AN EVALUATION OF BASE SOIL-FILTER COMPATIBILITY USING A TRIAXIAL PERMEAMETER

by

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Abstract

Filtration compatibility of base soil and granular filter materials must be addressed in the design of zoned engineered fill structures. The evolution of design practice governing filter compatibility is reviewed, with emphasis placed on the experimental studies that have made the greatest contribution to current guidelines. Design practice governing filter compatibility for cohesionless uniform materials has remained relatively unchanged for the last 70 years. A novel and improved triaxial permeameter is used to test base soil-filter grain size ratios \((D_{15}/d_{85})\) close to the limit of filter incompatibility. The configuration and operation of the test device are described. Thereafter, data are reported for select combinations of base soil-filter specimens of glass beads that are reconstituted, consolidated and subject to unidirectional seepage flow. Interpretation of the test results addresses the onset of filter incompatibility with reference to independent measurements of change in permeability of the two-layer system and mass loss of the base soil through the filter. A unified framework is presented for interpretation of filter incompatibility, taking into account the influence of stress and hydraulic gradient. The implications of the test results are analyzed and discussed with reference to a confident understanding of base-soil filter compatibility.
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List of Symbols

\( A \)  = cross-sectional area of the specimen \( \text{m}^2 \)
\( A_c \)  = corrected cross-sectional area of the specimen \( \text{m}^3 \)
\( C \)  = relative permeability
\( C_u \)  = coefficient of uniformity \( \% \)
\( D_a \)  = diameter of the sphere \( \text{mm} \)
\( D_{15} \)  = grain size diameter at which 15\% of the filter material is finer \( \% \)
\( d_{85} \)  = grain size diameter at which 85\% of the base soil material is finer \( \% \)
\( e \)  = void ratio
\( g \)  = acceleration due to gravity \( \text{m/sec}^2 \)
\( \Delta h_t \)  = difference in total head \( \text{m} \)
\( \Delta h_s \)  = head loss across the specimen \( \text{m} \)
\( i \)  = hydraulic gradient
\( i_{cr} \)  = critical hydraulic gradient
\( k \)  = permeability \( \text{cm/sec} \)
\( k' \)  = effective permeability \( \text{cm/sec} \)
\( k_{bf} \)  = permeability of the base soil-filter specimen \( \text{cm/sec} \)
\( k_v \)  = theoretical permeability values \( \text{cm/sec} \)
\( k_b \)  = permeability of the base soil layer \( \text{cm/sec} \)
\( k_f \)  = permeability of the filter layer \( \text{cm/sec} \)
\( L \) = total length of the specimen after consolidation \( \text{mm} \)

\( L_b \) = length of the base soil layer prior to consolidation \( \text{mm} \)

\( L_f \) = length of the filter layer prior to consolidation \( \text{mm} \)

\( \Delta l \) = axial deformation \( \text{mm} \)

\( M_{bc} \) = mass of the base soil after consolidation \( \text{g} \)

\( m \) = mass loss \( \text{g} \)

\( m_a \) = mass loss per unit area \( \text{g/m}^2 \)

\( m_{rc} \) = mass loss due to reconstitution and consolidation \( \% \)

\( m_t \) = seepage-induced cumulative total mass loss \( \% \)

\( q \) = flow rate \( \text{cm}^3/\text{sec} \)

\( t \) = time \( \text{min} \)

\( \text{Re} \) = Reynolds number

\( v \) = discharge velocity \( \text{cm/sec} \)

\( V_o \) = volume of the specimen prior to consolidation \( \text{cm}^3 \)

\( \Delta V \) = change of volume due to consolidation \( \text{cm}^3 \)

\( \gamma_w \) = unit weight of water \( \text{kN/m}^3 \)

\( \mu \) = viscosity of water at 20\(^\circ\)C \( \text{kg/(ms)} \)

\( \sigma'_3 \) = effective stress (cell pressure) \( \text{kPa} \)
List of Abbreviations

ASTM = American Society for Testing and Materials
CHD = Constant head device
DAS = Data acquisition system
DPT = Differential pressure transducer
LVDT = Linear variable differential transformer
NEF = No Erosion Filter test
NRCS = Natural Resources Conservative Service
TPT = Total pressure transducer
UBC = The University of British Columbia
USBR = United States Bureau of Reclamation
USCS = Unified Soil Classification System
WES = Waterways Experiment Station
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Dedication

To my wife Gianella and my parents:

Thank you for all your love and unconditional support.
Chapter 1: Introduction

1.1 Influence of seepage flow in granular filters

In geotechnical practice, verification of the filtration compatibility of adjacent layers with respect to soil retention and permeability must be addressed in the design of zoned engineered fill structures (such as dams and levees), as well as any evaluation of soil strata on which those structures are founded. Filtration compatibility between the soil (base soil), which needs to be protected against seepage flow, and the granular soil (filter) is crucial for the earth performance of many types of fill structure. This sort of compatibility between both adjacent layers is called hereafter “filter compatibility”.

