formation of the filter network for A34 appears to be well established, and very stable after 20 days. Both permeability and mass passed seem to maintain equilibrium after this time.

4.6.6. Bidim® A44 [refer to figure 4.18, page 105]  
As with the permeability results, A44 appears significantly different to the lower grades of Bidim® geotextile. A close correlation between increased mass passing and high permeability is apparent. This relationship is consistent with the possible explanation of periodic phases of soil movement to infill the voids between the soil and geotextile created as a result of the set up procedure. Although it is encouraging to see good permeability performance under such conditions, the amounts of material passing through the system is approximately 3-4 times greater than for the lower grades of Bidim®. Longer term testing would be very beneficial in assessing whether or not this increased level of mass passing is associated with a dynamic filter system (ie adapting to fit the changeable soil conditions induced by movement into void spaces), and if the mass passed will reduce once this system becomes stable.
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Fig 4.14

Mass Passed vs Elapsed Time for Bidim A12 Filter

Fig 4.15

Mass Passed vs Time Elapsed for Bidim A14 Filter

Fig 4.16

Mass Passed vs Elapsed Time for Bidim A24 Filter

Fig 4.17

Mass Passed vs Elapsed Time for Bidim A34 Filter

Fig 4.18

Mass Passed vs Elapsed Time for Bidim A44 Filter

Fig 4.19

Mass Passed vs Elapsed Time for Bidim A64 Filter
4.6.7. Bidim® A64 [refer to figure 4.19, page 105]

Although the values of mass passed are somewhat erratic, and are higher than for A12, A14, A24 and A34, observations of the appearance of the water suggest that from approximately 18 days onward, the vast majority of the material passing through the system ended up as crystalline calcite after evaporation of the water. This suggests that the volume of water passing through the system has a significant influence on the total mass measured. A possible means of accounting for this in future work may be to present the mass passed, in terms of grams per 100ml of water passed. However, as neither the permeability nor the mass passed has reached a clear equilibrium by the end of the test, discussion is as for Bidim® A44.

4.6.8. Terram® 1000 [refer to figure 4.20, page 108]

The amount of material passing through the T1000 system is lower than for the Bidim® range, both for the initial period, and the equilibrium period (after about 10 days). Although it appears as if there are significant variations after the 10 day period, a re-check of the scale suggests these variations are relatively minor in comparison to variations seen in the Bidim® range. The low permeability of the system, combined with the low amount of material passing through the system is consistent with a clogging or blocking system. As the system is blocked at the geotextile interface, relatively minor amounts of material (including water) are able to get through.

Interestingly, even though total mass passed is similar, or less than Bidim® equilibrium values, observations suggest the majority of material passing through the Bidim® systems is calcitic (estimated at 70-95% calcite), whilst the slight majority of material passing through the Terram® 1000 system is sediment (estimated 60-40% sediment). The process behind this is unclear and requires more detailed investigation.

4.6.9. Terram® 1500 [refer to figure 4.21, page 108]

The overall performance of T1500 appears to be more slightly favourable than T1000. This is unexpected, as the finer pore size of T1500 would, theoretically, tend to indicate a stronger tendency towards clogging. However, permeability has been found to be more favourable with T1500 than T1000, whilst mass passed performance is similar with regards to total mass. Observation of the material passing suggests that the percentage of sediment passed
by T1500 (approximately 20-40% sediment) is less than for T1000, but still greater than for the Bidim® range at equilibrium.

4.6.10. 7mm Premix [refer to figure 4.22, page 108]
It should be emphasised that the values of mass passed for 7mm premix can not be taken as an accurate representation of the total mass passed through the filtration system. This is as a result of the support mechanism used to hold the filter in place, specifically, the bucket of tailings that rest between the filter and the collection beaker. This space provides a number of spaces where material passed through the system may come to rest prior to being collected. However, a similar trend to the other filters is indicated, with an initially high amount of material passing while the filter structure develops, leading to an equilibrium period of low material passing. The increase in mass passed at 40 days does not correlate with any change in system permeability, and may simply represent an accumulation of past material (having been held amongst the tailings) being washed through to the collection beaker. As with T1000, the very low amount of mass passed at equilibrium, combined with low system permeability indicates some degree of system clogging.

4.6.11. Cyclone sand [refer to figure 4.23, page 108]
As with 7mm premix, mass passed values are significantly influenced by the tailings support bucket. However, an appreciable percentage of the mass recorded was identified as salts of some form (indicated by a taste test). It is unclear what the exact origin of this salt may be, however, the possible candidates can be narrowed down to either the sand or the tailings. Although the tailings were washed, evidence from the stage 2 test suggests salt may be associated with this material, and may not have been totally washed off. As the sand came from the same quarry, and was not washed, it should be considered a possible source also (see chapter 5 for discussion of stage 2 test findings). Evidence for clogging with this filter from the low permeability is supported by the very low equilibrium mass passed values (however, it is re-emphasised the mass passed values should not be considered as accurate representations for the granular filter tests).
4.6.12. General
Consistent trends have been seen for mass passed over time, with initial periods of high values indicating periods of filter structure development. A typical equilibrium value for mass passed of 0.1g for a 1 week period equates to 12g/m$^2$ for that week, and further expands to a figure of 624g/m$^2$ for a 1 year period. A significant portion of this value is provided by calcitic material that may well stay in suspension and be completely removed from the system. Without further study it is difficult to assess the total expected amount of material that would be passed by each of the tested systems. However, this is seen as an important avenue of study to assess the potential for the clogging of drainage piping. Figure 4.24.a shows the range of allowable weight of soil that may be passed through a filtration system into drainage piping of a given size (where the drain pipe has a specified “low point” that may have resulted from differential subsidence or poor installation practice) to prevent clogging. Figure 4.24.b shows the configuration of the “low point” in the drain pipe.

Figure 4.24.a Acceptable quantity of soil in a drainage pipe (from Austin et al, 1997)

![Figure 4.24.a](image)

Figure 4.24.b. Configuration of the lowest point in a drainage pipe (from Austin et al, 1997)

![Figure 4.24.b](image)
4.7. **Investigation of the Soil/Geotextile Interface**

4.7.1. **Introduction**

Due to time considerations, investigation of the soil/filter relationships has needed to be kept to a minimum. This is unfortunate, as a number of interesting observations have been made. As mentioned in section 4.4.3, initial attempts at investigating this relationship by grain size analysis and thin section investigations, were not successful. The majority of investigations of the soil/filter relationship were conducted using a SEM. The following sections outline the observations made. Figure 4.25 may be used as an indicator of the structure of loess away from the influence of the geotextile interface, however, it should be realised that natural variation does occur.

![Figure 4.25 Typical loess structure.](image)

4.7.2. **Soil Structures**

4.7.2.1. **Bidim® Range Observations.**

The suggestion made earlier in this chapter that the observed high permeability of Bidim® A44 and A64 may be due to the formation of efficient bridge networks has been supported by SEM observations. Figure 4.26 shows an image of the soil at the soil/geotextile interface (geotextile removed during sample preparation with as little as possible disruption to structures) of the sample filtered with Bidim® A44. This particular image was taken in the vicinity of one of the folds in the geotextile (approximate fold axis indicated by white dotted line). An abundance of relatively clean, large grains is seen, particularly around the fold, which is characteristic of a bridging network.
Brief investigations of the other geotextiles in the Bidim® range support the formation of bridging structures, particularly where there has been evidence of some movement or disruption of the soil (especially for A44, A64, and A14). However, bridging was not evident for Bidim® A34. Figure 4.27 shows an oblique cross section of the tested A34 sample. Above the indicated line is the surface that was in contact with the geotextile, whilst the material below the line represents a cross section leading up to the interface surface. A comparison of the structure of the material below and above the indicated line shows an abundance of fines at the interface surface. Further comparison of the scale of the image, and the $D_{95}$ of Bidim® A34 ($= 180\mu m$), suggests a bridging structure has not formed.

Considering the equilibrium permeability of this system was classified as “good”, the formation of a vault network is indicated.

Figure 4.27 Oblique cross section of A34 test sample. Surface above line is interface surface.
It is interesting to note that Bidim® A34 represents a middle ground between the highly pliable A12-A24, and the slightly stiff A44 and A64. It is speculated that the A34 may have formed a clear vault structure where the others have not, as a direct result of the soil being held firmly in place in the test. For A34, no void spaces (from folding) or sagging of the geotextile under loading from above, has prevented the mobilisation of fine particles that appears to have led to the formation of bridging structures in the other samples. There is some indication that some of the samples may, in fact have formed hybrid filtration structures with both vault and bridging structures acting in various places over a single sample. However, observations have not been in depth enough to confirm this.

4.7.2.2. Terram® range

Figure 4.28 shows a view of the soil interface surface of the Terram® 1500 test sample. The relatively large number of large grains visible suggests that bridging is present to some degree. However, there are areas that appear to have high amounts of clay particles. These areas of abundant fines differ in appearance to the vault network observed with Bidim® A34, as there appears to be fewer very fine silt particles, and the clays appear to be forming plate-like surfaces (as opposed to the more random orientation observed in figure 4.27). These observations, combined with the relatively low permeability of the Terram® range suggests some degree of blocking may be occurring. The plate-like areas of clay are therefore inferred to be accumulation of fines at blocked sites. It should be noted that a more in-depth investigation would be required to confirm this suggestion. Similar observations were made with Terram® 1000.

Figure 4.28 View of interface surface of T1500 test sample
4.7.3. Geotextile structures
For ease of comparison, figures 29 – 34 are shown on page 114

4.7.3.1. Bidim® range
Figure 4.29 shows a plan view of the interface surface of the A24 sample (after testing). Points to note are the lack of soil accumulated on the surface, and also the small number of very small bright patches within the geotextile. Figure 4.30 shows an enlargement one of these bright patches, indicating a localised clogging site. One of the striking points about this image, is that even though a clogging point is being shown, it is also clear that there are a number of alternative flow paths still available, which is a distinct advantage of the open structure of the needle punched geotextile range. Similar observations were made for all of the Bidim® range. More detailed work in the future may provide information on percentages of clogging.

The cross section shown in figure 4.31, also of the tested A24 sample, shows the relatively open structure of the needle punched geotextile, which provides many alternative flow paths should a site become blocked.

4.7.3.2. Terram® range
Figure 4.32 shows a plan view of the soil/geotextile interface surface of the T1500 sample after testing. Blocking is clearly observed by the accumulation of sediment particles at the geotextile surface. Figure 4.33 shows and enlargement of one of these blocking sites, showing a very high percentage of the available pore space being blocked off by sediment. This figure also shows how the melting of the fibres and also larger fiber diameter (compared to Bidim®) reduces the number, if not the size of available pore spaces. A similar observation is made from the cross section shown in figure 4.34 (it should be noted that the lower part of this cross section is obscured by glue used to secure the sample to the SEM stub).
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Filtration of Port Hills Loess for retaining wall situations

Figure 4.29 Plan view of tested Bidim® A24

Figure 4.30. Enlargement of clogging site in A24

Figure 4.31 Cross section of A24 test sample

Figure 4.32 Plan view of tested Terram® 1500

Figure 4.33 Enlargement of blocking site in T1500

Figure 4.34 Cross section of T1500 test sample
4.7.3.3. General
It was noticed with all geotextiles that small amounts of sediment were accumulating on the geotextile/free air boundary (that is the side of the geotextile not in contact with the soil). The accumulations were particularly prevalent for the Terram® range, and minimal for the Bidim® range (decrease of sediment accumulation on geotextile/air boundary with increase of geotextile grade within the Bidim® range). Figure 4.35 shows a series of plan views at increasing magnification of the geotextile/air interface for tested sample Terram® 1500. This reveals that not only is blocking occurring at the soil/geotextile interface, but significant accumulations of clay particles at the geotextile/air interface may also be affecting system performance.

Figure 4.35 Clay accumulations at the geotextile/air interface
It is recognised that in the field, geotextiles will generally be bounded on one side by soil, and on the other side by backfill material. Indications from the stage 2 test (see chapter 5) suggest that backfill materials commonly used on the Port Hills (eg tailings, river gravel) may be too coarse, and allow significant void spaces between the geotextile and the backfill. This situation is equivalent to that tested, where the geotextile is bounded on one side by "free air". Additional investigation into the effects of this phenomenon may be warranted, if appropriate backfills are not used in future.

4.7.4. Observations with granular filters
The cross section shown in figure 4.36 of the test sample filtered by 7mm premix. This very clearly shows a natural graded filter, or bridging network with the fine loess at the bottom grading into the sand particles of the filter. Unfortunately, the cyclone sand sample was not available for investigation in time. It is expected that a similar trend would be observed. However, due to the very low permeability of the cyclone sand system, it is very likely that the graded filter would appear much more tightly packed, causing the clogging (indicated by the permeability).

Figure 4.36 Well formed bridge network (natural graded filter) between 7mm premix filter (top) and loess.
4.8. Synthesis

4.8.1. Testing method

➢ The test method has provided a significant amount of information regarding the performance of a range of soil/filter systems. System permeability, mass passed over time, and observations of soil/filter interface structures all seem to provide consistent indications of system performance.

➢ A number of modifications to the current test method are suggested, including: addition of soil piping barriers; better support and securing methods for filter attachment; longer duration of testing.

➢ Small quantities of material passing through the systems has led to the development of a filter paper grain size analysis that must still be considered developmental.

4.8.2. Filter performance

➢ Results from permeability, mass passed over time, and observations of filter structure all indicate the suitability of the Bidim® range of needle punched geotextiles for filtering the specific soil type tested.

➢ Performance of the Terram® range of geotextiles is less favourable than the Bidim® range, with respect to permeability. Retention performance is similar.

➢ Concerns have been raised as to the suitability of the granular filters tested, particularly with respect to permeability performance, in comparison to the geotextile filter systems. Therefore use of 7mm premix or Cyclone sand as primary filters is recommended against, pending additional study.

➢ River gravel has been shown to be ineffective as a filter for loess, and should therefore not be relied upon as the only form of filtration in field situations.

4.8.3. Observations of structure at the soil/geotextile interface.

➢ Observations of the soil/geotextile interface indicate bridging as the dominant filter process for the Bidim® range. Bridge formation appears to be encouraged by soil mobilisation (minor movements) which encourages finer particles to pass through the system and leave coarser particles to establish a good bridge network. Additional work is needed to confirm this.

➢ Bidim® A34 was observed to have formed a vault network, which was effective in providing “good” permeability and comparable retention to the other geotextiles. The
vault network appears to have formed as a result of lack of soil mobilisation. However, other influences such as subtle variations in clay composition, may have influenced the formation of this network.

- The Terram® range showed indications of blocking at the soil/geotextile interface.
- The granular filter, 7mm premix, showed a well formed bridge network.

4.8.4. Future work

- Many relationships were only briefly observed under the SEM, and warrant further investigation. Particularly: the process behind the formation of a vault filtering network; percentage of clogging in needle punched geotextiles; percentage of blocking in heat bonded geotextiles; the processes and effects of clay accumulation at the geotextile/free air interface.
- Work by other authors suggests the possibility of long-term clogging with very thick needle punched geotextiles, suggesting the need for longer term tests (of the order of 1 to 2 years continuous testing).
- A number of variables may be trialed with this test, including soil type, water head, and other filter options. As the samples tested have only been slightly dispersive, it is strongly recommended that additional tests be performed with more dispersive samples before any sweeping recommendations can be made to the filtration of Port Hills loess in general.
Chapter 5
The Stage 2 Test
Chapter 5. **The Stage 2 Test**

5.1. **Introduction**

The concept of the stage 2 test is that of a laboratory simulation of a retaining wall design. The basic concept was suggested at a meeting with Marton Sinclair (of Eliot Sinclair and Partners, Ltd, Christchurch), and was subsequently developed extensively by the author and technical staff of the Department of Geological Sciences. The flexibility of the test is such that a number of situations can be investigated, with direct observations outlining potential problems with existing large scale field practises. A representation of the basic concept of the test is shown in figure 5.1

Figure 5.1 Basic representation of the stage 2 test.
5.1.1. Basic description of the test
The stage 2 test is essentially an acrylic tank measuring 500x500x800mm (external dimensions), with the walls approximately 6mm thick. One of the end walls (front) of the tank represents a retaining wall, behind which the drainage and filtration system can be varied. The majority of the tank consists of recompacted loess into which water is introduced by way of a water feeder system towards the rear end wall of the tank. A detailed outline of methodology for this test is given in appendix A.5. The following is a brief overview of the procedure.

Loess is compacted into the tank around the water feeder system (to approximate field densities), and behind a temporary support wall (allowing sufficient space between the final loess face and the front wall to place the filtration/drainage system). Once compaction is complete, the support wall is removed, and the resultant loess face is scraped clean, removing the surface that has been directly in contact with the support wall. The filtration/drainage system is then placed.

The particular filter being tested is placed against the compacted loess face, whilst the drainage medium is placed in a gently inclined and impermeable grooved drainage channel against the front wall (retaining wall). In the field the grooved drainage channel would typically be inclined concrete with a channel fashioned by a curved bottle while the concrete was still wet. In this case, an epoxy resin drain was fashioned and fixed in place dry. Once the filter and drainage medium has been placed, the backfill is then placed. Depending on the nature of the backfill it may be considered part of the drainage system, the filtration system or both (as is the case with river gravel, which provides both drainage and filtration).

Water is introduced to the system by way of the water feeder system, and then passes through the loess, into the filtration system, and then into the drainage medium. The incline of the drain channel is towards an opening in the side of the tank at the floor level in order to extract the water to either a collection beaker when measurements are being taken, or to a waste water drain at other times.

As this test is essentially an observational test, observations are recorded during the entire procedure to provide possible answers to later problems. Particular attention has been paid to water flow paths and sediment movement.
5.1.2. Objectives
The primary objective of this test is to investigate and compare the effectiveness of a range of filtration/drainage system designs behind retaining walls. The influence of tunnel gullies on retaining wall filtration/drainage systems is also investigated. There are a number of elements to most filtration/drainage systems, each of which has the potential to affect the overall performance of the system as a whole. It is therefore necessary to observe all possible elements of the system to identify any potential weaknesses. Results from this series of tests provide a practical basis for suggestions of good design practice for retaining wall filtration/drainage systems.

5.2. Discussion of Test Methodology
5.2.1. General
As no precedents of this type of test were found in the literature, the experimental procedure was adapted as problems were encountered. The scale of the tests and setup time means that only three situations have been tested (see section 5.2.2 for specific situations tested), each looking at different retaining wall geometries and also with slight differences to general test procedure. As a result of these factors, this test should be considered still in development, and it is hoped that this development will continue by subsequent researchers in the near future.

I. Soil Collection
The soil collection procedure was as for the stage 1 test. Approximately 300kg bulk samples of loess were collected for each test. Initial thoughts of obtaining in situ samples were abandoned due to time and logistical considerations. However, it is considered that efforts to obtain useable in situ samples in future studies would be very valuable.

II. Soil Preparation
The disaggregation method outlined in appendix A.5 section 4 is seen as satisfactory, given the volume of soil required. The removal of large calcareous concretions could possibly have influenced the performance of the system, unfortunately they would have complicated the compaction process significantly if left in.
Mixing was thorough in order to obtain relatively uniform grain size distributions, and also to ensure even moisture content for each individual layer. Tap water is used, as a result of the required volumes, to obtain moisture contents of approximately 10% (±2%) for satisfactory compaction and ease of handling. This is dry of the 15% optimum moisture content found by Jowett (1995) for Ahuriri quarry loess.

III. Soil Placement and Compaction
Placement and compaction of the soil was guided by the NZS 4402, 1981 test 4, and also by practical considerations. Although laborious, the compaction process is efficient at achieving dry densities of approximately 1700kg/m³ (similar to in situ density). The primary focus of the compaction method is for ease of reproduction, and approximation of field conditions.

IV. Placement of the Filter and Drain
As different filters have been used in each of the three tests, the specifics will be outlined in sections 5.2.2 I, II and III. In each case the filter is placed directly against the cut loess face, with emphasis on obtaining maximum contact between the loess face and the filter. Where geotextiles are used as filters, the option to wrap the filter around the drainage medium is available. In this instance, the drainage medium (Novaflo®) is placed into the fixed drainage channel at the same time as the geotextile. In the case of granular filters, the drain (also Novaflo®) must be placed first. It was hoped that Megaflo® would be tested as a drain, but due to time constraints, this was not possible.

V. Water Filling Protocol
In all cases, tap water is added slowly, and then maintained at a constant head just below the soil surface level. Test C1 was the first to be set up and used a section of perforated sewer pipe, wrapped in Bidim A64 as a feeder pipe. One of the reasons this option was chosen, was to simulate a point source, as for case study number 2 (chapter 3), where water from a leaking down pipe was the primary source of moisture on the retaining wall. Initial pilot tests, stage 1 tests and standard tests, had all indicated that saturation would be rapid, however, this proved not to be the case. Another problem with the water filling system for test C1, resulted from the geotextile being wrapped around the sewer pipe. The thick geotextile acted like a wick, and drew water from the feeder pipe to the surface. Over night,
the water from this process pooled on the surface of the compacted loess and began to flow towards the front wall. Significant erosion resulted and will be discussed in greater detail in section 5.2.2 I.