Over time, there has been general recognition of three functional requirements for filter compatibility: phenomenon of base soil retention; permeability; and internal stability. This study is focused only on the phenomenon of base soil retention which, for purposes of design, is typically characterized by a ratio between the grain size diameter at which 15% of the filter material ($D_{15}$) is finer, and the grain size diameter at which 85% of the base soil to be protected ($d_{85}$) is finer. Accordingly, the influence of the grain size ratio $D_{15}/d_{85}$ is examined in this thesis research.

More specifically, this thesis presents a systematic experimental study performed at the University of British Columbia (UBC), in which base soil-filter specimens were reconstituted, isotropically consolidated and subjected to seepage flow in a downward direction using a new stress-controlled test device. Aspects of the test device and procedure that are examined are
isotropic confining stress, multistage seepage flow, test repeatability, and the effects of D_{15}/d_{85}
on the critical gradient at which the onset of base soil-filter incompatibility occurred.

Filter studies that have examined the base soil-filter compatibility under an applied confiningstress are very limited. Only the study performed by Tomlinson and Vaid (2000) evaluated filtercompatibility under axial loading and multistage seepage flow induced by increasing thehydraulic gradient incrementally. Accordingly, a continuation of the base soil-filter compatibilityinvestigation is believed necessary in order to provide a unified and more confident frameworkfor interpretation of filter incompatibility, taking into account the influence of isotropic stressand hydraulic gradient.

1.2 Objectives of the study and thesis organization

The objectives of this study are as follows:

1. Develop an improve stress-controlled device to test base soil-filter specimens made of artificial glass beads, and obtain good quality laboratory test data.

2. Define the onset of filter incompatibility based on two independent test parameters, namely permeability and mass loss of the base soil through the filter.

3. Investigate if the onset of filter incompatibility for base soil-filter specimens is stress-dependent.

4. Evaluate the margin of safety of the soil retention design criterion rule for uniformly-graded coarse-grained materials.
The thesis consists of seven chapters, which are briefly outlined as follows:

- Chapter 1: Introduction to the topic of filter compatibility, and description of the objectives of this study.
- Chapter 2: A literature review of experimental studies, which focus on test equipment and relevant experimental findings which support the current design guidelines for granular filters. A brief review of seepage flow regime theory is also given.
- Chapter 3: Description of the new triaxial permeameter, including details of its configuration and associated instrumentation used to record test data. Details of a companion rigid-wall permeameter, which is used for complementary tests, are also described.
- Chapter 4: Description of test materials, testing program and procedure to operate the new triaxial permeameter and the rigid-wall permeameter.
- Chapter 5: Description of the results of the testing program.
- Chapter 6: Analysis and discussion of the results.
- Chapter 7: Concluding remarks and recommendations for further research.
Chapter 2: Literature review

This chapter presents a review of experimental investigations related to filter compatibility studies. Discussion addresses the evidence in support of base soil retention criteria in current design guidelines for granular filters, since some of these criteria are founded on experimental data and others arise from subjective rules. Experimental data are described with emphasis on test equipment and the relevant experimental findings. Seepage flow regime theory is also reviewed, because experimental findings of this study suggest a non-Darcian flow likely prevails in many experimental studies.

Design practice governing filter compatibility for uniformly-graded soil has remained relatively unchanged for the last 75 years, in spite of numerous filter compatibility studies that have been performed since the early work of Bertram (1940). Indeed, only a few experimental filter studies have significantly influenced the current design guidelines for granular filters. A generally accepted guideline for filter design was developed by the Soil Conservation Service (now Natural Resources Conservation Service (NRCS)) in Part 633 of the National Engineering Handbook, chapter 26 (NRCS, 1994).

A very important step in any filter design guideline is a categorization of the base soil material, which is typically, based on the percentage passing the No. 200 (0.075 mm) sieve. This categorization is performed after regrading the grain size distribution curve of the base soil material, as suggested by Karpoff (1955). According to the NRCS (1994), seven control points are determined in order to specify the filter gradation limits. The criteria to establish these
control points are based on experimental data and, in some cases, supported by subjective rules. The purpose of each control point is described below:

- Control point No. 7 defines the maximum $D_{90}$ based on estimating the minimum $D_{10}$ of the filter. The $D_{90}$ value is intended to minimize segregation during placement and construction. There does not appear to be any data published in support of a relation between $D_{90}$ and $D_{10}$, therefore this rule deemed to be subjective in origin. However, Sutherland et al. (2003) and Shourijeh et al. (2007) studied the phenomenon of segregation during placement and construction.

- Control points No. 5 and 6 are determined by the minimum $D_{5}$ and maximum $D_{100}$ sizes of the filter band, respectively. The evidence in support of these two control points is based on experimental data reported by Karpoff (1955), which established (i) that filter materials should pass the 75 mm (3") sieve in order to minimize segregation and bridging of particles during placement; and, (ii) that filter materials must not have more than 5 per cent minus 0.075 mm (No. 200) screen to prevent excessive movement of fines in the filter. Therefore, the criteria of these two control points are based on experimental data.