As a result of these problems, both subsequent tests replaced the sewer pipe water feeder with a section of Megaflo®. This approach speeds saturation of the system significantly. Additional wrapping of the Megaflo® with Bidim A64 was done around the bottom and front face to a level just below the soil surface. This is to ensure that a minimum of material will be removed from the system into the water feeder system.

VI. Data Collection and Sampling
Water samples are collected periodically by placing a pre-weighed collection beaker at the bottom of a drain chute, which leads directly from the open end of the drain channel. When removed, the beaker is reweighed to obtain an accurate representation of volume of water and non-water material passed (subtraction of the non-water fraction is possible by evaporation and reweighing). In an attempt to reduce loss by evaporation, cling wrap is placed over the length of the drain chute, and collection beaker. Other moisture losses from the system are minimised by closing the system with an acrylic lid (tests C2 and C3) or by covering the whole test surface with cling wrap (test C1).

5.2.2. Specific Geometries

I. Test C1
For this test the filter was Bidim A24 which is wrapped around the Novaflo® drainage pipe, and placed along the full length and height of the loess face. The backfill material is tailings, and consists of rounded greywacke clasts typically 20-40mm intermediate diameter. Figure 5.2 shows a representation of this test geometry. This geometry is commonly used in retaining walls on the Port Hills. During the placement of the backfill, emphasis must be placed on allowing the filter to conform to the shape of the loess face, but not allow the formation of wrinkles or folds.
II. Test C2
The filter tested in C2 was cap 75 river gravel (see appendix A.3.4 for grain size distribution). For this system, the filter is the backfill, and is therefore an integral part of the drainage system. River gravel has been observed in use in the field, most commonly for either non-critical walls, or those that require the backfill to be load bearing. To assist in providing intimate contact between the backfill and the loess face, the backfill material was lightly compacted (see appendix A.5.6.3 for details). This is not out of line with some field cases where the backfill must be load bearing. Figure 5.3 shows the geometry for this test.
As a direct result of the failure of the water feeder system, and slow saturation of the loess in test C1, this test used Megaflo® as a water feeder, providing a significantly increased surface area of loess directly exposed to the water feeder. As mentioned in section 5.2.1.V, the Megaflo® was wrapped around the bottom and the front (to a level just below the soil surface) with Bidim A64. Although there was no evidence to suggest the existing filter wrap on the Megaflo® would not be sufficient, it was considered prudent to provide the additional filter to prevent sediment loss from the system into the water feeder as this could not be retro-fitted.

For this test, the initial layer of soil was placed prior to the installation of the water feeder. At the rear wall, a small amount of loess was removed after compaction, to allow the water feeder to sit securely in the tank. Allowing the water feeder to rest directly on the soil in this way provides an even greater area of soil to be in contact with the water and speed the saturation process even further.
III. Test C3

This test investigated the effect of an artificially created tunnel (early to mid stage of a tunnel gully development) on a specialised filtration/drainage system. Compaction and water feeder processes are the same as for test C2. The method of creating the tunnel is detailed in appendix A.5.5.b. As a result of the inclusion of the artificial erosion cavity, a more complex filtration/drainage system is used as shown in figure 5.4. Bidim A14 is wrapped around the Novaflo® pipe, and placed along the whole length of the wall, but to a height just below the level of the outlet of the erosion cavity. This is then backfilled with 7mm premix to a similar level, where spare Bidim A14 is then laid overtop of the premix from the loess face, to the retaining wall. On top of this, tailings are placed to the full height of the loess face.

Figure 5.4 geometry of stage 2 tests C3 (dry density = 1750kg/m³)

Marton Sinclair of Eliot Sinclair and Partners (Christchurch) has recommended this geometry for use in retaining wall situations where erosion cavities are present. The particular advantage with this geometry is the additional filtration provided by including both a geotextile and a granular filter (premix). As the filtration system is not in direct contact with the loess face at the erosion cavity, this geometry concentrates on reducing the amount of sediment reaching the drain pipe to avoid excessive siltation. The drawback with this approach is that there is no attempt to reduce erosion from the cavity, which may continue to
this approach is that there is no attempt to reduce erosion from the cavity, which may continue to develop. It is suggested that if this is considered a threat to the project, then remedial measures as suggested by Yetton (1986) may be necessary. It should be noted that such remedial measures may significantly increase project costs.

It was expected that moisture would preferentially accumulate in the created void space, and eventually become free draining. Due to the highly erodible nature of loess, this free draining water should have eroded material from the tunnel floor. A problem identified with the method used to create the tunnel erosion cavity was the “over compaction” of the soil in direct contact with the aluminium pipe during compaction of subsequent layers. Two potential solutions to this problem are:

1) Cleaning the sides of the tunnel, by way of scraping with a piece of wire, subsequent to the removal of the pipe.

2) Cutting an additional narrow channel directly underneath the aluminium pipe prior to placement of subsequent soil layers.

5.3. Analysis and Observations

5.3.1. Permeability Over Time

As mentioned in section 5.2.1.VI, samples of water are collected periodically for each test over a known time. These samples form the basis of the permeability data obtained. Calculations for permeability are based on the entire loess face area. Calculations on this basis may yield a misleading representation of the system permeability as seepage may only be occurring over significantly smaller areas. As the true area of seepage can not be accurately estimated without derigging the test, the full area is assumed for tests C1 and C2. Therefore, permeability values obtained will only represent apparent permeability and are in no way comparable to a base level of typical loess permeability. However, changes in apparent permeability will indicate changes in system performance.

5.3.2. Sediment Movement Over Time

Visual investigation of collected water samples, and investigation of material remaining subsequent to evaporation of that water, has been the primary source of information on material passing over time. Additional observations of the system as a whole have provided other, non-empirical information of the movement of sediment through the system over time.
5.3.3. Flow Throughout the System
The provision of manometer ports through the system was intended to provide information on water pressure throughout the systems. It was hoped that, using a similar approach to that of the ASTM D5101-90 test, gradient ratio values could be calculated. Unfortunately, information has been very limited from these ports. The few results that have been obtained are discussed in various parts of section 5.4. In addition to this information, observations have played a key role in identifying potential problems with water flow within each system.

5.3.4. Other observations
As very little empirical information has been forthcoming from these tests, observations have formed a major part of the analysis of each system’s performance. It should also be noted that a number of the observations represent weaknesses in the test method that may be improved upon in subsequent studies.

5.4. Results for Test C1

5.4.1. General
As this was the first test to be set up, a number of problems were encountered. As mentioned previously, the geotextile wrap around the sewer pipe water feeder tube acted as a wick and drew significant quantities of moisture to the soil surface. Overnight, this water pooled on the soil surface, and began to flow toward the loess face. On reaching the face, the water flowed down the drain side wall primarily, but some water also migrated along the face at the top of the slope to the other wall, where it also flowed down. Figure 5.5 shows a schematic representation of the sequence of events resulting in surface erosion from the flow of water over the surface of the soil. Impacts of this event on the test will be discussed in the next four subsections. Although this surface erosion event could be viewed as a significant failure of the testing procedure, it does highlight a very important point. That is, that surface erosion of loess can be dramatic over relatively short time periods (in this case overnight), and with only small flow rates. This factor re-emphasises the need for adequate surface drainage measures in conjunction with well designed filtration/drainage systems, for efficient retaining wall design.
Figure 5.5 Sequence of events leading to surface erosion in test C1. Total time for entire sequence was approximately 14 hours (overnight). Please note that the above diagrams are plan views of the top surface of test C1.

1) Initial pooling of water on the soil surface.
2) Movement of water toward the loess face.
3) Movement of water along the loess face between the loess and geotextile.
4) Initial development of erosion and continued water flow over the surface.
5) Continuing development of surface erosion.
6) Final extent of surface erosion.
5.4.2. Permeability Over Time

Following the initial failure (creation of the surface erosion), saturation of the remaining system was slow. It was some 2 months before a sustained, albeit slow, flow rate was established. Data collected for permeability is displayed in figure 5.6 below.

Figure 5.6 permeability for stage 2 test C1 over time.

No clear trends are apparent from the permeability data. As mentioned previously, these values are apparent permeability only and should not be compared to the base line of 1.41E-08 m/s obtained by Jowett (1995) for the following reasons:

1) Flow paths in the system may not be fully established yet
2) Seepage is coming from an area less than the full face
3) Moisture may be lost elsewhere in the system (e.g., evaporation, condensation on backfill and walls)
4) Some part of the system may be clogged.

With regards to clogging, or more strictly reduced permeability of the system, performance of the filter may not necessarily be the cause. The considerable disturbance in the early stages of this test may very possibly have resulted in some areas of higher than normal concentrations of clay particles which will in turn, reduce the permeability in those areas. Close observations of the geotextile around the areas that were involved in the erosion event show these expected accumulations of clay particles are present. Clogging would best be identified by a continuing reduction in apparent system permeability which is not evident in this data set.
After a period of approximately 2 1/2 months, formation of both brown and green algae was noted and continued to develop from this time on. As these algae may have impacted on system performance through biological clogging, it is suggested that future efforts include algae inhibitors for the water supply. In addition, it may be useful to store the tests in a dark area while running to further limit algae growth. Unfortunately this factor did not appear until after the final test was set up, and so this problem has occurred in test C2, and would likely have occurred in test C3, had time allowed it to continue.

5.4.3. Sediment Movement Over Time

The early erosion caused a significant amount of loess to be deposited in the Novaflo® piping, and also to be removed from the system entirely. Visible sediment movement within the system since then has been minimal. Data on mass passed over time is shown in fig. 5.7.

Figure 5.7 Mass passed over time for test C1.

A distinct increasing trend is apparent from the above graph. However, it should be noted that at least 90% of the material weighed in this mass passed fraction, are recrystallised salts. The origin of these salts has been established as the backfill material. As these salts dominate the fraction so dramatically, it is difficult to assess how much material passing is in fact sediment from the system. The large total amount of salts, and the fact that they are still being collected 100 days after test initiation, suggests a need for more in depth investigation of the effect they may have on the various elements of a real life system (eg geotextile, Novaflo®, water proofing agents, and perhaps the masonry).
5.4.4. Flow Throughout the System

Very little information was provided by the manometers. This may be due to the water feeder being a point source in the middle of the tank. Flow from the drain channel was observed to originate from under the geotextile wrapped around the Novaflo®, even after the significant amount of siltation that occurred during the early erosion event. This raises a point of concern with current field practise, which often involves the Novaflo® being placed directly onto excavated loess. Since this has been observed to be a preferential flow path, the likelihood of accelerated erosion under the Novaflo® is high. Therefore, the use of impermeable drain channels (such as concrete channels) under the Novaflo® in the field is recommended.

Condensation build up was noticed in many areas. The end wall (retaining wall) typically was covered in small water droplets. Although it is recognised that the particular test method used exacerbates this situation, it is thought that condensation would also form in a field situation. This suggests the need for water proofing over a large percentage of the wall, if this level of water contact is deemed to be a concern, as is often the case for residential retaining walls.

Pooling of water was noticed in three main locations. Firstly, inside the Novaflo® in the corrugations not filled by sediment. This is to be expected, and is not a problem, provided the drainage pipe is resting on an impermeable drain channel. Water was also seen to be pooling to a height of approximately 10mm directly against the retaining wall end, on the flat surfaces of the drain channel. This suggests that it is prudent to place the drain channel at a level below the floor slab, and also to attempt to prevent flat areas or areas sloping towards the retaining wall on the drain channel edges. The final pooling site was where the geotextile that was wrapped around the Novaflo® came in contact with the retaining wall end. Again, up to 10mm of water was observed, and suggests a need for further investigation of the performance of geotextiles at very low hydraulic heads. On the practical side, ensuring the Novaflo® is far enough away from the wall to allow a geotextile wrap not to contact the retaining wall, should eliminate any problem.

5.4.5. Other observations

The erosion pattern arising from the initial event, suggests that contact between the geotextile and the backfill was not sufficient to allow the geotextile to perform to its
potential. This is primarily due to the fact that the grain size and clast roundness of the backfill is such that significant areas are not in contact with any backfill grain. This, combined with the relatively poor permeability performance of geotextiles at very low confining pressures means that the water was able to flow directly down the loess face, initiating and continuing the erosion. If pressure had been more uniform on the geotextile, there would be no free space for the water to flow between the geotextile and the loess face. The water would then have run through the back fill to the drain, minimising erosion.

5.5. Results for Test C2

5.5.1. Permeability Over Time
The inclusion of a section of Megaflo® as a water feeder system for this test significantly improved the time until measurable flow was produced in comparison to test C1. The results obtained are plotted on figure 5.8. It should be remembered that these values represent apparent permeability, as the area of seepage is likely to be less than that of the full loess face (which has been used for calculations). No firm trends are shown by the data, and no conclusions can be drawn from the data. Longer term testing may reveal some trends.

Figure 5.8. Apparent permeability vs time for stage 2 test C2.

5.5.2. Sediment Passing Over Time
A graph of mass passed over time is presented in figure 5.9. As with test C1, a significant portion of the mass collected was salts. The values obtained are similar to those for the C1 test. However, an increased sediment content was observed from approximately 60 days.
onwards. The approximately stable overall mass suggests that this slight increase in sediment passing may be off set by a slight reduction in the mass of salt passing. This is an indication that the salt within the system is progressively diminishing. Unfortunately, termination of the test due to time constraints has not allowed this trend to be investigated further.

The exact origin of the visually observed sediment passing could not be determined. The two possible origins are: fines from the backfill material, or loess. Regardless of the origin, the observation of sediment passing through the system from approximately 60 days onwards suggests a lack of stability within the system. Wash out of the fines from the backfill indicates a progressive reduction in filtration capacity, whilst wash out from the loess would suggest inadequate performance of the river gravel as a filter.

Figure 5.9 Mass passed over time for stage 2 test C2.

5.5.3. Flow Throughout the System
The water discharging from the system was observed to be coming from directly under the Novaflo® pipe, along the impermeable drain channel. Further investigation of the Novaflo® indicated that no moisture whatsoever was visible at any point inside the drain pipe for the entire duration of the test. This observation has significant implications to situations where the drain is placed directly onto excavated loess. In these situations, flow under the drain pipe will encourage erosion of the loess over which the flow is occurring, and may lead to undermining of the retaining wall in severe cases.
Significant moisture was observed to be held against the retaining wall end of the test. This is both in the form of condensation, and pore water of the finer material in the backfill. This water against the retaining wall end indicates the potential for ongoing seepage into and through porous retaining walls without adequate water proofing.

5.5.4. Other Observations

General observations of the appearance of the backfill material in the test indicated a significant potential problem. The maximum grain size of the backfill used in this test was 75mm (intermediate diameter). These large grains were commonly seen to be arranged in such a fashion as to produce a "shadowing" effect (see figure 5.10 for a schematic representation). The result of this effect is the formation of void spaces of significant size.

Figure 5.10. Schematic representation of shadow effects caused by large grains in the backfill.

Although these shadows are not a problem on the retaining wall, they will be a problem on the loess face. Wherever a shadow creates a void on the loess face, that area has effectively no filtration, and has the potential to erode freely. Water from both condensation and pore water from the fines of the backfill may assist in this erosion. Compaction of the backfill certainly reduced the problem of shadow formation, but did not eliminate it.
5.6. Results for Test C3

5.6.1. Permeability Over Time

As with test C2, the time required for measurable flow was significantly improved as a result of the use of Megaflo® as a water feeder. Values of apparent permeability are similar to test C2, suggesting the intended function of the artificially placed tunnel in the test was not as successful as hoped.

Figure 5.11. Apparent permeability vs time for stage 2 test C3.

5.6.2. Sediment Passing Over Time

Although the trend is not clear from the data obtained, there appears to be a slight decreasing trend towards the end of the test, from about 38 days onward. This may support the indication given in test C2 that the amount of salt in the system is reducing measurably after this time. The trend is more visible in this test, as no visible sediment was noticed coming from the test, thereby making the mass passed dominated by salts.

Unfortunately, the primary aim of this test (to assess the influence of tunnel gully erosion on a filtration/drainage system) was poorly achieved. Had water been free flowing from the artificial tunnel, then significant levels of erosion would have resulted. It was hoped that the two level filtration system (geotextile plus premix) would have been effective in reducing the level of sediment reaching the Novaflo® piping to satisfactory levels.
Implementation of the suggested procedural changes given in section 5.2.2.III, will be necessary before this can be assessed.

Figure 5.12. Mass passed over time for stage 2 test C3

5.6.3. Flow Throughout the System

Although moisture was noted to be preferentially coming from the artificial tunnel, free running water had not been observed from the tunnel by the end of available testing time. As this was the intended outcome, the lack of flow can be attributed to deficiencies in the testing method. Remedial options to the test method for the formation of artificial tunnels have been presented in appendix A.5.

Small amounts of water were noticed to pool on the geotextile directly below the tunnel opening. Owing to the geotextile being placed just short of the side wall (an error in placement, the geotextile should be against the side wall), the full potential extent of this pooling could not be observed.

As with test C2, water flow was noticed to be exclusively from under the Novaflo®, and the inside of the Novaflo® was also noted as having no moisture visible for the duration of the test. Also as with C2, the fine particles in the premix backfill retained moisture against the retaining wall. However, there was little evidence of condensation as such. This is primarily due to the fact that the finer grain size of this backfill did not result in large void spaces (shadows) where condensation commonly forms.
5.6.4. Other Observations
The most striking advantage of this system, other than the two-fold filtration, is the effectiveness of the premix backfill for providing even pressure on the retaining wall and therefore the loess/geotextile face. That said, there is a tendency for this backfill material to segregate noticeably on filling. Methods for backfilling attempted to simulate the common field practise of tipping the material directly into the system. This method very clearly results in segregation, which further results in differences in filtration capacity and performance throughout the test. In areas where segregation has resulted in a high concentration of coarse particles, filtration may not be adequate, and loess fines may be free to pass through. Further investigations are suggested to provide readily applicable solutions to avoid this potential problem.

5.7. Discussion

5.7.1. Empirical results
Results of permeability and mass passed over time have been presented. However, there are strong indications that these may not be accurate indicators of system performance under the conditions used. Firstly, the scale of the tests suggests equilibrium may take significantly longer, than was available for testing, to be established. Also, problems associated with these prototype experimental methods mean that comparison between systems is not valid.

5.7.2. Pooling on geotextiles
The observation of minor pooling on geotextiles in tests C1 and C3 has prompted further investigation. A test was conducted, with the full range of available geotextiles being subjected to very small hydraulic heads in a funnel configuration. Full details of methodology and results are given in appendix A.6. Results suggest that it may be relatively simple to dramatically improve geotextile permeability with low heads at low confining pressures. However, more investigation of this is suggested.

Possible impacts of this minor pooling tendency in a Port Hills retaining wall situation include: ongoing seepage into porous retaining wall media (where water proofing is inadequate); and diversion of water onto the loess face (permitting erosion).
5.7.3. Presence of salt in backfill

All tests have indicated the presence of salt in the backfill material. The quarry these materials obtained from is a common source for backfill material used on the Port Hills. Observations reported by Markham Distributing (distributors for Aquaron concrete preservation solutions) state that deterioration has been noticed in existing concrete structures (in some cases less than 20 years old) as a result of "chloride ion ingress" initiating corrosion in steel reinforcing (Anonymous, 1998). The presence of salt on backfill material, the common level of condensation and minor pooling of water observed in the stage 2 tests suggests a strong potential for long term corrosion of retaining wall steel reinforcing. Before any recommendations can be made regarding this problem (other than reinforcing the need for water proofing over the whole wall), more study needs to be done on the exact nature of the salt and it's likely impact on retaining wall systems.

5.8. Conclusions

The stage 2 tests have attempted to simulate field conditions as closely as possible for three retaining wall designs commonly used on the Port Hills. The test results indicate a need for longer testing (a minimum of 6 months is suggested) and further development of the test methodology. A summary of the findings from each test is given below.

Test C1

- The tailings backfill is too coarse to provide sufficient contact between the geotextile filter and the loess as evidenced by the free running of water between the geotextile and loess face which exacerbated a surface erosion event. The lack of uniform pressure on the geotextile allowed the water to run down the loess face behind the geotextile, thereby eliminating the geotextile's function as a filter.

- The tailings backfill does provide relatively free drainage but does not prevent some condensation resting on the retaining wall under certain conditions.

- Surface drainage is a vital aspect of any retaining wall system (particularly during construction) to prevent the formation of erosion features that may influence the effectiveness of a system design. Also to reduce the possibility of accumulation of clays on geotextile filters, altering their performance. This is suggested by the observations made in relation to the erosion event that occurred in the early stages of the test.
> Condensation and minor water pooling against the retaining wall reinforces the need for water proofing to prevent slow seepage into porous retaining walls.

Test C2
> Void spaces created by shadowing effects of large grains produce areas of no filtration, leaving potential for uninhibited erosion. This emphasises the unsuitability of river gravel as a primary filter, indicated by the stage 1 test.
> Compaction may reduce the above effect, but not eliminate it.
> Observation of sediment after approximately 60 days indicates system instability, indicates either the reduction of filter capacity, or the ineffectiveness of the river gravel as a filter for loess.

Test C3
> Moisture preferentially comes from the artificial tunnel erosion cavity, but over the course of the test had not become free flowing, suggesting a longer test duration and the implementation of suggested improvements to the creation of the artificial tunnel erosion cavity.
> The 7mm premix backfill provides excellent contact between geotextile and loess face, but does produce a more constant level of pore water on the retaining wall.
> Placement procedures used in the field for placing backfills such as 7mm premix are very susceptible to creating segregation effects. Further study is needed to investigate readily applicable options for reducing these effects.

General
> Presence of significant amount of salt in backfill suggests need for investigation into long term effects of salt on all aspects of a retaining wall system (particularly influence on steel reinforcing within wall). Detailed investigations as to the origin and nature of the salts are recommended.
> Water discharge from directly under the Novaflo® piping observed in all tests (even when drain pipe was partly silted) suggests a definite need for an impermeable channel for the Novaflo® to rest on to prevent erosion of loess under drain pipe.
> Minor pooling of water on geotextiles suggests a need for further investigation of geotextile permeability performance at low hydraulic heads.
Chapter 6
Gradient Ratio Test
Chapter 6. **ASTM D 5101-90 Standard Gradient Ratio Test**

6.1. Introduction

Rod McKenna (formerly of Geosynthetic Testing Services, Australia) recommended the use of this standard test for obtaining comparisons of filter performance with loess. This American Society for Testing and Materials (ASTM) test designation D 5101-90, is a version of a gradient ratio test. As gradient ratio (GR) is becoming a more widely recognised parameter, indicative of filtration performance, this test is seen as appropriate for comparative purposes both within this study, and to other studies. Permeability is also seen as a vital aspect of any soil/geotextile system, and is also indicated by this test.

For the purposes of this test, hydraulic gradient can be defined as the change in water head over a known thickness of soil. Gradient ratio is a measure of the change in hydraulic gradient throughout a soil/geotextile system. Calculations for gradient ratio are given in appendix A7.2 (section 10). Figure 6.1 a and b below illustrate opposite extremes of what may occur in a soil/geotextile system. In both of these figures, the hydraulic gradient is represented by the slope of the curve. Gradient ratio can be thought of as a comparison of the curve slope over the middle section of the soil (75mm to 25mm) to the section near the geotextile (25mm to 0mm). The indicated 6mm position relates to suggested adaptations of the ASTM D 5101-90 test method by Austin et al, (1997).

Figure 6.1. Distribution of water head for two conditions in a soil/geotextile system (Austin et al, 1997)

- a. Piping
- b. Blinding/clogging
6.1.1. Basic description of the test
The soil to be tested is held in the centre section of a soil/geotextile permeameter, with the geotextile to be tested held in direct contact to the soil directly underneath (as shown in figure 6.1). The geotextile is supported by a 5mm metal mesh. Water enters the system in the top section of the permeameter from an inflow constant head device (I/CHD), and leaves the system from the bottom section to an outflow constant head device (O/CHD). Adjustment of the CHDs allows the hydraulic gradient of the system to be set as required. Six manometers are positioned over the three sections of the permeameter to provide information on piezometric levels throughout the system, which may subsequently be used for gradient ratio calculations. Permeability of the system may be established by collection from the overflow of the O/CHD over known times.

Figure 6.1. Basic set up for the ASTM D5101-90 gradient ratio test.

6.1.2. Objectives
The primary objective of the gradient ratio test for this thesis is to provide a quantitative, repeatable and widely recognised comparison between various filters. Gradient ratio information provides indications as to degree of clogging of each system. Permeability data also indicates system performance. The overall comparisons obtained from this series of
tests provide a useful basis for evaluation of the stage 1 test. If not directly comparable due to differences in hydraulic conditions between the two tests, then performance of filters relative to one another will indicate the reliability of the stage 1 test for comparisons of filter performance with site specific soils.

6.1.3. History of the Gradient ratio test

According to Christopher and Holtz (1985), the first gradient ratio test was developed in 1959 by Soil Testing Services Inc to test the performance of woven filter fabrics. Further development of the test by Calhoun (1972) lead to the test being adopted by the US corps of Army Engineers. Testing within the Army Corps of Engineers resulted in the specification of GR=3 (see section A.7.2 section 10 for calculation of GR) as a minimum limit for accepting a geotextile for use with a particular soil type. Recognition of this test method’s usefulness for indicating clogging potential, led to the American Society of Testing and Materials adopting a version of the test (as given in appendix A7.2). The ASTM D5101-90 notes that GR<1.0 "indicates internal instability of the soil with some of those particles adjacent to the geotextile moving out of the system.", and that GR>1.0 "indicates system clogging or restriction at or near the surface of the geotextile". Most importantly, the ASTM D5101-90 states that "The allowable gradient ratio value for various soil/geotextile systems will be dependent on the specific application. It is the responsibility of the design professional to establish this allowable value on a case-by-case basis."". Therefore, the suggested maximum value of GR=3 for rejecting a geotextile given by the US Army Corps of Engineers should only be considered as a guideline as specific projects may have specialised requirements permitting higher, or requiring lower values.

6.2. Discussion of test Methodology

The full ASTM D5101-90 test procedure is included in appendix A7.2.

6.2.1. Soil Collection

Soil was collected in a single bulk sample from the same location (as for stage 1 test) for comparative purposes. Sub-samples of loess from each individual test have been taken for grain size analysis and are given in appendix A3.4 and show minimal variation.
6.2.2. Soil Preparation

It is widely noted that extreme care must be used during the soil preparation phase. Owing to the fine particle size of the loess used for this series of tests, all soil has been reduced as close to constituent grains as possible (as recommended by Rod McKenna, pers. comm, 1997). Also as a result of fine particle size, air drying times are all greater than two weeks to ensure the samples are thoroughly dry.

6.2.3. Soil Placement and Compaction

Placement follows the standard. However, compaction methods have been slightly altered. Instead of the 6 taps specified for compaction of each layer, 10 taps on each quadrant (for a total of 40 taps) have been used. This is done in an attempt to alleviate the problem of soil shrinkage found to occur in an initial pilot test on saturation of the dry sample. The increased compaction reduced this effect but did not eliminate it. This shrinkage is discussed further in section 6.4.4.1. The compaction method resulted in an average dry density for all tests of 1429 kg/m$^3$ ($\pm 2.8\%$), which is very loose in comparison to typical field dry densities of 1650 kg/m$^3$.

The compaction procedure given in the standard was of concern (appendix 7.2, section 9.4.2 of the standard). The compaction method of tapping the side of the permeameter 6 times with a certain sized wooden rod appears a less than satisfactory description. For instance, the use of balsa wood, compared to hard wood, would have a significant influence on the level of compaction, as would the force of the “tap”. Another factor to be considered is where to tap. Again, significant difference in result would be obtained if tapping at the current soil level, or the base of the inverted permeameter. It is suggested that an approximate weight of the wooden rod be specified along with an indication of how hard to tap and where, or at the very least an indication of the desired result of compaction be given in the standard.

6.2.4. Treatment and placement of the Filter

As a result of the ‘funnel test’ (see appendix A6) saturation of the geotextile prior to placement in the permeameter was by way of pouring de-aired water over the geotextile and then allowing it to stand for 2 hours in de-aired water. The pelting action of the pouring water has been indicated (by the funnel test) to be a more effective method of saturating the geotextile. Otherwise, treatment and placement of geotextile filters is as per standard.
Placement of the cyclone sand filter (see appendix A3.4 for grain size distribution) is somewhat of a departure from the standard, as it only has provision for geotextile filters to be tested. Detail of how this granular filter was placed is given in appendix A7.1.

6.2.5. Water Filling Protocol and Treatment
Tap water was used for all tests due to its very low level of impurities. However, dissolved oxygen levels directly from the tap were of the order of 12ppm. This was reduced by way of heating the water to slightly above room temperature. This was found to be effective in lowering the dissolved oxygen levels of water entering the test apparatus to approximately 6ppm as required by the standard.

Observations in the first two tests showed significant disruption of the soil at the soil/geotextile interface when the recommended backfilling procedure was followed. As a result of this, backfilling was done at a significantly slower rate. This adjustment produced less visible disturbance of the soil at the interface, but did not eliminate all disturbance.

All tests were carried out at a hydraulic gradient \((i)\) of 1, with Bidim® A24 and Terram® 1500 being subjected to hydraulic gradient of 2 after system equilibrium had been established at the lower level. Hydraulic gradients of 1 are indicated (by McKenna, 1995 and Fannin et al, 1998) as typical for de-watering trenches, and application to which most Port Hills retaining wall situations can be readily compared from a hydrological perspective.

6.2.6. Data Collection and Sampling
Water is collected from the overflow of the O/CHD continuously from the start of the test until its completion with approximately 2 minute breaks during measurement (accounted for in permeability calculations). To reduce loss from the system to evaporation (which may have had a significant influence at the low permeabilities measured) all open parts of the system were covered in cling wrap, with the exception of the open end of the manometers. These were not covered so as not to unnecessarily influence pressure readings. Loss from these small openings is thought to be minimal.

Measurement of all manometers is done in sequence at approximately the same time that the volume of water passed since the last measurement is checked. Each manometer is measured from a common reference elevation to an accuracy of ±0.5mm. Notes are also taken on temperature to note any deviations from the norm, and also on the general appearance of the system that may indicate piping or any other visible phenomenon.
6.2.7. General Discussion
As with all aspects of geosynthetics, development and study of the gradient ratio test is ongoing. As a result of this ongoing study, a number of limitations of the gradient ratio test have been identified. Koerner (1998) lists the following factors that may influence the accuracy of results obtained: piping along test cylinder walls [reduced but not eliminated by piping barriers]; possibility for air pockets in the soil, geotextile, and head monitoring system. Halse et al (1987) also notes the long term instability of the gradient ratio value as an additional complication of the test. Christopher and Holtz (1985) suggest this test be restricted to system comparisons as a result of sample preparation not realistically modelling field conditions. In addition to this, Christopher and Holtz (1985) warn of the extreme care needed for this test method to provide repeatable results. Particular attention was paid to each of these limitations and concerns throughout this comparative testing program.

6.3. Analysis
6.3.1. Permeability Over Time
The three elements of data required for the permeability calculations are: area of geotextile (known from the dimensions of the permeameter); time over which the measured volume of water has passed; and finally the total volume of water passed through the system over the known time. Measurements are taken regularly, with the time of the measurement being recorded to the nearest 10 seconds. A minimum of 100mls must be collected to keep error as low as practicable. The reading is taken, and the collection cylinder replaced, the time of placement being also recorded to ±10 seconds, as this represents the start time for the next collection period. Permeability results must be considered to be an average over the period of collection, rather than a true representation of the permeability at the time of measurement.

6.3.2. Gradient Ratio
As mentioned in section 6.1, gradient ratio is a measure of change in hydraulic gradient throughout a soil/geotextile system. Measuring water head at fixed points throughout the system, by way of manometer ports provides the required data for hydraulic gradient calculations over specific sections of the soil sample. The hydraulic gradient of the soil section close to the geotextile is divided by the hydraulic gradient of the soil in the centre section of the permeameter to give gradient ratio values. A high gradient ratio is indicative
of reduced permeability close to the geotextile as a result of clogging (in its various forms). Low gradient ratio is indicative of increased permeability near the geotextile resulting from piping. Precise calculations for gradient ratio are given in appendix A7.2 (section 10).

6.3.3. Other
Observations recorded throughout the test period are compared to results obtained for permeability and gradient ratio to establish any possible outside causes for changes in system performance. On completion of the test, the water is allowed to drain through the soil naturally (once the inflow has been turned off, and the O/CHD lowered to allow flow through the soil). The soil is then allowed to dry in the permeameter to allow the possibility of collecting samples for investigation under the SEM. Unfortunately, very few samples have been investigated in detail under the SEM. This is primarily as a result of disturbance to the interface during test dismantling. Time has also been an influencing factor.

6.4. Results
For ease of comparison, permeability results are plotted on p148 and gradient ratio results on page 149. All plots have been made at scales most appropriate to the individual system performance, so careful inspection of scales should be made when making comparisons.

6.4.1. Bidim® A14 [permeability results: fig. 6.3; gradient ratio results: fig. 6.9]
Although this sample was only tested for just over 12 days at a hydraulic gradient of 1, the system had reached stability both in gradient ratio, and in permeability in this time. Equilibrium gradient ratio was just under 2.0. Permeability established equilibrium after only 5 days at a steady $7.5 \times 10^{-7} \text{ m/s}$. A very brief period of visible sediment passing through the geotextile was noted from about 2.5 – 6.2 days from test set up. No sediment was observed passing after this time.

6.4.2. Bidim® A24 [permeability results: fig. 6.4; gradient ratio results: fig. 6.10]
This sample was held at a hydraulic gradient ($i$) of 1 for 23 days, and then subjected to a hydraulic gradient of 2 for the remainder of the test. A small increase in gradient ratio is noted with the increase of hydraulic gradient; from approximately $1.7 @ i=1$, to $2.0 @ i = 2$. Although some minor variations are noted in the permeability curve shortly after the change of hydraulic gradient, the system permeability remains close to its equilibrium value of $6.5 \times 10^{-7} \text{ m/s}$. Slight piping was observed between 2 and 8 days only.
6.4.3. Bidim® A34 [permeability results: fig. 6.5; gradient ratio results: fig. 6.11]

The influence of bubbles forming within the test cylinder can be readily seen in the results of the A34 test. Values of gradient ratio of approximately 3 in the first day are directly attributed to a bubble that was cleared after this time. Although the gradient ratio performance subsequent to the clearing of this bubble appears to be slowly approaching an equilibrium value of approximately 2.5, the low equilibrium permeability of 1.5x10^-7 m/s, and observations during the deriging of the test, suggest other bubbles may have been present and affecting results over the course of the test. Ideally this test should be re-run. However, gradient ratio appeared to be levelling out to the aforementioned value of 2.5 when the test was terminated, which is still below the suggested guideline of 3.0 as a maximum acceptable gradient ratio. The passing of sediment was observed to continue longer for this test than all the others: from 3 to approximately 16 days, but at a reduced rate.

6.4.4. Terram® T1000 [permeability results: fig. 6.6; gradient ratio results: fig. 6.12]

After a period of approximately 6 days permeability appears to have attained an equilibrium value of 4.0x10^-7 m/s. Gradient ratio also seems to begin to level out between 8 and 13 days at around 2.6. However, a concerning change is noted after 13 days, where both permeability and gradient ratio increase. The increase of gradient ratio to above 3.5, indicative of blocking or clogging of the filter, appears to be contradictory to the observed increase in permeability at the same time. Without more detailed information, only speculation is possible on this phenomenon. However, it can be said with some confidence that this is indicative of instability within the soil/geotextile system. Piping through the system was not unlike the other tests, with only small amounts being observed passing from 1.5 to 6 days.

6.4.5. Terram® T1500 [permeability results: fig. 6.7; gradient ratio results: fig. 6.13]

An improvement in the performance of this system is noticed with respect to gradient ratio, following the increase of hydraulic gradient from 1 to 2 at 25 days. Gradient ratio values of approximately 3.7 were recorded at i = 1, but reduced to a relatively stable level at 3.1 following the increase in hydraulic gradient. Although an improvement, both levels are considered indicative of system clogging or blinding. This is supported by the comparatively low permeability of 4.0x10^-7 m/s, which remains reasonably consistent from
about 11 days on. As with the other tests, a small amount of sediment was noted passing from 1 to 7 days only.

6.4.6. Bidim® A14 with cyclone sand [permeability results: fig. 6.8; gradient ratio results: fig. 6.14]
The results of this test are somewhat contrasting. High gradient ratio values of approximately 3.7 indicate a strong tendency for clogging or blinding, whereas system permeability is medium in comparison to the other tests, at $5.5 \times 10^{-7}$ m/s. A possible explanation for this is the positive influence of the Bidim® A14 assisting the overall system permeability, whilst the clogging may be attributed to the granular filter.

6.4.7. Other Observations

6.4.7.1. Soil shrinkage
As mentioned earlier, backfilling of the dry loess sample resulted in some shrinkage. This was typically of the order of 3 mm that could be observed as a gap between the top of the soil and the top support screen. In most instances, this gap became visible directly after backfilling. However, in some cases the gap only became apparent shortly after test initiation (i.e., first flow through the system). This phenomenon indicates the collapsible nature of loess at very low dry densities. In all cases the gap was accurately measured (to ±0.5 mm), and the calculations adjusted accordingly.

6.4.7.2. Disturbance of soil geotextile interface.
Particularly for the tests on Bidim® A24 and Terram® T1500, disturbance of the soil in direct contact with the geotextile was visible during water backfilling. Following these two tests, the backfilling procedure was adjusted to a slower rate, with the O/CHD (used for backfilling) being held at approximately the level of the geotextile, rather than 25 mm above as suggested in the standard for an initial level. This visibly reduced the disturbance at the interface, but did not eliminate it.

In all cases, varying degrees of disturbance resulted in grain size segregation. As the water rises though the system, the loose structure of the dry, poorly compacted loess, allows grain movement, and formation of preferential flow areas. As water percolates up through
these areas of preferential flow, grains can be seen moving in the flow path. Once flow has reduced, and grains settle, obvious segregation into coarse and fine can be observed in small areas. This segregation will clearly have an influence on the formation of the soil/geotextile filter system, at least locally. Therefore, where very fine grained soils susceptible to collapse at low densities are being tested, backfilling should be significantly more careful.

6.4.7.3. SEM observations (see chapter 2 for structure terminology)
Minimal observations have been made with the scanning electron microscope for this series of tests. This is primarily due to the fact that significant disturbance of the soil/geotextile interface is required to dismantle the test and obtain the samples. Observations of two geotextile samples have been completed and are presented in figures 6.15a&b and 6.16a&b. Corresponding soil samples are not presented due to disturbance during removal from the test apparatus.

Figure 6.15. SEM images of A24 (GR test)
a. Light patches represent localised clogging
b. Enlargement of localised clog site reveals an abundance of alternative flow paths still available

Figure 6.16 SEM images of T1500 (GR test)
a. Blocking over a high percentage of geotextile
b. Enlargement of blocking site reveals limited number of available flow paths.

ERRATUM:
P152, SEM images for figures 6.15a and 6.16a should be swapped.
Geotextile samples taken from the Bidim® A24 and Terram® T1500 tests reveal similar structures to those observed from the stage 1 test observations. Blocking is apparent on the T1500 sample to an even greater extent than observed in the stage 1 sample. Similarly, a greater degree of clogging within the geotextile structure is visible for the A24 sample tested in the gradient ratio test, as compared to the stage 1 test. The increase in these negative aspects of soil/geotextile interaction is suggested to be caused, at least in part, by the loose structure of the soil tested in the gradient ratio test. The loose structure has been observed to collapse on a large scale, and may also be following similar trends at the soil/geotextile interface. This means that collapse of soil structure to a slightly more dense and stable arrangement has resulted in the mobilisation of fine particles. This mobilisation is the suggested cause of the increased clogging and blocking observed on these samples.

This observation suggests that the soil preparation and compaction used in the gradient ratio test represents a worse case scenario. The implication to field situations is that compaction of soil backfills should be carefully monitored. If compaction is inadequate, excessive clogging of filters may result.

It has been noted that the brief period of minor piping observed in all tests corresponds closely to periods of increasing gradient ratio for each test. This is consistent with the suggestion made in section 6.4.7.3, that mobilisation of fine soil particles resulting from soil structure collapse (or perhaps more appropriately, structure readjustment) are responsible for increases in clogging or blocking, and therefore gradient ratio. As gradient ratio approaches equilibrium, observed piping ceases, suggesting that the soil/geotextile system has attained a semi-stable structure. The structure can not be defined as being completely stable, as testing duration is not sufficient to confirm this.

6.5. Discussion of Test Method
Several authors have suggested modifications to the test apparatus in various attempts to increase the versatility, accuracy and repeatability of this test method. One such suggestion has been put forward by Austin et al (1997). They suggest the addition of another manometer port 6mm from the geotextile to provide a more accurate representation of filtration phenomenon at the soil/geotextile interface. A similar modification to the test method is proposed by Fannin et al (1998), with additional manometers being placed at
8mm, 50mm and 88mm from the geotextile. In conjunction with the changes to the apparatus, Fannin et al (1998) suggest the consideration of the “excess water head loss” as an appropriate parameter for better linking design criteria for geotextile and soil permeabilities with performance data obtained from gradient ratio tests. Although both of these approaches may potentially provide more accurate indications of likely field performance, further (perhaps collaborative) calibration and testing is required before these changes will become widely adopted.

Austin et al (1997) also trialed the test method under cyclic changes in hydraulic gradient, with encouraging results. However, as the retaining wall situations being considered in this study are of a unidirectional flow nature, this approach was deemed unnecessary to investigate.

### 6.6. Summary and Comparison to Stage 1 Test.

Table 6.1 below shows a useful summary of the data obtained from both the stage 1 and standard tests. The filters have been ordered from highest to lowest equilibrium permeability. It should be noted that factors other than type of filter alone have influenced these results (especially A34 in the standard test).

<table>
<thead>
<tr>
<th>Filter</th>
<th>Permeability (m/s)</th>
<th>Filter</th>
<th>Permeability (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A44</td>
<td>1.5x10⁻⁶</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>A64</td>
<td>2.39x10⁻⁷</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>A14</td>
<td>1.5x10⁻⁷</td>
<td>A14</td>
<td>7.5x10⁻⁷ (GR=1.9)</td>
</tr>
<tr>
<td>A12</td>
<td>1.1x10⁻⁷</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>A24</td>
<td>6.0x10⁻⁸</td>
<td>A24</td>
<td>6.5x10⁻⁷ (GR=1.7)</td>
</tr>
<tr>
<td>A34</td>
<td>5.7x10⁻⁸</td>
<td>Cyclone sand+A14</td>
<td>5.5x10⁻⁷ (GR=3.7)</td>
</tr>
<tr>
<td>T1500</td>
<td>4.8x10⁻⁸</td>
<td>T1500</td>
<td>4x10⁻⁷ (GR=3.7)</td>
</tr>
<tr>
<td>T1000</td>
<td>2.5x10⁻⁸</td>
<td>T1000</td>
<td>4x10⁻⁷ (GR=3.6⁺)</td>
</tr>
<tr>
<td>7mm premix</td>
<td>2.4x10⁻⁸</td>
<td>A34⁺</td>
<td>1.5x10⁻⁷ (GR=2.5)</td>
</tr>
<tr>
<td>Cyclone sand</td>
<td>1.0x10⁻⁸</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Cap-75 river gravel</td>
<td>FAILURE</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>

Notes: GR = gradient ratio at hydraulic gradient = 1
⁺ At test completion
⁺⁺ Test affected by air bubbles in the system.
As the above table shows, although there may not be direct correlation between test with respect to absolute values of permeability, there does appear to be good agreement at a comparative level. There are obvious differences in the hydraulic characteristics of both tests, with the stage 1 test having free space on the under side of the geotextile, whereas the standard test maintains contact with water throughout. These differences may account for the differences in permeabilities obtained.

Both tests identify Bidim® A14, and A24 as the best performers. Also, both indicate the use of cyclone sand resulting in clogging: stage 1 by it's very low permeability, and the standard test by the unacceptably high gradient ratio. Similarly, Terram® 1000, and 1500 both show signs for concern with lower permeabilities in both tests, and gradient ratios significantly above the 3.0 guideline. Comparisons with Bidim® A34 are unfortunately impaired by the influence of bubbles in that particular standard test.

The implications of this are that the simple stage 1 test does provide meaningful comparisons between filter options. Although it is recognised that further development of this testing method is imperative before it’s use becomes widespread, the potential exists for this test to provide very cheap and simple results for comparative purposes with site-specific soils.

The stage 1 test, in addition to providing permeability data, also provides information on the mass of non-water material passing through the system over time (retention), which is considered to be an important parameter for filtration applications. There are distinct difficulties in obtaining the same information from the gradient ratio test in it’s current form, as there are a number of low points in the system prior to the point of collection (eg bottom section of permeameter, the O/CHD, and potentially in the tubing connecting these two elements of the test apparatus). Another advantage of the stage 1 test over the gradient ratio test is the preservation of soil structures for SEM investigation.

6.7. Conclusions

The testing of six different filtration options in the gradient ratio apparatus has indicated the following:

- Disturbance of the soil at the geotextile interface during backfilling using the standard test procedure results in minor segregation of the soil at the interface that may influence results. Altering the backfilling procedure may reduce this effect. Further investigation is necessary to find the best method of reducing this problem.
Partial collapse of the loess on backfilling is indicative of its collapsible nature at low densities, and suggests a need for better compaction methods. It is suggested that this soil structure collapse or adjustment results in mobilisation of fine particles, which can increase clogging and or blocking processes.

Gradient ratio is observed to increase during the first 2-10 days of the test (approximately) for all tests. Also during this time, minor amounts of sediment are observed to be piping through the filter system. These observations are consistent with the concept of soil mobilisation during this time and progressive establishment of a semi-stable soil/geotextile structure (long term stability to be established by longer duration testing).

Indications of clogging and blocking levels greater than for the stage 1 test from SEM observations suggest the test procedure of the gradient ratio case represents a worse case scenario. This is a direct result of the very low densities obtained by the compaction procedure. The implication to field situations is that wherever recompressed loess is to be filtered, poor compaction will result in an increase filter clogging.

Bidim® A14, 24 and 34 all yield gradient ratios within the 3.0 guideline, and are therefore considered acceptable as filters for the specific soil tested.

Terram® 1000 and Terram® 1500 geotextiles yield gradient ratios in excess of 3.0. This combined with lower permeabilities comparative to the Bidim® range indicates clogging or blinding to a concerning level. SEM observation of the tested T1500 sample indicates blocking to be the primary cause of reduction in permeability and increase in gradient ratio. These factors suggest that use of Terram® T1000 and T1500 is unadvisable with the specific soil type tested.

The granular filter “cyclone sand” also yields unacceptably high values of gradient ratio when used in combination with Bidim® A14. Although system permeability remains within an acceptable range, the tendency toward clogging is considered significant enough to recommend not using this combination for filtering the specific soil tested.

Comparison between results of the standard test and stage 1 test suggest the stage 1 test is a valid method to provide cheap comparative data for filtration performance with site specific soils. The name “standpipe permeameter” is suggested for future reference for the stage 1 test.
Chapter 7

Conclusions
Chapter 7. **Summary and Conclusions**

A study has been done, investigating both theoretical and practical aspects of filtration of Port Hills loess for retaining wall situations. Theoretical aspects have been investigated by the ASTM gradient ratio test, and the stage 1 test (standpipe permeameter test), both of which provide comparative performance information for a range of filter options for filtration of loess. Geotextiles form the majority of the filters tested, and therefore, a brief review of the field of geosynthetics (of which geotextiles are a sub-group) has also been conducted to outline terminology and indicate aspects of design practice with geotextiles. This review has indicated considerations of both a theoretical and practical nature for geotextile use as filters on the Port Hills in retaining wall situations.

A review of four case studies has indicated a number of practical aspects that should be considered in good design practice. Laboratory simulations of common retaining wall designs have provided additional practical information relevant to design of filtration/drainage systems in retaining walls on the Port Hills.

The following sections outline the main findings from each of these avenues of research. The culmination of these findings will be a series of recommendations for good design practice of filtration/drainage systems in retaining wall situations on the Port Hills.

7.1. **Laboratory Testing**

7.1.1. Stage 1 (standpipe permeameter) test

This test places 100mm of loess, recompacted to approximately field density, at the bottom of a 1.2m section of sewer pipe. The particular filter being tested is placed in contact with the loess, so as to hold the loess in place. The system is subjected to a head varying between 0.8 and 1.0 (approximately) above the top level of the loess. The head is periodically raised to simulate rainfall events. The intention of the test is to provide cheap and simple comparisons between filter options with a site-specific soil (in this case, Ahuriri quarry loess). As this is a developmental test, critical evaluation of the test method has also been a focus, as has validation of results by way of comparison to the results obtained from the standard gradient ratio test.

- SEM observations of tested soil samples indicate the formation of bridge networks for the majority of the Bidim range, and the formation of a vault network for A34. Both of these filter networks provide satisfactory filter performance with the specific soil tested.
- There are indications that the formation of bridge networks with the Bidim range, is assisted by small movements in the soil being filtered. This movement allows the
mobilisation of fines, which then pass through the geotextile, and allow remaining coarser particles to bridge. Where soil movement is limited, as in the case of A34, a vault network is formed. Initial observations indicate the possibility of these two filter structures co-existing in some samples, in proportion to the amount of soil movement, but is in need of further study for confirmation.

- Observations of blocking (a specific form of clogging) occurring at the soil/geotextile interface of Terram filters, combined with lower system permeabilities than the Bidim range, tends to indicate the lack of suitability of the Terram range for filtering the specific soil tested.

- The Bidim range of needle punched geotextiles has consistently indicated it's suitability for filtration of the specific soil tested with respect to permeability, retention, and soil/geotextile structure. However, past research has shown a tendency for very thick needle punched geotextiles (such as A44 and A64) to clog after long time periods (400 days) of apparent stability with loess, suggesting the need for longer term testing with these geotextiles.

- The granular filter 7mm premix was shown to form good bridging filter structure, however, system permeability was close to the permeability of Ahuriri quarry loess. This leaves little margin for clogging over time to reduce system permeability before it reaches an unacceptable level.

- The granular filter cyclone sand gave system permeabilities lower than the base level for loess, and is therefore rejected as a filter option with the specific soil type tested.

- River gravel should also be rejected as a primary filter option. This is recommended as a result of two consecutive failures of this filter during water filling.

- Comparison of results given by the stage 1 test to results given by the ASTM D5101-90 test, suggest that the stage 1 test does provide useful comparisons between filter performance with a specific soil type. Although absolute values of permeabilities obtained from both tests are not similar, both indicate similar relative performance of filters tested. Also, both tests have indicated the formation of similar structures at the soil/geotextile interface: the stage 1 test by way of SEM observations of soil and geotextile samples; and the ASTM by way of gradient ratio and limited SEM observation of geotextile samples.
The stage 1 test method is in need of further development, and the following improvements are suggested: inclusion of silicone piping barriers in the soil section of the test; better support and securing methods for filter attachment.

7.1.2. Stage 2 test.

The stage 2 test simulates retaining wall designs at a laboratory scale. Three specific retaining wall geometries commonly used on the Port Hills have been tested as outlined in chapter 5. The test methodology used is still in need of significant development, however, a number of useful observations were made.

- Water tends to flow along the lowest possible path in the drainage system, and was observed leaving the test cells directly underneath the Novaflo® drainage piping in all tests. This strongly suggests a need for impermeable drain channels for the drain pipe to rest on in field situations.

- Surface erosion that resulted from a failure in test “C1”, emphasised the importance of adequate surface drainage in conjunction with good retaining wall design in field settings. Surface erosion may have a number of significant impacts on the effectiveness of a filtration/drainage system. The accumulation of clays on the geotextile was observed after this erosion event. These accumulations would significantly impede the geotextiles performance in a field situation. Although not observed directly in the stage 2 test, another inferred consideration regarding surface drainage is the need for prevention of surface flow prior to placement of the filtration system to avoid the formation of rills. The presence of rills on loess face to be filtered will make obtaining uniform contact with the soil difficult, and reduce the effectiveness of the filter.

- The erosion event in stage 2 test “C1” resulted in erosion of the loess face behind the geotextile, suggesting the backfill material was not providing sufficiently uniform pressure to allow the geotextile to function to it’s potential.

- Observation of the shadowing effects of large grains indicates river gravel provides no filtration in some areas. Also, the observation of sediment passing through this specific system is indicative of either progressive degradation of the filter, or ineffectiveness as a filter. Therefore, the use of river gravel as a primary filter is not recommended.

- Observation of salts originating from the backfill material suggests the possibility of potential corrosion of steel reinforcing where water proofing of porous retaining walls is inadequate (see future work suggestions).
7.1.3. ASTM D 5101-90 Gradient ratio test.
This compatibility test provides information on soil/geotextile system permeability, and indicates clogging potential by way of comparing hydraulic gradient throughout the system expressed as gradient ratio.

- The following values of gradient ratio were obtained at a hydraulic gradient of 1:
  \[ T_{1000} \approx 3.5, \quad T_{1500} \approx 3.7, \quad A_{14} \approx 2.0, \quad A_{24} \approx 1.7, \quad A_{14} + \text{cyclone sand} \approx 3.7. \]  
  A gradient ratio value of 3 or greater is commonly accepted as indicating system clogging or blocking to undesirable levels. These values support the observations made in the stage 1 tests, that the heat bonded Terram range is susceptible to blocking with the specific soil type tested, and that the needle punched Bidim range forms satisfactory filter structures with the specific soil tested.

- Relative permeability values obtained support the findings from the gradient ratio values.

- The test with Bidim A34 was affected by air bubbles in the test cylinder, making the results for this test unreliable.

- Observation of excessive clogging and or blocking in tested samples (compared to observations from stage 1 tests samples) is thought to be a direct result of the very low densities (approximately \( 1429 \text{kg/m}^3 \) dry density) obtained by the test compaction method. This suggests the test method simulates a worse case scenario, and emphasises the need for adequate compaction in field settings where recompacted material is to be filtered.

7.2. Geosynthetics and Design

- There are concerning indications that the approaches to design of “design by chance” and “design by cost and availability” may still be in use on the Port Hills. Both of these design methodologies have been identified as unsatisfactory. The design methodology of “design by experience” is seen as a useful compromise between the potentially time consuming and expensive approaches of “design by function” and “design by specification” and the unsatisfactory options mentioned above. The application of “design by experience” methodology is endorsed only if the experience of the design engineer is extensive, includes a history of design by function case studies, and site investigations for each project are thorough.

- The state of design with geosynthetics is continually improving. Development of testing methods and numerical design criteria is ongoing and in most cases when applied with a
good knowledge of their limitations, good results are obtained. However, there are a number of influencing factors still to be resolved before design with geosynthetics can become a more precise science, as outlined in figure 7.1.

Figure 7.1 Summary of issues associated with designing with geosynthetics.

**General comment**
Standardisation of index and compatibility testing has advanced to a point where most results can be considered fairly accurate, however, many different variations in test apparatus and methodology, and potential sources of error still exist. Until all testing and measurement procedures are standardised, agreement between criteria and compatibility test recommendations is unlikely to be consistent. Care must also be taken to ensure all testing and criteria model expected field conditions to a satisfactory level.

**Index tests**
Provide values for use in criteria (eg $O_0$)

**Criteria**
Calculations to establish required geotextile properties for specific soils

**Recommendation**
Of suitable geotextiles

**Compatibility tests**
Test performance of recommended geotextile with site-specific soil (eg Gradient ratio test)

**Alternative recommendation**
Poor performance of geotextiles from compatibility results may suggest criteria recommendation incorrect

**Modification of Criteria**
Lack of agreement between compatibility test and criteria often lead to modification of existing criteria, or development of new criteria

**Implications**
- Criteria can only provide accurate recommendations if data from index tests is reliable. Therefore, index tests must be fully standardised and statistically significant before work on criteria can yield reliable results.
- Compatibility tests must also be standardised fully and improved to a consistently repeatable level to allow the modification of criteria to proceed without influence from experimental errors from both index and compatibility tests.
7.3. Recommendations for Field Practise

- Laboratory testing has indicated the suitability of the Bidim range of geotextiles for the filtration of the specific soil type tested (see appendix 3.4 for grain size distributions).
- Testing has indicated the Terram range of geotextiles may be susceptible to blocking (a specific form of clogging) which reduces the effectiveness of the filter. At this stage the discontinuation of use of this geotextile range as a filter for the specific soil type tested is recommended, until such time as additional testing indicates otherwise.
- Testing has indicated river gravels should not be used as the primary filter for filtration of loess.
- Other granular filters such as 7mm premix and “cyclone sand” (see appendix A3.4 for grain size distributions) have demonstrated less desirable filtration performance than all geotextile filters, and their use should be restricted to less critical situations, or as a secondary filter.
- Testing has also indicated the need for impermeable drain channels (eg concrete) for drain pipes to rest on in retaining wall situations. This is to avoid water running over excavated loess causing erosion directly under the drain pipe.
- Water proofing of porous retaining walls (eg concrete block walls) is a vital aspect of good design. Laboratory simulations of retaining wall designs indicate condensation remains on the retaining wall for all backfills tested indicates a continual supply of moisture to seep into and through the wall if water proofing is inadequate. This is especially important for backfills containing finer particles, such as river run and premix, where pore water is also held against the retaining wall.
- Field observations have emphasised the need for care during the installation of geotextiles. Poor installation can reduce or even totally negate the properties of the installed geotextile, and may therefore result in subsequent failure of the project. Specific aspects to be aware of are: the need for uniform contact between loess and geotextile; need for continuous coverage of the geotextile over the area to be filtered (ie the geotextile should not be torn and overlaps should be maintained where multiple lengths of geotextile are used).
- Backfill materials used should provide uniform pressure between the geotextile and loess face. Indications from testing are that the commonly used tailings backfill may be too coarse for this purpose.
Elimination of surface flow over loess through surface drainage measures is emphasised, particularly during construction. This is to prevent the unnecessary stress on a retaining wall drainage/filtration system, firstly by additional sediment derived from sheet and rill erosion being washed into the system; secondly to avoid accumulation of clays on the filter, which may significantly affect its performance, and finally to prevent formation of rill channels on the loess slope to be filtered (which inhibits uniform contact between filter and loess).

The practice of designing with limited professional input ("design by chance") should be eliminated wherever possible. Design engineers should base geotextile selection on detailed site investigations (including characterisation of the site-specific soil) and the application of numerical design criteria and compatibility tests (such as the gradient ratio test or stage 1 test once further developed and reviewed) where appropriate.

7.4. Future Work

The stage 1 test (the stand pipe permeameter test) has shown promise as a useful and cheap test for comparison test of filter performance with site-specific soils. Further development of the test method is suggested and encouraged. Possible investigations include: different head options; investigation of viability of adding manometers to system monitoring; use of transparent piping to provide visual information over test duration.

Longer term testing (minimum of 1 year) is indicated for thick needle punched geotextiles at least, to confirm or refute indications from past research of clogging after long periods or stability.

Additional investigation of the structures at the soil/geotextile interface using the scanning electron microscope may help to better understand the modes of formation of bridge and vault networks. Understanding of the processes behind the formation of these positive filter structures may help to design more efficient geotextiles for filtration of loess.

The stage 2 test has the potential to investigate a large range of variables including: filter type and configuration; effect of tunnel gullies on filtration/drainage systems; effect of water back flushing through drainage piping; performance of alternative drainage options; effect of water flow over loess beneath drainage piping, etc. Development of the test method and investigation of these variables is suggested and encouraged.
Observation of salts originating from backfill material, and condensation resting on the retaining wall end of the test cell in all stage 2 tests suggests a potential problem. Without adequate waterproofing, water rich in dissolved salts may seep into the wall and begin to corrode steel reinforcing. Further research is needed to investigate many aspects of this potential problem, including: exact nature and origin of salts; corrosion potential of identified salts; typical amounts of salts present for each backfill material; lifetime of salts in retaining wall situations; methods of removal; and possibly extent of damage in existing systems.

Indications that backfill material may be too coarse requires further investigation to establish the degree of the problem, and possible solutions that may be easily implemented into field practise.

Before the indications of filter performance from this series of tests can be broadly applied to all loess on the Port Hills, further testing is vital with the full range of loess soils, particularly more dispersive samples.

7.5. Final statement
Designing on the Port Hills brings with it a number of potential pit-falls. Of major concern is the very limited amount of testing with geotextiles and this highly variable soil. As demonstrated by a study of the Ahuriri quarry by Jowett (1995), dramatic variations in soil character (especially grain size distribution) can occur over a matter of meters. This emphasises a need for continuing testing of the loess/geotextile system to investigate the potential design problems with the full range of soils of the Port Hills.
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**Tailpiece:**

"Farewell to the land of confusion and frustration"
Reference List.


Bruce, J.G. (1972) Loess soils in the south island. Loess soils and problems of land use on the Downlands of the South Island New Zealand. Otago catchment board publication. No 5, 495p


Carroll, R.G (1983) Geotextile filter criteria. Transportation Research Record, 916


Giroud, J.P. (1994) Quantification of geosynthetic behaviour. Special lecture In Fifth international conference on geotextiles, geomembranes and related products (preprint special lecture and keynote lectures)


Appendices
Appendix 1. Requirements of the Resource Management Act and Building Act

1.1. An Introduction to the Acts
Along with covering a great many other considerations, the Resource Management Act 1991 (RMA) and the Building Act 1991 (BA), are concerned with the suitability of land for residential development. These Acts substantially took over and improved on the Town and Country Planning Act of 1977, and the Local Government Act of 1974. The Acts provide guidelines for local authorities for the investigation and designation of sites as suitable for development or not.

Of relevance to this study are the requirements of the acts regarding natural hazards. In this regard the RMA is mostly applicable to subdivision consents, whilst the BA is concerned with building consents for specific sites. The Acts require that the existence of potential for a number of specified natural hazards (see sections 1.2 and 1.3) must be investigated for the purposes of issuing consents for development and construction. This provides a useful security for the local body, since it may save them from future litigation in the event of any one of the hazards forming into a localised “disaster” (here intended to mean any loss due to one of the hazards becoming reality at one particular site). This is achieved through careful wording of the acts, and provision being made for the development of “marginal land”. The term “marginal land” here is intended to mean land which may be “subject to, or is likely to be subject to” any one of the hazards specified in the particular Act that is relevant to the particular situation.

Both Acts do, however, compel the local body to refuse any consent if the building work itself will accelerate, worsen, or result in any of the listed natural hazards (see below) on that land or any other property. This is where accurate geotechnical assessment of a site becomes vital, since it may be subjective if the building will “accelerate, worsen, or result in” any of the listed natural hazards without detailed investigation. If it can be proven subsequent to a localised disaster that the act of building had caused the disaster, and the consent had been given, then the issuer of the permit may be liable for not identifying the hazard. Such a case becomes complex in both geotechnical interpretations and legal terminology and may result in a lot of wasted time and money, so it is important to get things right first time.
1.2. The Resource Management Act

Unless the issuing authority is “satisfied that sufficient provision has been made or will be made to avoid, remedy, or mitigate the effects...” section 106 of the RMA compels them to refuse consent to a subdivision development in the following situations:

“(a) Any land... or any structure on that land, is or is likely to be subject to material damage by erosion, falling debris, subsidence, slippage, or inundation from any source; or

(b) Any subsequent use that is likely to be made of the land is likely to accelerate, worsen, or result in material damage to the land, other land, or structure, by erosion, falling debris, subsidence, slippage, or inundation from any source.”

Therefore, there are 5 natural hazards to consider: erosion, falling debris, subsidence, slippage, or inundation from any source. The hazard, falling debris, was included in 1993 to the Act by an amendment.

1.3. The Building Act.

In addition to the five hazards mentioned in the RMA, the BA also includes:

avulsion: “The sudden removal of soil from the land of one person, and it’s deposit upon the land of another, by the action of water”

alluvion: “Land that is gained from the sea by the washing up of sand and earth, so as in time to make terra firma”

(Definitions from Butterworth New Zealand Law Dictionary, 4th edition)

If marginal land has been identified, then consent may still be issued, but under a separate section of the BA (s36(2)). The issuing of a consent under s36(2) is conditional on the certificate of land title being annotated by the District Land Registrar to the effect that a hazard exists on the property. Section 36(4) of the BA removes any liability from the issuer of the consent under s36(2) (namely the territorial authorities and their employees or agents) in the event of loss or damage due to one or more of the specified hazards.

1.4. Relevance to Typical Loess Sites

The nature of loess is such that three of these natural hazards are a common problem: erosion (including sheet, rill, and subsurface), subsidence (due to the collapse of subsurface erosion
cavities), and slippage (both small shallow earth failures, and large landslides). All of these hazards may be mitigated against using accurate investigation and careful design of prevention measures.

Erosion is perhaps the major problem with loess on the Port Hills (see Jowett, 1995; Glassey, 1986; Yetton, 1986 for more detail), and there is no single measure that will suit every situation. Designs to reduce the effects of natural erosion often involve many separate elements. Retaining walls can form an integral part of such a remedial design and may be required to satisfy an issuing authority to give consent for subdivision. Surface drainage is also a key factor in reducing the effects of erosion.

In order to reduce the potential for subsidence on the Port Hills, two main courses of action must be taken. Firstly, existing subsurface erosion cavities must be stabilised which could include back filling or excavation (the reader is directed to Yetton, 1986 for more information). Secondly, steps should be taken to reduce the potential for any future subsurface erosion cavities to form. Measures that can be recommended include: adequate surface drainage (to minimise water flow through the soil and therefore the potential for piping); placement of subsurface cut off drains upslope of high risk areas; adequate filtration in retaining walls and cut off drains to prevent initiation of erosion.

Slippage in loess is a complicated issue and in many cases requires long periods of time to design remedial measures. Goldwater (1990) and Buckner (1998) give examples of the complexity of slippage in loess on Banks Peninsula. In rare circumstances, retaining walls may be part of a solution to prevent slippage.

Avulsion is also a potential problem in loess, depending on the strict definition of "sudden". However, in loess, the same processes that cause erosion are most likely to be the same in the case of avulsion, although perhaps on an accelerated time scale. Therefore, remedial measures for erosion processes should also mitigate against avulsion, provided these remedial measures have sufficient additional capacity (or factor of safety) to cope with high intensity events.

Appendix 2: Methodology for Stage 1 Test.

A.2.1. Apparatus

1.2m section of 110mm external diameter (103mm internal diameter) sewer pipe; approximately 2kg of soil sample; 2 litre water collection beaker; filter to be tested; 103mm diameter bung with 1.1m long rod; wall hanging point (securing peg); tap water; water proof tape; three pre-weighed beakers; metal spatula; garden trowel; cling wrap.

In addition for granular filter tests:

110mm diameter sample of 200μm soil support cloth; 110mm diameter section of approximately 5mm support mesh; additional length to sewer pipe as required; small bucket; tap; tubing; approximately 5kg of tailings.

A.2.2. Soil Collection

Material for this series of tests was collected from the same location from the Ahuriri quarry in an attempt to maintain constant properties. A shovel was used to collect the material. The use of this tool may have affected some of the grains, although attempts were made to use the shovel sparingly so as not to damage much of the soil. Large veins of calcite were observed near the collection site, but were excluded from the samples, although smaller veinlets and infilled root casts were allowed to remain in the test samples.

A.2.3. Soil Preparation

In keeping with the simple nature of this test, soil preparation is kept simple. The main aggregates are crushed down to a maximum size of approximately 2mm (average typically <1mm) using a rubber tipped pestle and mortar. Tap water is then added to the crushed material to obtain a thick paste and a moisture content of approximately 17%. Extensive mixing is carried out to ensure the homogeneity of the sample. It is important during this stage of preparation not to add excessive water, which may allow segregation to occur. Therefore, no free water is allowed to be visible at any stage during the mixing of the ‘paste’. Once the soil is sufficiently mixed, it may be placed into one of the pre-weighed beakers and reweighed. From this point the sample must be covered to avoid loss through evaporation. The soil is then placed into the sewer pipe as described below.
A.2.4. Soil Placement and Compaction

After inverting the sewer pipe from the intended test position, the bung is inserted providing a secure base on which to compact the 100mm of loess. Prepared soil is carried carefully from the beaker to the sewer pipe, ensuring no loss of soil or moisture, as this will disrupt density calculations. A cleaned garden trowel (or similar tool) is used to carry the soil from the prepared sample container to the tube. The soil, at no time, may be allowed to fall or run more than 25mm to reduce the possibility of segregation.

Placement is performed in a series of levels each of approximately 25mm in thickness. After the initial placement of each level the base of an 11mm diameter cylindrical tool, in this case a 12cm length of wooden dowelling rod, is used to compact the loess. Compaction is primarily done in order to reduce the amount of void space, and thereby reduce any later segregation due to movement of air bubbles. Compaction consists of ‘poking’ the loess with the tool in points around the circumference of the tube, and then moving into the centre in a concentric manner. Additional pokes are made where it is obvious that the loess has not settled. This process continues until the surface is approximately even, at which point the next level may be placed (the number of ‘pokes’ is typically of the order of 60 per layer).

Once the final level has been placed and compacted, levelling is conducted using a simple flat surface (eg a metal spatula). Wherever gaps are obvious, small amounts of loess are added, and levelling continued. The soil remaining in the beaker should be reweighed. Directly following this, a representative sub-sample should be taken from the beaker and placed in the second pre-weighed beaker for the purpose of moisture content calculations (see appendix A.8). A final representative sub-sample must be taken from the beaker for grain size analysis. Once the surface is level, and the required sub-samples have been taken from the first beaker, the geotextile can be placed.

Where a granular filter is to be tested, the above procedure is also followed. The only addition is that provision must be made for the granular material to be placed after the loess. This can be achieved by withdrawing the bung the required distance, subsequent to soil levelling.
A.2.5. Placement of Filter
A.2.5.1. Geotextile
A sample of geotextile must be selected approximately 30x30cm square. The middle of the geotextile is placed directly onto the levelled soil. The fringes of the geotextile are then secured around the pipe by means of water proof-packing tape. Care must be taken not to tension the geotextile against the outer edge of the tube in order to avoid alteration of the geotextile structure and any potential damage.

A.2.5.2. Granular Filters
The amount of space that should be left for the granular filter is dependant on the maximum grain size of that granular filter. Therefore, the larger the maximum grain size, the larger the space required (a suggested minimum is five times the maximum grain size diameter). Once the soil has been levelled, the bung should be withdrawn the required amount for placement of the granular filter. Where placement is done in a series of levels, the bung should also be withdrawn progressively. Once the bung is withdrawn the required amount, the granular filter may be placed, with emphasis on avoiding segregation of grain sizes.

Once the filter is placed, the granular filters require additional support to avoid loss over the course of the experiment; this is in the form of a 110mm diameter, 200μm opening size soil support cloth (placed directly over the filter), followed by a 110mm diameter section of approximately 5mm support mesh. Both of these are glued securely to the end of the sewer pipe using epoxy resin.

A.2.6. Placing in Test Position
Once the soil and filter are secure, the pipe may be inverted and the bung removed. Care must be taken whilst removing the bung to avoid the soil being dragged up the pipe. Material will be caught on the bung. This material must be scraped into the final pre-weighed beaker and placed in the oven. The dry weight should be subtracted for density calculations as the moisture content of this small sample will be higher than the sample as a whole.

The sewer pipe may then be placed on its securing peg in the test position. For granular tests the glue and mesh system may not be assumed to be strong enough to hold the test entirely, therefore a bucket almost full of tailings is used as additional support. The
bucket must be placed directly under the pipe and must have a drainage tap at the bottom, with a slight incline towards that drain. The end of the sewer pipe rests directly on the tailings for support, while the material passing through the system is allowed to pass through the tailings, out the tap, and down a tube to a collection beaker. In all cases, the collection beaker should be placed prior to commencement of the water filling process. Cling wrap needs to be placed around the collection beaker and sewer pipe to prevent losses by evaporation. For granular tests, cling wrap is also necessary around the tailings bucket. Cling wrap must also be placed on the top end of the pipe after filling in all cases.

A.2.7. Water Filling Procedure

Once placed on the securing peg, each experiment is filled with water in a careful process. The initial stages of the filling are most important, and therefore time must not be a concern (total filling time for one set up should be approximately 1 hour). Disturbance of the loess could result in the formation of a clay layer on the surface, which would then dominate the hydraulic properties of the entire system. Initial filling must therefore be very slow, using a tube (1.5m long, in order to sit directly on the top of the loess) with only small perforations at the loess end (5 perforations at 0.5 mm diameter). This pipe is used to fill the first 3 litres (approximately) with the first litre taking approximately 20 minutes to fill. Subsequently a pipe (1.35m long) with larger holes (8 at 1.0 mm diameter) may be used at increasingly faster rates. For both of these pipes, the holes were created by placing a water proof tape over the end, and then puncturing the tape with a spike. As the rate of water filling is increased, the tube should be withdrawn further out of the tube, or more importantly, away from the water/soil interface. Finally, a funnel is used for the last 2 litres.

A.2.8. Measurements and Sampling.

Sampling is continuous, with a collection beaker always present. Permeability of the system may be established in two ways. Firstly by measuring the amount of water collected in the beaker over a known time. Or secondly by measuring the change in water level from the top of the pipe, as water leaves the system. For this series of tests, both measurements were taken, but calculations have been based on the change in water level. This is primarily as a result of the granular tests not necessarily allowing all the material passed to reach the collection beaker. Records made typically include time, mass of material passed (including
water), appearance of material passed (eg silty, clear, etc) current water level, general appearance of system.

Assessment of amount of material (other than water) passing can be made simply by placing the total material passed (including water) into a pre-weighed beaker, and evaporating off the liquid. Reweighing after drying will provide total mass passed over time data. Observations will supplement this by describing the nature of material passed. If time is available, more in depth analysis of the material passing through the system may be possible, for example the method outlined in appendix A.4.

Significant changes in temperature should also be noted as this affects permeability. Other notes should be made on general appearance of the filter system, noting and timing any obvious changes that may help explain changes in system performance.

A.2.9. Test Maintenance.

Water refills of the test should be made as required. Typical variations in water head in this series of tests were 20cm (from approximately 1m to 0.8m head above top soil level). Bumping of the test should be avoided, as this may disrupt the developing filter system. However, the effects of disturbance may be a useful parameter to investigate in subsequent tests.

A.2.10. Test Completion.

Once the test is complete, the water may be drained naturally (as happens throughout the test). If more rapid drainage is required without significantly impacting the soil/geotextile interface a long tube may be lowered into the pipe to a level just above the soil. Sucking on the other end at a level below the sewer pipe will provide a head difference that will remove most of the water from the sewer pipe via the tube (so have either a bucket or large jug ready). Allow the remaining water in the test to evaporate off (removing cling wrap from around the system will speed this process).

A.2.11. Removal of soil from the sewer pipe.

Once the soil has dried, the test may be taken from its securing peg. The tape holding the geotextile to the pipe is then removed or at least cut. It may be necessary to cut the geotextile along two vertical lines almost to the level of the bottom of the pipe in order to
reduce resistance for the next step. The bung is then re-inserted into the pipe from the top and is used to extrude the sample as gently as possible. It is important not to touch the geotextile at the end, as samples may be taken from here for investigation with the scanning electron microscope (SEM), which requires that no grease (even from fingers) come into contact with samples. In most cases the geotextile is easily removed from the dry, extracted soil where samples may be cut.

The resulting dry soil cylinder may be bisected lengthways by means of placing a thin straight edge (eg strong knife) along the top section of the length, a hammer is then used to tap the other edge of the knife. The result should be two halves with clean broken surfaces close to the interface. The required sample is then identified and carefully cut out using a craft knife in a scraping manner around the sample. The surfaces that are scraped should not be investigated, as they will be significantly altered. These altered surfaces should be used when handling the obtained sample with tweezers during the mounting of the sample onto an SEM stub.


<table>
<thead>
<tr>
<th>Time</th>
<th>Water level</th>
<th>Initial beaker weight</th>
<th>Beaker plus collected material</th>
<th>Beaker plus remaining solids</th>
<th>Temperature</th>
<th>Appearance notes</th>
</tr>
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<tbody>
<tr>
<td></td>
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Appendix 3. Grain size analysis.

A.3.1. Introduction.

The methods of grain size analysis used throughout this project are as described by Lewis and McConchie (1994). Both sieve and pipette analyses have been used, and therefore the procedures as presented by Lewis and McConchie (1994) are presented in section A.3.2 and A.3.3 Results of analyses on all tested samples are given in section A.3.4. For all pipette analyses, the deflocculant used was sodium hexametaphosphate (Calgon), at the rate of 1g/litre.

For visual reference to Grain Size Distributions of samples used in testing procedures, please refer to the following graphs included with this appendix.

Granular Filters

- Figure I: Cap 75 River Gravel
- Figure ii: Cyclone Sand
- Figure iii: 7mm Premix

Standard Test Procedure

- Figure iv: Bidim® A14
- Figure v: Bidim® A24
- Figure vi: Bidim® A34
- Figure vii: Cyclone Sand and Bidim® A34
- Figure viii: Terram® T1000
- Figure ix: Terram® T1500

Stage One Test Procedure

- Figure x: Bidim® A12
- Figure xi: Bidim® A14
- Figure xii: Bidim® A24
- Figure xiii: Bidim® A34
- Figure xiv: Bidim® A44
- Figure xv: Bidim® A64
- Figure xvi: Terram® T1000
- Figure xvii: Terram® T1500
- Figure xviii: 7mm Premix
- Figure xix: Cyclone Sand

Stage Two Test Procedure

- Figure xx: Bidim® A24 with Tailings Backfill
- Figure xxi: River Gravel Filter and Backfill
- Figure xxii: Bidim® A14 and 7mm Premix with Tailings Backfill
Appendix 3.2. Sieve analysis method (Lewis and McConchie, 1994)

Preparation

1. Disaggregate thoroughly, remove salts and organic matter (see Chapter 5). Select a representative subsample and label a data sheet for it (e.g., Fig. 7-3). Weigh to 0.001 g.

2. (a) For samples with less than about 10% mud and when analysis of the mud is not necessary, dry the subsample at no more than 65°C (to avoid baking clays). Leave to cool and equilibrate with the atmosphere for at least 1 hr before weighing. Then thoroughly disaggregate the sample—for most loose sands, a rubber bung on a piece of glazed paper is adequate.

   (b) For samples with a mud fraction to be analyzed, wet-sieving is necessary (see Table 7-3, steps 3 and 4). It is wise to perform a wet-sieving operation on two subsamples—for one, dry both fractions and determine the proportion of mud to sand, whereas for the second only the sand fraction is dried and weighed and the wet mud fraction used for pipette or hydrometer analysis. After wet-sieving, dry the coarse fraction and weigh.

Analysis

3. Select a nest of sieves to cover the grain size range of the sample. If the sample has been wet-sieved, the finest sieve should be 4μm; otherwise sieves as fine as 4.75μm may be used. For detailed work and where polymodal distributions are present, use 0.25μm intervals.

4. Clean the sieves before using them: invert each sieve and tap it gently onto a flat surface or, using your hand, rap the side diagonally to the mesh to knock out any loose grains. Then brush the screen, again diagonally to the mesh, with a soft sieve brush. If any grains are trapped in the mesh, do not attempt to force them out—leave them there (or distortion of the mesh may result). Stack the sieves in order, with the pan at the bottom. If two nests are necessary, use the coarser set first, then transfer the contents of the pan to the finer stack (with another pan under it!).

5. Pour the sample into the top sieve and add the cover (the greatest load on a sieve should not exceed 5 grain-diameter thickness; otherwise mass-trapping effects or mesh distortion will occur). Secure the sieve nest firmly in the sieve shaker. Shake for a standardized time—usually 10 or 15 min.

6. After shaking, invert and clean each sieve as in step 4; retain each fraction on a large sheet of glazed paper, and transfer each to a labeled, preweighed beaker or envelope. If the sample has previously been wet-sieved and mud analysis is to follow, add sediment passing the 4μm sieve (pan fraction) to the mud fraction.

7. Weigh the beakers (or envelopes). Retain each fraction in a labeled envelope for future use.

8. Check each fraction for grain aggregates and other properties (e.g., compositional differences, shape properties) with hand lens or under a binocular microscope (there may be significant differences between fractions). If aggregates are common, either disaggregate and resieve, or carefully estimate the percentage of aggregates in each fraction and subtract this percentage of the weight of the fraction from both the weight of the fraction and the total weight of the subsample.

9. Compute the weight percentage of each fraction, then compute cumulative percentages. The weight percent of each sand fraction is:

   \[ \frac{\text{weight of sand on sieve}}{\text{total sample weight (sand plus mud)}} \times 100 \]

   Add these percentages incrementally to obtain cumulative weight percentages.

10. Plot the data on a histogram (if desired) and as a cumulative curve on graph paper (e.g., Figs. 7-4-7-7). Consistent "kicks" at the same size grade in cumulative curves for different samples may indicate a defective sieve.
Appendix 3.3. Pipette analysis method (Lewis and McConchie, 1994)

**Preparation**

1. Obtain a representative subsample that will yield no more than 15–20 g of mud.

2. Fully disaggregate the subsample (see Chapter 5). It may be adequate to cover the sample with a little distilled water plus dispersant (keep track of dispersant added) in a beaker and to use fingers in a rubber glove to break up the sample fully (rinse mud off glove back into the beaker). Alternatively, standardize on time with an ultrasonic device.

3. Wet-sieve the sample with a reserved-for-the-purpose 60-μm sieve.

4. Transfer all sand to a large basin and dry-sieve it to cover break up the sample fully (rinse mud off glove back into the beaker). Alternatively, standardize on time with an ultrasonic device.

5. Transfer all the mud collected in the sieve to a 1-L measuring cylinder via a large funnel (label each cylinder).

6. Add 20 mL of prepared dispersant solution to the column if you have not previously used a solution with dispersant in your wash bottle or for disaggregation (see Chapter 5, "Dispersion of Clays"). Between about 0.5 and 1 g of sodium hexametaphosphate ("Calgon") is normally sufficient to prevent flocculation of clays, but this compound may dissolve fine carbonate grains such as foraminifers and may interfere with later X-ray analysis of clays. This is especially critical since this may appear to cover the column with a watchglass and let it stand overnight to check for further wet-sieving; wash finally with distilled water for several minutes may assist dispersion.

**Analysis**

Begin pipette analysis early in the morning, because the time between first and last withdrawals is at least 8 hours.

1. Before beginning, check that no columns have flocculated. Flocculation can be recognized by a curdled and rapid settling of clumps of particles, or by the presence of a thick, mucous layer on the bottom of the cylinder that passes abruptly into relatively clear water above. If flocculation is evident, try adding more dispersant solution or make up a new suspension with a smaller amount of sample. Using a mechanical stirrer for 5 minutes may assist dispersion.

2. Take the temperature of the water in the beaker of tap water and look up the correct depths in Table 7-2. Note these depths on the pipette schedule, and monitor any temperature changes during the analysis (or ensure constant temperature by air conditioning). Viscosity changes with temperature and settling velocities will change significantly if there is variance.

3. Select a 20-mL pipette (one that empties quickly) with depth graduations. Connect a rubber pipette filler and check that the suction works efficiently. Have a large beaker of distilled water ready on the bench for rinsing.

4. Start the timer: 1 min before the initial withdrawal (if using an electronic timer, set it at 1:59:59). Immediately begin stirring column 1 using a brass stirrer. Start with short, quick strokes at the bottom and air up all the settled mud, then work up the column with long, vigorous strokes, being careful not to mix air in with the suspension. Precisely at time zero (12:00:00 on the electronic timepiece), withdraw the stirrer. Lower the pipette to 20 cm. At exactly 20 sec, extract a 20-mL sample. Empty it into the respective 50-mL beaker and then_fast the pipette into the same beaker after sucking up 20 mL distilled water (also wash outer part with distilled water from the wash bottles).

This first withdrawal is particularly critical since it represents everything finer than 49 (that is, total mud). Insertion of the pipette for subsequent withdrawals should be made with much more care to avoid creating turbulence.

5. The next withdrawal is for the fraction finer than 4.5 μm. At exactly 2 min, withdraw 20 mL empty into the next beaker, and rinse as before.

6. Repeat the procedure for all subsequent withdrawals. Efficiency is essential, particularly where multiple samples are to be analyzed. Initially, a withdrawal must be made and the next column stirred within 1 min. Withdrawal and stirring need to be completed in 30 sec, leaving 30 sec for stirring the next column. (To ensure thorough stirring of every column, carry out a preliminary stir in each one during an earlier spare moment.)

If withdrawal must be made at the wrong depth or time, make a note of the error and use Fig. 7-11 to find the grain size represented.

When there are long periods between withdrawals, cover each column with a watch glass. Any external source of vibration must be eliminated during the analysis.

1. When all withdrawals are completed, put beakers onto trays and allow dry them: it may take up to 48 to evaporate all the water. If further analysis of the clays is to follow, do not heat above 65°C.

15. Remove dry beakers from the oven and leave them to equilibrate with the atmosphere for at least 1 hr. Weigh to 0.001 g; record on data sheet.

16. Calculate cumulative weight percentages:

(a) Subtract beaker weights from beaker + sediment weights to get sediment weights.

(b) Multiply the weight of sediment from the 45 sample by 50 and subtract the weight of dispersant in the column. This gives the total weight of mud, e.g., 0.405 g (45 sediment weight) * 50 – 1 g (wt. of Calgon in the procedure suggested) = 19.25 g (weight of mud, D).

This value, added to the weight of the sand fraction (S) determined from step 4, provides total sample weight. To test for experimental error, either (1) measure total sample dry weight initially (however, even low-temperature drying may cause problems in subsequent dispersion of the clay fraction); or (2) dry and weigh the suspension remaining in the cylinder after full analysis. If error has crept in to the above calculations, correct as necessary.

(c) Add the sand percentages cumulatively to obtain their cumulative percentages (step 9 of Table 7-1).

(d) Remember that each pipette sample represents material in the column finer than a certain grain size. To obtain cumulative percentages for mud intervals, multiply each mud weight by 50, subtract the weight of dispersant, divide by the total sample weight, and subtract from 100:

\[
\text{cum. } \% = (100 - (50 x (pipette sample wt.3) - 1) (assuming 1/5 dispersant)) \times S / F
\]

A computer program can be constructed easily in standard spreadsheet software packages to process the raw data (all cells other than those for data entry should be "locked"; see also Slat and Press 1976; Coates and Frater 1985).

17. Plot results on graph paper as required (see Figs. 7-4–7-7) and proceed to graphical statistical analysis, or process by Method of Moments.
Appendix 3.4. Grain Size Distributions.
Figure iv

Bidim A14 (Standard)

Cumulative Weight (%) vs. Particle Size (mm)

Figure v

Bidim A24 (Standard)

Cumulative Weight (%) vs. Particle Size (mm)

Figure vi

Bidim A34 (Standard)

Cumulative Weight (%) vs. Particle Size (mm)

Figure vii

Cyclone Sand & Bidim A14 (Standard)

Cumulative Weight (%) vs. Particle Size (mm)
Figure xii

Figure xiii

Figure xiv

Figure xv
Bidim A24 with Tailings Backfill (Stage 2)

River Gravel Filter & Backfill (Stage 2)

Bidim A14 & 7mm Pre-mix with Tailings Backfill (Stage 2)

Figure xx

Figure xxi

Figure xxii
Appendix 4 Filter Paper Grain Size Analysis: Methodology and Reasoning.

A.4.1. Introduction

A.4.1.1. Need for, and Development of the Procedure

The problem of describing the sediment that passed through each geotextile was one that could not be dealt with by conventional techniques, although several such options were considered. For example, pipette analysis as outlined in Lewis and McConchie (Analytical Sedimentology, 1994) was considered as an option. However, the amount of sediment collected in each sample was considerably less than the 10-20 grams suggested as a minimum for this form of analysis. The same can be said for hydrometer analysis. Automatic settling tubes were also considered, as was light transmission, but the availability of the necessary equipment was a problem.

Another grain size analysis option was to take a sample from the collected material and mount it on a slide for viewing through transmitted light microscopes. Once in this form, point counting could be used for grain size distribution, and mineralogy of the coarser grains could also be determined. The main problem of this option was that of obtaining a truly representative sample, since the sample would only be around 1 millilitre. Therefore this option was relegated to a secondary possibility for investigation of the coarser fraction.

A variation on this idea was to dry the sample completely, and then mount all the sediment on the slide. This option was discarded because of obvious problems in moving the sediment in dried form from the beaker to the slide without altering the grains, and without losing significant quantities of sediment.

The idea of doing a form of grain size analysis, using filter paper, came originally from an effort to remove dry grains from the beaker to a useable form for microscope work, without damaging grain size. From this, a suggestion from Cathy Knight lead to the procedure outlined below, being developed.

A.4.1.2. Basic Description of Test

Three grades of quantitative Whatman filter paper were folded, and then dried for a minimum period of 2 hours at a temperature of 50°C (in order to remove excess volatile material in the paper), and were then weighed to an accuracy of 0.0001g 2hr after removal from the oven. From this point, the papers were arranged in a series of funnels, one under the other in order of grade - in similar fashion to sieves used in conventional analyses.
Under the final funnel a beaker was placed (pre-weighed to an accuracy of 0.001g) in order to catch the material smaller than the finest grade paper, and the residual water of course.

The papers used were: 40, 42 and 43. These were chosen because of their quantitative nature, and the low level of acid treatment involved in their production in comparison to the stronger 50 and 540 series papers. Acid levels were considered important due to the presence of calcite, and the potential for alteration of clay minerals which may have biased any later analyses.

A.4.2. Methodology

A.4.2.1. Paper Preparation

Papers of each of the four grades are taken from their boxes, ensuring clean conditions and hands. The side facing up in the box should remain up throughout the test preparation and run (that is, sample water will be poured onto this surface). Fingers are lightly run around the edge of the paper to remove loose paper from the edge that may fall off between weigh ins and influence results significantly. The grade of the paper and the sample number should be written clearly on the “up side” to reduce confusion where multiple samples are tested. The writing implement has not been shown to matter, so long as it is used consistently.

One full set of three papers should be prepared in addition to the test samples, and be labelled “control”. This set should undergo all the same processes as the test samples, including running control water samples (and whatever additives, such as deflocculant are used). All papers must be folded prior to placement into a 50°C oven, where they are held for a minimum of 2 hours. Once removed, the samples should then be placed in a stable environment near the balance to be used, and allowed to equilibrate for 2 hours prior to weighing to 0.0001g accuracy.

A.4.2.2. Sample Preparation

Test samples should be tested as soon after collection as possible to reduce flocculation, and/or biological influences. Where flocculation is apparent, deflocculant such as sodium hexametaphosphate ("calgon") may be used, provided amounts are recorded and are consistent between samples (including control). To reduce the potential problems of clogging, the beakers containing the samples are stirred and then allowed to settle for 2 hours prior to the running of the test, allowing the coarser material to settle. The first additions of
the test solution added to the set up were intended (as a result of the settling) to pass directly through the first paper, and mostly through the second also, with the third beginning to capture a significant amount. Subsequently, a higher percentage would be caught on the upper papers as the coarser material was added.

A.4.2.3. Test Running Procedure

Once weighed, the filter papers may be placed into the thoroughly cleaned funnels. Grade 43 goes into the top funnel, 40 in the middle and 42 in the bottom funnel. The preweighed collection beaker is also placed directly under the grade 42 funnel. After the required 2 hour settling period, approximately 50mls of the test solution may be poured onto the surface of the grade 43 filter paper. Pouring should be gentle so as not to damage the paper, and consistent for all tests. Depending on the expected nature of the test material, a regime of agitation and settling may be used to reduce clogging effects. Therefore, after the initial pour, the test sample may be allowed to settle again for an appropriate period of time so that the majority of the fine material may pass through the open pore sizes of the first filter, before the larger grains are introduced, which may create clogging effects. The following is a suggested procedure (and the one used to obtain the sample results given in section A.4.4):

<table>
<thead>
<tr>
<th>Stir</th>
<th>Settling time</th>
<th>Amount poured.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stir</td>
<td>2 hours</td>
<td>50mls</td>
</tr>
<tr>
<td></td>
<td>30 minutes</td>
<td>50mls</td>
</tr>
<tr>
<td></td>
<td>Repeat</td>
<td>Until 50ml remains</td>
</tr>
<tr>
<td>stir</td>
<td>15 minutes</td>
<td>25mls</td>
</tr>
<tr>
<td>stir</td>
<td>5 minutes</td>
<td>10mls</td>
</tr>
<tr>
<td>stir</td>
<td>no delay</td>
<td>15mls</td>
</tr>
</tbody>
</table>

The final pour should be accompanied by the washing out of the test beaker with distilled water (material should be washed into funnel). It may be necessary to alter these times, and further investigations will very probably suggest more suitable timetable for pouring and settling.

A.4.2.4. Disassembly Procedure

The test will need some time before disassembly can begin. The 43 paper must be dry enough to be removed from it’s funnel without damaging the paper. The paper should be
rested on a clean surface and covered. The funnel should immediately be washed through with distilled water. This will dampen the subsequent papers, but will wash through any material caught in the funnel. This routine of paper drying, removal and funnel washing should be repeated for all papers to ensure all material is either collected on the papers, or in the collection beaker.

A.4.2.5. Reweighing Papers and Beakers
Papers should be re-dried in the same oven for the same time. Equilibrium time should also be allowed as for preparation. Notes may be taken during reweighing to indicate the level of visible sediment on each paper, and therefore suggest the level of crystalline material not visible. Evaporation of collected water may take some days and decisions must be made as to whether or not material collected in those beakers will be used for subsequent investigation. If the samples are required, temperatures over 65°C are likely to damage clay particles, and should not be used.

A.4.3. Analysis
The weights taken after testing are subtracted from weights before testing. These values must be adjusted by subtracting or adding the difference for each fraction recorded by the control set. The grade 42 filter paper is stated to retain particles to a size of 0.0025mm, 40 retains 0.008 and 43 retains 0.016. 98% of material of the stated size and larger will be retained, however, this level of retention is quoted customarily to allow for secondary filter effects (Whatman product guide, 1996) (ie minor clogging). Investigation of the largest grain size retained on the 43 grade paper may be useful for plotting information obtained. Once values are established, graphs may be plotted as for traditional sieve analysis with the beaker fraction being equivalent to a pan fraction and so on.

A.4.4 Sample results
The following results are from samples taken over the duration of one of the stage 1 tests (Terram 1000 test).
A.4.5. Comment/Conclusions/Future Work

The results shown above show a good correlation with observations. That is, as the observed amount of sediment increases over time, the curve is lowered, indicating a higher percentage of the slightly coarser fractions. Unfortunately, the results are dominated in this case by a very high percentage of suspended crystalline material (calcite), and prevent any firm conclusions being made for these samples.

This technique has been only briefly developed here as a secondary tool, and although the results can not be taken to be as accurate as conventional methods, it does show promise. Potential certainly exists for this technique to be investigated further, and compared with other available options, and perhaps using other filter media (e.g. the 50 and 540 series, with investigations as to the effects on clays, also, the glass fibre filters appear to be a possibility for more detailed investigations within the clay size range).
Appendix 5. Stage 2 Test Methodology

An abbreviated photographic record of the set up procedure for all three stage 2 tests is given in figures A.5.ii, iii, and iv towards the end of this appendix.

A.5.1. Apparatus

Reinforced 6mm thick acrylic tank measuring 50x50x80cm (approximate external measurements); approximately 300kg loess sample; water feeder system (eg geotextile covered, perforated sewer pipe or Megaflo®); constant head system; filter and/or backfill materials to be tested; drainage system to be tested (eg Novaflo® and impermeable drain cradle); silicone sealant; large mixing tubs; manometers; 100µm mesh; tubing to fit manometers; strong table (to support up to 450kg); material suitable for a drain chute (eg cut sewer pipe).

A.5.2. Soil Collection

Soil was collected from Ahuriri quarry, courtesy of Fulton Hogan. Samples were collected on a test by test basis. For each individual test, samples were obtained from one place. However, since Ahuriri is an active quarry it was impracticable to collect samples for all tests from the one location. Collection did not follow any standard, however, weathered surface material was avoided in all cases and material was otherwise collected “as it came” (ie bias was avoided for or against any variation of material). Where possible only a shovel was used to load the material into sample bags, however, in some cases geological hammers were used to dislodge the material from the loess face.

A.5.3. Tank set up

Prior to the placement of prepared soil, a number of preparations must be made to the tank. A number of these preparations may vary depending on the specific nature of the individual test. For example, in all of the tests in this series, an impermeable drain cradle made of epoxy resin was placed at the “retaining wall end” of the tank prior to any soil placement. The drain cradle is held in place with silicone sealant and is shaped to provide flow toward one of the side walls. In order for the water to drain from the system a hole must be cut in this side wall at the floor level. Other test configurations may not use this drain cradle.

Manometers have also been used in this series of tests, but may be deemed unnecessary by other testers. Manometers are positioned along the drain side wall at a height of 100mm and at 10, 15, 30, 50 and 70cm from the retaining wall end. Each of these
manometers must have a soil protection mesh placed on the inside of the tank over these openings to reduce loss of soil from the system. The mesh used has 100μm openings, and may be secured with silicone sealant.

Piping barriers are seen as a prudent measure to reduce uncontrollable piping along the walls or floor of the tank. The piping barriers are continuous lines of silicone sealant running along the full width of the floor and height of the walls in at least three positions along the tank. The first barrier is placed some 10cm from the proposed loess face, and may be made to conform to the proposed slope profile. The remaining barriers are placed upgradient of the manometers. An optional additional barrier may be placed directly down gradient of the water feeder system.

Depending on the specific test, it may also be necessary to place the water feeder system prior to soil placement, or soon after placement of initial layers.

A.5.4. Soil Preparation
Soil collected from the field is disaggregated by means of, as gently as possible, tamping with sledge and geological hammers in plastic tubs. This method is relatively effective for disaggregating the large amounts of soil required without significant damage to individual grains, or alteration of grain size distribution (ie loss of fines), provided the soil from the field is not overly dry. During the disaggregation process in this series of tests, a very small number of calcitic cemented concretions were found and removed (as they would disrupt the compaction significantly).

Having been tested for moisture content (see appendix A.8), the bag lots (19-33.5kg, 23kg average), once disaggregated, are mixed thoroughly with the appropriate amount of water required to obtain approximately 10% moisture content. Once mixing is complete (approximately uniform moisture and consistency), the soil may be added to the tank in levels of no more than 25kg.

A.5.5. Soil Placement and Compaction
A.5.5.a. General Procedure
For the purposes of these tests, it is necessary to simulate a cut slope in loess. To achieve this, soil is placed and compacted in layers of 5-7cm (after compaction) behind a temporary support wall. The wall is constructed of tongue and groove timber, and is progressively built up as soil depth increases. Bracing is necessary to prevent the compaction procedure from
compromising the support wall; bracing takes the form of support struts placed between the front wall of the tank and the temporary support wall.

Soil is placed behind the support wall on the base of the tank, or whatever current layer is exposed, and not allowed to fall, thereby reducing potential for segregation. Placement should be done such that an even layer of soil of approximately 10-12cm depth is obtained for compaction.

For the majority of the procedure, a standard 2.5kg compaction tool (as defined in the NZS 4402 1981 – test 14) is used (see figure A.5.i). The entire area of the prepared soil must be compressed by the hammer. This is done in three passes, the first being with little overlap of blow areas. For this first pass, the compactor piston is raised to its full height and the collar held approximately at the soil surface, the piston then being released. About 150 blows are used on the first pass. For those areas where the standard compactor is unable to reach effectively (e.g. tank walls, temporary support wall and around water feeder system), a straight edged geological or sledge hammer is needed to compress the soil in those areas to the same consistency as that compacted by the standard tool.

Figure A5.i. NZS 4402 1981 compaction hammer used for compacting loess in stage 2 test.
The same method is used for a second pass of around 200 blows, and also for a third and final pass of around 250 blows, with each successive pass having greater overlap of compaction piston placement. A second compression in the difficult areas is required at the end of the third pass. The overall result should have taken about 600 blows and leave a consistent layer about 5cm thick. If there is need for a long time break between placement of two layers (eg overnight), then a light spraying of water over the compacted layer shortly after compaction, covering overnight, and a second light application of water in the morning may be advisable to avoid excessive cracking.

A.5.5.b. Forming an artificial tunnel gully.

The compaction process must be adjusted when a developing tunnel gully is to be simulated. The level at which the tunnel is required must be compacted, and then an appropriate shape carved out of the surface. For this test, a hollow aluminium pipe approximately 2cm in diameter was placed into a rounded carved channel. The subsequent layers are compacted as normal, with particular care over the pipe to not disrupt it from its position. On completion of compacting all layers, the aluminium pipe may be withdrawn. As significant pressures are exerted on the pipe, it is recommended to drill a small hole in the end of the pipe at the face end in order to place a small rod for additional leverage. The pipe may only be removed in sections (due to the end wall), and must be hack sawn off (see fig A.5.iv.b.). As this method did not give the intended result, two potential solutions to this problem are suggested:

1) Cleaning the sides of the tunnel, by way of scraping with a piece of wire, subsequent to the removal of the pipe.

2) Cutting an additional narrow channel directly underneath the aluminium pipe prior to placement of subsequent soil layers.

A.5.6. Placement of the Filter

A.5.6.1. General

Directly following compaction, the temporary support wall may be lowered. The resultant face is invariably affected by its contact with the wooden support. Therefore, a couple of millimetres are scraped off this face to produce the face onto which the filter will be placed.
Placement of the filter is very much dependent on the specific geometry. In each case the filter is placed directly against the cut loess face with emphasis on attempting to obtain maximum contact between the loess face and the filter.

A.5.6.2 Geotextiles
Where geotextiles are used as filters, the option to wrap the filter around the drainage medium is available. If this option is taken, the drainage medium (Novaflo®) is placed at the same time as the filter. The geotextile is placed onto the loess face as evenly as possible. The installation can not be completed without the placement of the backfill, which holds the geotextile in place. The backfill procedure is described in section A.5.7.

A.5.6.3. Granular filters
In the case of granular filters, the drain must be placed first. Different methods of placing backfill may be trialed in order to find the best method of keeping segregation to a minimum. The option to compact the granular filters is available, and was used at the rate of 50 blows of the standard compactor each level for three levels in the C2 test (cap-75 river run).

A.5.7. Backfilling
Once the filter has been placed, the next step is to place the backfill material. In all cases, the backfilling must be gentle to avoid damage to the acrylic, and the drainage system that is in place. In some cases one material acts as both the filter and the backfill (eg river run). In all cases, the purpose of backfill is to allow whatever water passes through the filter system to pass directly to the drainage medium. Where geotextiles are used as a filter, extra care must be taken to ensure no folds or voids are developed when placing the backfill. In some instances this effort may be fruitless as the nature of the backfill may be such that it is impossible to prevent some voids forming between the geotextile and loess face or between the geotextile and backfill. This situation is an important aspect of the system and must therefore be carefully noted. It should also be accounted for that field practises are generally significantly less precise and controlled, and therefore best efforts in a laboratory situation represent an “best achievable” with a particular system.
A.5.8. Water Filling Protocol

This is one aspect of this test that has needed to be developed by trial and error. However, each trial has produced useful results in its own right, as is often the nature of science.

For the test C1, the water feeder tube was filled soon after the backfill was placed. The intention was that the capillary action of the fine soil would draw the water from the water feeder tube (in this case a length of sewer pipe, perforated to approximately 5mm from the top, the whole system being wrapped in Bidim 64 geotextile). It was intended that the water be kept at an approximately constant head (5mm below the top of the tank), but an early failure of the constant head system resulted in an overflow.

A.5.9. Data Collection and Sampling

This test is greatly observational in nature. However, features are in place to attempt to obtain empirical data. The manometer ports do provide information on water flow throughout the system, but may require significantly longer than was available to obtain a satisfactory equilibrium to provide meaningful results.

Material passing through the system may be collected for permeability and mass passed analysis via the drain chute leading from the hole in the tank. Assessment of amount of material (other than water) passing can be made simply by placing the total material passed (including water) into a pre-weighed beaker, and evaporating off the liquid. Reweighing after drying will provide total mass passed over time data. Observations will supplement this by describing the nature of material passed. If time is available, more in depth analysis of the material passing through the system may be possible, for example the method outlined in appendix A.4 to determine particle size distributions, or x-ray diffraction analysis to more accurately determine composition.

A.5.10. Test maintenance

The test must be checked periodically to ensure the required water level is still present in the water feeder system. The use of a good constant head device should eliminate the need for any adjustment over the course of the test. Checks should also be made to ensure all joins in the tank remain water tight throughout the test.
A.5.11. Test completion

Once the testing time has elapsed, the water supply may be turned off and the test be allowed to drain naturally. Observations during the de-rigging of the test may be valuable, and so decisions must be made as to the required state during de-rigging (eg dry, wet, still flowing, etc). This will determine how long the test needs to be left after the water is turned off (if at all), with longer periods resulting in drier conditions.

Figure A.5.ii. Photographic record of stage 2 test set up C1 (using Bidim A24 & tailings backfill).

a. Early stages of set up.

b. After completion of compaction, and scraping face clean, geotextile is placed.

c. After placement of the geotextile tailings backfill is added and the test commenced.
Figure A.5.iii Set up of stage 2 test C2 (river gravel as filter/backfill) general procedure as for C1 with the exception megaflow water feeder, and the backfill being placed directly on novaflo.

River gravel as backfill and filler

Megaflow water feeder system

Figure A.5.iv Set up of stage 2 test C3 including formation of artificial tunnel.

a. Aluminium pipe placed into groove in loess
b. Once fully compacted, pipe removed in sections and hack sawn through.

c. Once pipe is removed and face scraped clean, geotextile is wrapped around novaflo and 7mm premix is placed as a secondary filter, to just below opening.

d. A few days after test commencement, moisture is noticed preferentially coming from artificial tunnel.
Appendix 6. Funnel Test

A.6.1. Introduction.
The test method and results described below, look at the performance of a range of geotextiles at very low hydraulic heads. The test has been performed as a direct result of observations throughout the stage 2 test (see ch.5), where water has been seen pooling on the surface of geotextiles.

A.6.2. Objectives
The primary objective of this test is to compare the performance of various geotextiles under very low hydraulic head, with variation in degrees of saturation. It is also hoped that some potential solutions may be indicated to improve permeability performance under these hydraulic conditions.

A.6.3. Methodology.
A.6.3.1. General description.
The test procedure involves placing samples into filter funnels, using traditional filter paper folding methods. Once placed in the funnel, a measured amount of water is poured carefully into the geotextile with timing beginning as soon as water touches the geotextile. The lap timer is stopped when the first drip is seen to pass through the geotextile, and the running time stopped when a specified quantity has passed through. The test is run a number of times with the same samples, after subjecting them to a range of conditions.

A.6.3.2. Geotextile specimen selection and handling
The samples were obtained from supplied bulk samples as per ASTM D5101-90 (See appendix A.7.I. section 7.2), with the only modification being the sample size; in this case the diameter of each sample was 205mm. Care was taken to assure samples collected were no less than 30cm from any edge of the bulk sample. Although handling was necessary, where possible the samples were handled by the edges, or placed flat on clean paper, to reduce any potential for contamination.
A.6.3.3. Placement into funnel.

Each geotextile sample undergoes the relevant conditioning required for the specific test run (see A.6.3.5.), and is then folded. Folding is done in a similar manner to one of the commonly accepted filter paper techniques. The circular sample was folded in half, and then in half again. The remaining quadrant is then parted in such a fashion that a cone is formed, with one side having three layers of geotextile, and the other side having only one. The folding technique used must be kept consistent throughout the test series.

The differing properties of the various samples for these tests meant that some required more effort to create the folds, and also that some samples had to be held in place in the funnel during testing. The heavier Bidim geotextiles (A44 and A64) and both Terram geotextiles (T1000 and T1500) were held in place using an additional funnel from above, which was positioned on to the three layer half of the sample, high up, so as to avoid any interference with the test results.

A.6.3.4. Water treatment and pouring.

De-aired water is used for each sample. Water is poured as carefully as possible, until it either settles at the bottom, or begins to flow freely so as not to influence the properties of the geotextile by fast water flow. Minimal drop heights are important, and should be no greater than 5cm above the drop point. It is also important to be consistent with where the water is poured. For this series of tests, water was poured approximately halfway up the three-layer side of the geotextile.

Initially, 50mls of water are poured. If, after 1 hour no drop has been observed, this level is classified as a failure and an additional 25mls of water is added. The test is continued for another 30 minutes with the 75mls, after which another 25mls are added. The final addition of another 25mls is made 15 minutes later, provided no drops are evident. This total of 125mls was the highest required in this series of testing, however, it is suggested that if necessary additional 25ml lots should be added after half the preceding time interval (e.g., 1 hr, 30 min, 15 min, 7.5 min etc). Since the experiment must be watched non-stop, one hour is considered adequate to establish a "no drip" tendency under those conditions; this is not to say that the particular conditions will never yield a drip, just that the drip may take longer than is reasonable for this test. The addition of 25mls increases the likelihood of a drip considerably, as it has the effect of creating some minimal turbulence (due to the pouring),
and increasing the hydraulic head, and thus, the time considered reasonable is reduced by half (decided arbitrarily, and may be adapted for specific requirements).

A.6.3.5. Sample conditioning.
The following is a detailed description of the conditions the geotextiles were tested under. Please note that the tests were held in sequence, that is each sample was subjected to the following treatments IN ORDER. All weights are taken after surface water has been removed by use of gentle dabbing with towels, followed by tissue paper (prolonged soaking with either of these is not permitted, as it may remove water that has entered into the structure of the geotextile). It should be noted that, for the highly saturated samples, the definition of surface water is somewhat subjective. In this case it is necessary to simply allow most of the free water to drip off the sample, and then weigh the sample using a plastic bag to protect the scales (pre-weighed to subtract from the total). The conditions used in this series of tests are:

1) As from swath: The geotextiles are cut from the provided bulk sample (as detailed above), and tested directly with no induced change in conditions.
2) 40°C oven until dry: The samples are placed into a 40°C oven until all moisture is removed (ie, no further reduction in weight). Weighing is completed after a 1 hour exposure to air (room temperature) to allow equilibration.
3) Soaked in de-aired water for 30min: The samples are placed flat into a large evaporating dish full of de-aired water. If edges are obviously raised from the surface, they are gently pressed down to ensure even contact with the water. At no stage is the particular sample forced under the water surface. Each sample is turned after 15 minutes, and again, even contact with the water is established.
4) Soaked in de-aired water for 20 hours: Procedure is as for 3, except that turning is done at approximately 5 hours and 15 hours.
5) Placed under reasonable tap flow: The sample is held by the edges under a reasonable flow of tap water. The particular set up for this series was: tap nozzle diameter =5mm, flow rate of approximately 1.5 litres/minute (note this is kept as constant as possible, but some variation is unavoidable), the sample is held approximately 300mm below the outlet. In order to obtain good saturation, the sample was moved in a systematic pattern across the flow. For the thicker samples it is necessary to subject both sides to the flow in order to totally saturate the sample. For samples that do not obviously show a level of saturation visibly, both sides should be subjected to the flow.
6) Air-dried for 8 hours: It is vital that there is no time delay between test 5 and 6. Immediately following test 5, the samples are placed on a drying rack (elevated grid) to allow air to circulate over an optimum surface area.
7) Air-dried for 12 hours: As for 6, with 12 hours of air-drying time.
8) Saturated then air dried for 24 hours: Saturation is by tap flow as described in 5. Air-drying is carried out as described in 6 for 24 hours.
A.6.4. Results.

Data obtained on time to first drip and time taken for 80% of the water to pass through from the first drip is presented in table A.6.1. Data on weight of geotextile after each conditioning is presented in table A.6.2, and indicates the amount of moisture held within the structure of the geotextile for each condition.

All show optimum performance directly after being subjected to a reasonable tap flow. In this condition, the geotextiles are fully saturated, and flow paths are well established, and therefore allow water to pass freely. However, significant differences are noted between the needle punched range and the heat bonded range of geotextiles for most other conditions. The most striking differences are seen in conditions 6, 7 and 8 (the air dried conditions), with the needle punched geotextiles all displaying performances close to fully saturated, whilst the heat bonded geotextiles display significantly impeded performance (up to three orders of magnitude slower to first drip).

The needle punched range in particular shows poorer performance after saturation in de-aired water, than saturation by pelting water from the tap. This perhaps suggests that the technique suggested in ASTM D5101-90 for geotextile saturation may not be 100% efficient.

The Terram range (especially T1500) shows particular susceptibility to drying. After each drying period, either by oven or by air, significant increases in time to first drip, and time for 80% of the water to pass, are noted. This suggests that in a field situation after each drying period, some time will be required to re-establish flow paths and performance on wetting. Contrastingly, the needle punched range tends to display only slight reduction in performance on drying, after an initial phase of saturation. It should be noted that for samples A12, A14, and A34, weights under condition 8 (saturated then air dried for 24hr) are all less than original swath weights, suggesting little or no moisture still present in the geotextile structure. Even with apparently little moisture in the structure, times are significantly improved (eg A14, time to 1st drip from swath = 12s, condition 8 = 1s; time for 80% to pass from swath = 60s, condition 8 = 2.5s). Samples A24, A44 and A64 under condition 8 all show increased weight (suggesting moisture still within geotextile structure), and improved times compared to the from swath condition. These factors suggest that the needle punched range retains moisture well, and flow paths remain established even when little or no moisture is apparent.
Table A.6.1. Data obtained from the “funnel test”

<table>
<thead>
<tr>
<th>Geotextile</th>
<th>A12</th>
<th>A14</th>
<th>A24</th>
<th>A34</th>
<th>A44</th>
<th>A64</th>
<th>T1000</th>
<th>T1500</th>
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</thead>
<tbody>
<tr>
<td>Condition</td>
<td>1st drip 80% passed</td>
<td>1st drip 80% passed</td>
<td>1st drip 80% passed</td>
<td>1st drip 80% passed</td>
<td>1st drip 80% passed</td>
<td>1st drip 80% passed</td>
<td>1st drip 80% passed</td>
<td></td>
</tr>
<tr>
<td>As cut from swath</td>
<td>1794 37</td>
<td>12 60</td>
<td>1422 50</td>
<td>21 23</td>
<td>6577 180</td>
<td>6202 140</td>
<td>4 6</td>
<td>13 189</td>
</tr>
<tr>
<td>40°C oven until dry</td>
<td>63 54</td>
<td>9.5 23.5</td>
<td>3.5 11.5</td>
<td>8 24</td>
<td>217 437</td>
<td>2003 232</td>
<td>8 12</td>
<td>4415 2785</td>
</tr>
<tr>
<td>De-aired water for 30min</td>
<td>1 4.5</td>
<td>1 5.5</td>
<td>1 5</td>
<td>1 9</td>
<td>2.5 50.5</td>
<td>1.5 38.5</td>
<td>9 41</td>
<td>——— ———</td>
</tr>
<tr>
<td>De-aired water for 20hr</td>
<td>1 4</td>
<td>1 9</td>
<td>1 23.5</td>
<td>1 6</td>
<td>2 38</td>
<td>1 27.5</td>
<td>2 7</td>
<td>3 119</td>
</tr>
<tr>
<td>Reasonable tap flow</td>
<td>1 4</td>
<td>1 2.5</td>
<td>1 2.5</td>
<td>1 5.5</td>
<td>1 3.5</td>
<td>1 3</td>
<td>2 18</td>
<td>1 9</td>
</tr>
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<td>Air dried for 8hr</td>
<td>1 5</td>
<td>2 8</td>
<td>1 4.5</td>
<td>1.5 6</td>
<td>1 4.5</td>
<td>1 5.5</td>
<td>17 1753</td>
<td>5702 2307</td>
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<tr>
<td>Air dried for 12hr</td>
<td>2.5 7.5</td>
<td>5 5.5</td>
<td>1 4.5</td>
<td>1 9</td>
<td>1.5 7.5</td>
<td>2 8</td>
<td>337 867</td>
<td>4604 9796</td>
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<td>Tap flow + air dried for 24hr</td>
<td>1 2.8</td>
<td>1 2.5</td>
<td>2 4.5</td>
<td>1.5 5.5</td>
<td>3.4 5.4</td>
<td>2 6</td>
<td>340 196</td>
<td>208 16142</td>
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</table>

Notes:
*1. Time to first drip through (in seconds from start). 50mls placed initially increased to 75mls after 3600 seconds, 100mls after 5400s, and 125mls after 6300s.
*2. Time for 80% of water to pass through.
*3. For these thick nonwoven geotextiles, it is necessary to estimate the time when 20% remains visible on the geotextile, rather than when 80% has passed, since greater than 20% of 50mls may be absorbed into the geotextile structure.

Table A.6.2. Before and after weights for funnel test

<table>
<thead>
<tr>
<th>Geotextile</th>
<th>A12</th>
<th>A14</th>
<th>A24</th>
<th>A34</th>
<th>A44</th>
<th>A64</th>
<th>T1000</th>
<th>T1500</th>
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<tr>
<td>Condition</td>
<td>Weight (g)</td>
<td>Weight (g)</td>
<td>Weight (g)</td>
<td>Weight (g)</td>
<td>Weight (g)</td>
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<td>Weight (g)</td>
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<td>1</td>
<td>3.75</td>
<td>3.89</td>
<td>6.04</td>
<td>8.85</td>
<td>11.03</td>
<td>16.13</td>
<td>4.30</td>
<td>6.65</td>
</tr>
<tr>
<td>2</td>
<td>3.74</td>
<td>3.88</td>
<td>6.03</td>
<td>8.85</td>
<td>11.02</td>
<td>16.1</td>
<td>4.30</td>
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<td>4</td>
<td>4.34</td>
<td>4.98</td>
<td>7.35</td>
<td>11.21</td>
<td>15.98</td>
<td>21.35</td>
<td>7.45</td>
<td>8.60</td>
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<td>5</td>
<td>36.40</td>
<td>39.84</td>
<td>57.2</td>
<td>8.84</td>
<td>95.87</td>
<td>121.24</td>
<td>10.86</td>
<td>15.38</td>
</tr>
<tr>
<td>6</td>
<td>10.16</td>
<td>15.00</td>
<td>32.43</td>
<td>35.23</td>
<td>62.74</td>
<td>87.7</td>
<td>4.28</td>
<td>6.6</td>
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<tr>
<td>7</td>
<td>3.72</td>
<td>3.88</td>
<td>12.73</td>
<td>35.72</td>
<td>38.09</td>
<td>58.45</td>
<td>4.29</td>
<td>6.61</td>
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<tr>
<td>8</td>
<td>3.73</td>
<td>3.87</td>
<td>13.40</td>
<td>8.82</td>
<td>16.64</td>
<td>48.6</td>
<td>4.27</td>
<td>6.61</td>
</tr>
</tbody>
</table>

Note: conditions in order as for table A.6.1. Weights given are directly following conditioning, prior to placement in funnel.
A.6.5. Discussion

A.6.5.1. Test methodology

The method, as described above, has a number of shortcomings and variables that must be considered. The major variable in the experiment, is the placement of the geotextile; which includes folding, and the physical geometric relationships in place.

Folding: The heat bonded nature of the Terram product makes its structure much more difficult to bend easily, and the resultant folds once placed, have a more significant impact on the sample as a whole, than for the more pliable needle punched materials. Folding in the Terram can cause realignment, significant stretching, and even localized breakages in the fibers. Also, the folds tend to remain in place after each trial. It could be inferred that these alterations of structure are partly responsible for the greater ease of flow along these folds. The needle punched material, however, is considerably more pliable, with folding resulting in significantly less strain and likelihood of breakage; folds are only barely detectable in the heavier grade needle punched material after each test.

Method of pouring water: Although all attempts were made to pour the water in a consistent manner high up on the folded side, in some instances it was difficult to control the flow exactly (whilst trying to observe and time water flow, or the first drip). The results of the experiment show that the pelting effect of water has a significant influence on the performance of the geotextile. A slight change in attitude of the pouring container can change the impact of the flow significantly, and in most cases these changes will not be observed, as attention is focused on the capturing container. It is suggested that practice runs be done to perfect a consistent pouring procedure prior to test runs.

Alignment of geotextile in filter funnel: It is difficult to place each sample in exactly the same manner, and as a result, some have the three layer fold at the lowest point in the funnel, while others have the single face at the lowest point in the funnel. The net effect of this should be minimal, but with samples having the single face as the lowest point, having a greater chance of beginning flow sooner. Again, trial runs should reduce this problem, but can not eliminate it.
Moisture history: It may be possible that at some stage in the brief lives of the various samples prior to testing, some may have been subjected to greater amounts of moisture in the air, and that this moisture may have been incorporated onto the fibres. The tests show that increasing saturation results in faster initiation of flow through the geotextile, and it is possible that even low levels of moisture may have a similar effect.

Temperature: Little variation occurred over the course of the tests, so influence was minimal.

Repeatability: As this test was performed as an aside to the main focus of this study, time was not available to test how repeatable results are. It is thought that variations between samples will be significant, but general trends will most likely hold.

A.6.5.2. Relevance to field situations

As pointed out by Tony Lingley of Maccaferri Ltd in Christchurch (pers. comm. 1998), as soon as pressure is applied to the geotextile, permeability greatly increases. In most retaining wall situations confining pressures are present. However, it seems important to consider performance under the investigated conditions, as they are commonly associated with conditions behind retaining walls on the Port Hills, as has been observed in the stage 2 tests.

In a number of cases, pressure may not always be placed on a critical spot on a geotextile, thereby reducing its performance to those indicated by this series of tests. Geotextiles are exclusively placed dry, and in many situations drying will continue with the sometimes long, hot, and dry summers in Christchurch. Hydraulic conditions can readily be equated with these tests, with low heads being a common occurrence within a retaining wall filtration/drainage system. Reasons for poor contact of geotextiles with loess faces and backfill are varied. These include poor installation, including placement of too coarse backfill; poor securing of geotextiles during backfill resulting in folds or wrinkles; placement on very uneven faces; poor joining procedures at seams; differential subsidence after installation; inadequate surface erosion protection measures resulting in erosion of loess face (during and post construction).

The fact that there are situations where less than favourable conditions could result in poor contact between loess, geotextile and backfill, and where low water pressures may be present, means that it may be prudent to investigate possible measures to increase the
performance of geotextiles in such conditions. The above tests show that saturation by water pelting greatly increases performance, even for a significant period following that saturation (for the Bidim range); perhaps saturation of geotextiles by hose once placed or prior to placement may be a useful technique. This would require significant study to investigate effectiveness and develop safe procedures. It may be possible to exploit the dipolar nature of water molecules to assist in attracting water into the structure of the geotextiles by coating the fibres with an appropriate compound. Again, this would require extensive study, including investigation on the effects, if any, there may be on the development of the soil geotextile interface.

A.6.6. Conclusions

A number of conclusions may tentatively be drawn from this brief study:

➢ Geotextiles tested show significant delays in allowing an initial drip to pass through at very low heads.

➢ Performance of geotextiles at very low heads is improved with increasing saturation.

➢ Bidim geotextile range shows prolonged improvement in performance after initial saturation and subsequent drying.

➢ Terram geotextile range shows significant reduction in performance on drying after initial saturation.

➢ Pelting water (eg from a tap) is more efficient at saturating geotextiles than soaking in de-aired water.

➢ Initial study suggests saturation of Bidim range prior to installation in a retaining wall situation may improve permeability performance at very low heads long term.

➢ Further study is required to establish repeatability of test results and to investigate performance enhancing treatment methods.
Appendix 7. ASTM D5101-90.
A.7.1 Introduction and adaptations.
The ASTM D5101-90 testing procedure as presented (in full) in section A.7.2. has a number of features that have been adapted for this series of tests. Were it is considered changes to the wording of the standard may be appropriate, the relevant section will be quoted, and then the re-worded versions will be proposed, followed by a brief explanation of the implications, and justification for the suggestion. Additional adaptations not considered worthy of changes to the standard will then be listed.

Suggestion 1:
- Section number and quote:
  Section 6.20 “Wooden rod, 20mm diameter by 150mm long”
- Suggested change:
  “Wooden rod that will provide sufficient consolidation following the method outlined in section 9.4.2. Typically dimension are 20mm diameter by 150mm diameter with a weight of XXX grams [to be established].”
- Discussion:
  This description does not specify the wood type or weight, both of which are factors that would significantly influence the effect of the later described procedure with this piece of apparatus. The important aspect of this piece of apparatus is what effect it has on the test sample. Therefore the suggestion offered relates to the performance of the apparatus rather than the simple dimensions of it; the inclusion of typical values, including weight, provides a simple starting point for trial with the particular samples used. Since this suggestion alters the uniform nature of this part of the standard, the exact nature of the rod used should be included into the report.

Suggestion 2:
- Section number and quote:
  section 9.4.2. “...Consolidation of each layer shall consist of tapping the side of the permeameter six times with a 20mm diameter by 150mm long wooden rod.”
- Suggested change:
  ‘...Consolidation of each layer shall consist of tapping the side of the permeameter sufficient times so as to provide a satisfactory level of consolidation (i.e. so that no abnormally large pore spaces remain, and none open up during backfilling - this may require trial and error)’
- Discussion

The current standard method does not specify where to tap, or give an indication of what constitutes a tap, both of which may significantly influence the effectiveness of the procedure. As it is admitted it may be rather difficult to quantify the force of a “tap” the suggestion is more a performance based one, indicating the desired outcome rather than suggesting rather vague methodology.

Other adaptations:

1) Use of 100µm soil support screen and manometer covers instead of 200µm as a result of the fine grain size of the tested samples.

2) CO₂ Backfilling as rate of approximately 0.1/minute for extended periods (see set up check list in A.7.3)

3) Use of retort stand instead of clamps to hold permeameter securely (issue of cost and logistics)

4) Also adapted was the water backfilling procedure after the first two tests. Observations of significant disruption of soil at the soil/geotextile interface when the O/CHD was initially placed at a level 25mm above this interface (as suggested in the standard) suggested the necessity for change. For all subsequent tests the initial level of the O/CHD was as close to the geotextile level as possible to allow saturation of the soil in direct contact, before subjecting it to any pressure, thereby allowing the soil structure to adjust without significant segregation occurring. As this approach was still not 100% effective, more study is needed to reduce this effect that may significantly impact on the system’s ability to create a filtration system.

5) For the test where cyclone sand was used in conjunction with Bidim A14 as a filter, the adapted procedure was as follows: Soil was added to 5mm below the bottom flange (in the inverted position) as per normal. The bottom of an appropriately sized beaker was then used to compress the soil to a similar level as would be obtained in the standard procedure, on placement of the bottom section and geotextile. The soil level was now at the level of the bottom piping barrier (15mm from bottom flange), at which point the sand was added. Once sand had been placed to the level of the bottom flange, the geotextile and bottom sections were placed and fitted. Appropriate adjustments were made to gradient ratio calculations.
Standard Test Method for Measuring the Soil-Geotextile System Clogging Potential by the Gradient Ratio

1. Scope

1.1 This test method is a performance test applicable for determining the soil-geotextile system permeability and clogging behavior under unidirectional flow conditions.

1.2 The values stated in SI units are to be regarded as standard. The values in parentheses are for information only.

1.3 This standard does not purport to address all the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2. Referenced Documents

2.1 ASTM Standards:

D123 Terminology Relating to Textiles
D653 Terminology Relating to Soil and Rock
D737 Test Methods for Air Permeability of Textiles
D4354 Practice for Sampling of Geotextiles for Testing
D4439 Terminology Relating to Geotextiles

3. Terminology

3.1 Definitions:

3.1.1 clogging potential, \( n \)—in geotextiles, the tendency for a geotextile to decrease permeability due to soil particles that have either lodged in the geotextile openings or have built up a restrictive layer on the surface of the geotextile.

3.1.2 geotextile, \( g \)—any permeable textile material used with foundation, soil, rock, earth, or any other geotechnical engineering related material, as an integral part of man-made product, structure, or system.

3.1.3 gradient ratio, \( \gamma \)—in geotextiles, the ratio of the hydraulic gradient through a soil-geotextile system to the hydraulic gradient through the soil alone.

3.1.4 hydraulic gradient, \( i \), \( \gamma (D) \)—the loss of hydraulic head per unit distance of flow, \( \Delta H/\Delta L \).

3.1.5 For definitions of other textile terms, refer to Terminology D 123. For definitions of other terms related to geotextiles, refer to Terminology D 653.

3.2 Symbols and Acronyms:

3.2.1 \( \text{CO}_2 \)—the chemical formula for carbon dioxide gas.

3.2.2 CHD—the acronym for constant head device.

4. Summary of Test Method

4.1 This test method requires setting up a cylindrical, clear plastic permeameter (see Figs. 1 and 3) with a geotextile and soil, and passing water through this system by applying various differential heads. Measurements of differential heads and flow rates are taken at different time intervals to determine hydraulic gradients. The following test procedure describes equipment needed, the testing procedures, and calculations.

5. Significance and Use

5.1 This test method is recommended for evaluating the performance of various soil-geotextile systems under controlled test conditions. Gradient ratio values obtained may be used as an indication of the soil-geotextile system clogging potential and permeability. This test method is not appropriate for initial comparison or acceptance testing of various geotextiles. The test is intended to evaluate geotextile performance with specific on-site soils. It is improper to utilize the test results for job specifications or manufacturers' certifications.

5.2 It is important to note the changes in gradient ratio values with time versus the different system hydraulic gradients, and the changes in the rate of flow through the system (see Section 1).

6. Apparatus

6.1 Soil-Geotextile Permeameter, (three-piece unit) equipped with support stand, soil-geotextile support screen, piping barriers (caulk), clamping brackets, and plastic tubing (see Fig. 2).

6.2 Two Constant Water Head Devices, one mounted on a jack stand (adjustable) and one stationary (Fig. 3).

6.3 Soil Leveling Device (Fig. 4).

6.4 Manometer Board, of parallel glass tubes and measuring rulers.

6.5 Two Soil Support Screens, of approximately 5 mm (No. 4) mesh.
For laboratory samples, take a full width swath of geotextile from each roll of material in the lot sample at least 1 m (3 ft) long cut from the end of the roll after discarding the first meter of material from the outside of the roll.

7.3 Test Specimens—Cut three circular specimens from each swath in the laboratory sample with each specimen having a diameter of 110 mm (4.33 in.). Locate two specimens no less than 300 mm (11.8 in.) from each edge of the swath and one at the center of the swath width.

8. Conditioning

8.1 Test Water Preparation:

8.1.1 Test water should be maintained at room temperature about 16 to 27°C (60 to 80°F) and deaired to a dissolved oxygen content of 6 parts per million (ppm) or less before introducing it to permeameter system. This will reduce or eliminate the problems associated with air bubbles forming within the test apparatus.

8.1.2 An algae inhibitor or micro screen should be used to eliminate any algae buildup in the system.

8.2 Specimen Conditions:

8.2.1 Condition the specimen by soaking it in a container of deaired water for a period of 2 h. Dry the surface of the specimen by blotting prior to inserting in the permeameter.

9. Procedure

9.1 Preparation of Apparatus:

9.1.1 Thoroughly clean and dry permeameter sections.

9.1.2 Close all valves and cover the inside openings of all manometer ports with fine wire mesh or lightweight non-woven fabric (the equivalent of No. 100 mesh).

9.1.3 Lubricate all O-ring gaskets.

9.2 Permeameter Preassembly:

9.2.1 Stand center section of the permeameter on end and place a soil support cloth 110 mm (4.33 in.) diameter on recessed permeameter flanges.

9.2.2 Insert support screen 110 mm (4.33 in.) diameter on top of support cloth with mesh side against the cloth.

9.2.3 Align and insert top section of the permeameter into center section and press until there is a tight fit to secure the support cloth and screen in place. Assure that all gasket edges secure against the support cloth, support bracket, and between the center and top permeameter sections.

9.2.4 Invert and place permeameter into holding stand.

9.3 Process Soil:

9.3.1 Thoroughly air dry the soil sample as received from the field. This shall be done for a minimum of three days. Pulverize the sample in a mortar with a rubber-covered pestle (or in some other way that does not cause breakdown of individual grains), to reduce the particle size to a maximum of 10 mm (3/8 in.). Select a representative sample of the amount required (approximately 1250 g) to perform the test by the method of quartering or by the use of a soil splitter.

9.3.2 Select that portion of the air-dried sample selected for purpose of tests and record the mass as the mass of the total test sample uncorrected for hygroscopic moisture. Separate the test sample by sieving with a 2-mm (No. 10) sieve. Pulverize that fraction retained on the 2-mm (No. 10) sieve in a mortar with a rubber-covered pestle until the aggregations of soil particles are broken up into the separate grains.
9.3.3 Mix the fractions passing the 2-mm (No. 10) sieve along with the portion that was retained on the 2-mm (No. 10) sieve to form the test soil. All particles larger than 10 mm (3/8 in.) should be eliminated.

9.4 Soil Placement:

9.4.1 Weigh out approximately 1350 g of air dried processed soil.

9.4.2 Place air dried processed soil above the support cloth to a depth of 110 mm (4.33 in.). The final depth of soil after settlement will be approximately 100 mm (4 in.). The soil should be placed in 25 mm (1-in.) to 40-mm (1½-in.) layers, making sure that no voids exist along the permeameter walls at manometer ports, or the caulk piping barriers. The soil shall be placed carefully into the permeameter with a scoop or appropriate tool with a maximum drop of the soil no greater than 25 mm (1 in.). Consolidation of each layer shall consist of tapping the side of the permeameter six times with a 20 mm (3/4 in.) diameter by 150 mm (6 in.) long wooden rod.

9.4.3 When the level of the soil in the permeameter reaches a depth of 100 mm (4 in.), insert the soil leveling device (Fig. 4), with the notch down, on the top edges of the permeameter. Continue placing soil and rotating the leveling device until the total soil height of 110 mm (4.33 in.) is reached.

9.4.4 Remove the soil leveler and any excess soil. Determine the mass of the soil in the permeameter for unit weight calculations.

Note 2—The specified soil placement procedure results in a relatively loose soil condition and is conservative for many applications. If a density approximating actual field soil conditions is desirable, the test could be run at this specified soil density. It should be recognized, however, that predicting field soil conditions may be very difficult due to construction installation procedures that generally disturb and loosen soils adjacent to the geotextile.
9.5 Permeameter Assembly and Setup:
9.5.1 Clean the inner flange of the center section of the permeameter and insert the geotextile to be tested.
9.5.2 Insert support screen on top of geotextile with the mesh side against the geotextile.
9.5.3 Align and insert the bottom section of the permeameter into the center section and press tightly to secure the geotextile and support screen. The soil will compress from 110 mm (4.33 in.) to approximately 100 mm (4 in.) when the bottom section is secured. Check gaskets to assure contact is made between permeameter sections, support screen, and geotextile.
9.5.4 Secure the permeameter sections together within clamp brackets and tighten bolts on bracket rods evenly.
9.5.5 Invert permeameter into holding stand so that the geotextile will be below the soil level.
9.5.6 Connect the inflow and outflow constant head devices (CHD) to their corresponding permeameter ports (see Fig. 3) with plastic tubing. The outflow CHD is attached to the bottom permeameter port and inflow CHD is attached to the top permeameter port.
9.5.7 Connect all manometer tubes (1 through 5) to their corresponding permeameter manometer ports, and all outflow tubes to their corresponding outlet ports.
9.6 Saturating the Soil/Geotextile System:
9.6.1 Open the top vent valve, and close off the permeameter water outlet hose.
9.6.2 Backfill permeameter with water through the outflow CHD until the water level is approximately 10 mm (% in.) below the open manometer port 6. Stop waterflow into the permeameter by clamping off the hose between outflow CHD and permeameter.
9.6.3 Expel oxygen and other gases in permeameter and soil system by (1) attaching a carbon dioxide (CO₂) line to manometer port 6, and (2) regulating the gas flow at 2 L/min and purging the system for 5 min.
NOTE 3—The permeameter may be backfilled without purging with CO₂; however, the potential for air pockets within the soil to cause erratic results for flow and pressure measurements will be greater without the purging.
9.6.4 After 5 min of gas saturation, seal off (plug) the open end of each manometer tube (1 through 5) and continue to purge the system with CO₂ for an additional 5 min with only the top vent valve open.
9.6.5 Remove the CO₂ gas line and replace the No. 6 manometer hose. Remove the seals (plugs or clamps) from all manometer tubes (1 through 5).
9.6.6 Loosen hose clamp between outflow CHD and permeameter, and fill soil section of permeameter with water. Filling is accomplished by adding water to and raising the level on outflow CHD slowly. Start with outflow CHD at 25 mm (1 in.) above the geotextile level and raise 25 mm (1 in.) every 30 min until water level is 50 mm (2 in.) above the top support screen bracket. This slow saturating process is necessary to prevent air pockets or internal soil movement during loading.
9.6.7 Clamp hose between outflow CHD and permeameter to prevent flow. Continue to raise the water level in the
9.6.8 Close off top vent valve and allow the system to stand overnight in a static condition. This should ensure complete saturation of the system with water. The system should be in a no-flow condition overnight.

9.6.9 Check for and remove air bubbles found in the tubes or manometers by light vibration or tapping. It may be necessary to disconnect tubing from the manometer board and slowly lower the tubing, allowing water and entrapped air to run out.

9.6.10 Place a thermometer into the inflow CHD to monitor temperature of water flowing into permeameter.

9.7 Running the Test:

9.7.1 Check to make sure that all scales on the manometer board are set to a common reference elevation.

9.7.2 Adjust the inflow CHD to a level so that a hydraulic gradient (i) of 1 is obtained (see 10.1).

9.7.3 Unclamp hoses between the permeameter and CHD's to allow flow, and record the initial starting time.

9.7.4 Record the following data (using Fig. 5) at 0, 1/2, 1, 2, 4, 6, and 24 h from the initial starting time:

- The time in hours (accumulated).
- The flow rate from the system (outflow CHD); time (t) in seconds for a measured quantity of flow (Q) in cubic centimetres. Measure for a minimum duration of 30 s and a minimum quantity of flow of 10 cm³.
- The temperature in degrees Celsius of the water in the system.
- The water level readings from the individual manometers.
- The date and time of day.

9.7.5 After the 24-h reading, raise the inflow CHD to obtain a system hydraulic gradient (i) = 2.5. Record time. After 1 h at this level, record all data.

9.7.6 Raise the inflow CHD to obtain i = 5. Repeat measurements as in 9.7.4.

9.7.7 After 24-h reading, raise the inflow CHD to obtain i = 7.5. Record time. After 1 h, record all data.

9.7.8 Raise the inflow CHD to a level to obtain i = 10. Repeat measurements as in 9.7.4.

NOTE: This test can be run at hydraulic gradients other than those specified in this procedure, for example, i = 3 for 24 h. In all cases, the system hydraulic gradients should be increased gradually and in increments no greater than i = 2.5 and maintain those incremented levels for a minimum of 30 min. The test may also be run at longer intervals than 24 h, until some recognizable equilibrium or stabilization of the system has occurred.

9.7.9 The test must be run continuously. Once the test has started, it cannot be stopped and then resumed.

10. Calculation

10.1 Hydraulic Gradient—Calculate the hydraulic gradients for the system i, using Eq. 1. Figure 6 shows the meaning of the values in the equation schematically.

\[ i = \frac{\Delta h}{L} \]  

where:
\[ \Delta h = \text{difference in manometer readings for soil zone analyzed, manometer 1 minus manometer 6, cm} \]
\[ L = \text{length or thickness of soil between manometers being analyzed, cm} \]

10.2 System Permeability—Calculate the system permeability at the temperature of the test and corrected to 20°C using Eqs 2 and 3:

\[ k_T = \frac{Qf(iLt)}{100} \]  

\[ k_{20} = k_T \frac{k_{20}}{k_T} \]

where:
\[ k_T = \text{system permeability at test temperature, m/s} \]
\[ k_{20} = \text{system permeability at 20°C, m/s} \]
FIG. 5 Gradient Ratio Permeameter Data

\[ GR = \frac{Q}{A \cdot i \cdot t} = \frac{(\Delta h_i / L_i) / (\Delta h_f / L_o)}{\Delta h_f / L_o} \]

where:
\[ \Delta h_i = \frac{(M_3 - M_4) + (M_5 - M_6)}{2} \]
\[ \Delta h_f = \frac{(M_4 - M_5) + (M_5 - M_6)}{2} \]

(Mₙ = the manometer reading, cm, for the manometer numbered n.)
Lₙ = 5.10 cm (2 in.) and
Lₒ = 2.55 cm (1 in. + the geotextile thickness) (Test Method for Measuring Thickness of Geotextiles, Geomembranes, and Related Products)

Calculate values from two sets of manometers, as shown above, to detect any changes in pressure from one side to the other. If a significant difference exists between manometers, the system should be investigated for air bubbles, algae buildup, plugged manometer tube, or a plugged port.

11. Report
11.1 State that the specimens were tested as directed in Test Method D 5101. Describe the material or product tested and the method of sampling used.
11.2 Report the following information:
11.2.1 Unit weight of dry soil in the permeameter,
11.2.2 All instrument readings, such as flow volume, flow time, temperature, and manometer readings,
11.2.3 System permeability corrected to 20°C,
11.2.4 Gradient ratio for the system,
11.2.5 A plot of the gradient ratio to the nearest 0.1 unit against time (hours) for each hydraulic gradient tested,
11.2.6 A plot of the permeability to three significant digits against time (hours), and
11.2.7 A plot of the gradient ratio versus the system hydraulic gradient (i).

12. Precision and Bias
12.1 Precision—Precision of this test method is being established.
12.2 Bias—The procedure in Test Method D 5101 for measuring the soil-geotextile system permeability and clogging potential has no bias because the value of the gradient ratio and permeability can be defined only in terms of a test method.

13. Keywords
13.1 clogging potential; gradient ratio; soil-geotextile system
The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, PA 19103.
Appendix 8. Moisture Content Methodology.

A.8.1. Apparatus.
250ml beaker, at least 100g of material to be tested, balance accurate to 0.01g, thermostatically controlled oven, thermometer, record sheet.

A.8.2. Procedure.
The weight of the clean beaker is recorded. The sample to be tested may then be placed into the beaker, and reweighed immediately. The sample is then placed into the thermostatically controlled oven at 50°C (±5°C). The temperature should be confirmed with the thermometer. This temperature range should not significantly affect clay minerals if further analysis of the sample is required later. The sample must remain in the oven for a minimum of 72 hours after which time it should be removed from the oven and be allowed to equilibrate for a period of at least 1 hour prior to reweighing. The sample should be returned to the oven for an additional 24 hours, removed, then rested for an hour before another weighing. If the second dry weight is consistent with the first, this value may be used for the calculations. If the second dry weight is less than the first, the sample must be returned for as many 24 hour periods as necessary to obtain a consistent weight.

A.8.3. Sample data sheet.

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<th>Sample No.</th>
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Appendix 9. **Modified Emerson Crumb Dispersion Test**

The following has been copied directly from Yetton (1986). The suggestion of Jowett (1995) that the classifications below be subdivided to include 0.5 divisions between the given classifications, is supported, but was not required. Samples tested for this thesis were consistently found to be class 2.0.

**Procedure**

A crumb of soil, preserved at in situ moisture content and about 4 to 6 mm in diameter, is dropped into a beaker of distilled water. The extent to which the clay fraction goes into colloidal suspension (disperses) **without agitation or disturbance** is observed after ten minutes.

**Classification** (see figure below for photographic representation)

**Class 1** No Reaction.
Crumb may slake and run out on the bottom of the beaker in flat pile, but no sign of cloudy water caused by colloids in suspension.

**Class 2** Slight Reaction
Slight cloud in water near the surface of the crumb.

**Class 3** Moderate Reaction
Easily recognisable cloud of colloids in suspension around the sample.

**Class 4** Strong Reaction
Colloidal cloud virtually obscures the whole bottom of the beaker and in extreme cases the whole beaker becomes cloudy.

Figure A.9.i. Dispersion classes, from left to right: 1, 2, 3, 3, 4