- Control points No. 3 and 4 verify the width of the filter band to prevent gap-graded filters based on: (i) the coarse and fine sides of the band should have a coefficient of uniformity $C_u \leq 6$; and, (ii) the width of the band should be such that the ratio of maximum to minimum diameters of $D_{15}$, which are control points No.1 and No.2 respectively, is less
than or equal to 5 for all points on the gradation curve below 60% passing. Using a $C_u$ of 6 or less means that the filter band is a reasonably uniform material, since according to the Unified Soil Classification System (see ASTM D2487-11) a well-graded soil has a $C_u$ greater than 6. No evidence which supports these rules was found in the literature; therefore, these criteria are deemed to be subjective in origin.

- The permeability criterion is based on the minimum $D_{15}$ and is defined by the control point No. 2. This empirical rule, first defined by Terzaghi (1939), was tested by many filter studies such as Bertram (1940), Karpoff (1955), Waterways Experiment Station (WES 1942, 1948 and 1953) and United States Bureau of Reclamation (USBR 1947 and 1973) to mention just some of the most relevant studies. Therefore, this criterion was an empirical rule which was tested and supported with experimental data.

- The soil retention or filtering criteria, which is the term used by NRCS (1994), is based on the maximum $d_{85}$ and defined by control point No. 1: it is by far the most tested of all criteria due to its importance. The criterion addresses four categories of base soil, which are based on percentage passing the No. 200 (0.075 mm). Three of the categories [Category 1, 2 and 3 of the NRCS (1994) for base soils with percentage passing the No. 200 (0.075 mm) larger than 15% (finer base soils)] arise from only one study. This extensive laboratory study was performed by the United States Department of Agriculture, Soil Conservation Service from 1981 to 1985. Results and conclusions of this study were presented in Sherard et al. (1984a), Sherard et al. (1984b), and Sherard
and Dunnigan (1989). Since that time, only a few refinements have been incorporated into the filter design guidelines for these three categories, which are summarized in McCook and Talbot (2008). In contrast, the soil retention or filtering criterion for the fourth category [Category 4 of the NRCS (1994) with percentage passing the No. 200 (0.075 mm) smaller than 15% (coarser base soils such as sand and gravels)] has been tested for more than 70 years since it was proposed by Terzaghi (1939). Most of the experimental studies focused on this criterion agreed that a margin of safety exists and it is believed conservative:

\[
\frac{D_{15}}{d_{85}} \leq 4
\]

(2.1)

The studies on this criterion had different and non-standardized test methods. Consequently the inherent nature of the margin of safety is not well-understood.

Given the specific objectives of this study, the evolution of design practice governing the \( D_{15}/d_{85} \leq 4 \) criterion for Category 4 base-soil of the NRCS (1994) is now reviewed in detail, with emphasis placed on test devices and experimental findings.

### 2.1 Test devices used in previous filter compatibility studies

The main features of the test devices used for filter compatibility experimental studies are described chronologically in this section. Only rigid-wall permeameters were used on these investigations. In contrast, some test devices that have been used to investigate geotextile filter compatibility, and also the phenomenon of seepage-induced internal instability, which are not
part of the scope of this study, exhibit an evolution of test devices from rigid to flexible wall permeameters. Thus, it was also considered important to review studies where flexible wall permeameters have been used, because some of the findings influenced the design and operation of the test device used in this study.

Bertram (1940) conducted an experimental study to verify Terzaghi’s (1939) filter design criteria for uniformly graded base soils. The test specimens were 60 mm in height, with a diameter which was 50 or 100 mm. No surcharge load was applied to the specimens, which were placed inside a rigid-wall cylinder made of Lucite. This constant head test device was able to impose unidirectional seepage flow across the specimen in either the downward or upward direction. Test duration, where specimens were subjected to constant hydraulic gradients, was either 2 hours \( (i = 18 \text{ to } 20) \) or 4 hours \( (i = 6 \text{ to } 8) \). This study showed evidence that the use of de-aired and distilled water is essential to eliminate dissolved air and suspended particles, respectively. Furthermore, it is inferred from inspection of the drawings of the device that the difference of total head was measured based on the position of the inflow and outflow tanks; therefore, energy losses of the system may not have been taken into account, and hence reported values of hydraulic gradient across the specimen may be relatively larger than it actually experienced. Flow measurements were recorded at discrete intervals, and any mass loss of the base soil through the filter was collected only at the end of test. No axial deformation measurements were reported.
Over the next 40 years, many test devices with similar characteristics were used in filter compatibility studies, most notably those of Newton and Hurley (1940) and Karpoff (1955). In contrast to these studies, a novel feature of measuring the head loss across the specimen by means of piezometers located at various points along the rigid-wall permeameter was developed by WES (1941), and used in WES (1953 and 1987). Furthermore, as reported in WES (1953 and 1987), the control of seepage flow by applying increments of hydraulic gradient across the specimen in various time steps yielded a multistage seepage flow test procedure.

A novel contribution of Lafleur (1984) was to develop and use a flexible wall permeameter to evaluate filter compatibility in broadly-graded soils for the James Bay project. The test specimens were 150 mm in diameter, with a length of 150 mm (base soil) and 200 mm (filter). In order to ensure saturation of the soils tested, a back pressure of approximately 800 kPa was applied, and then a maximum effective stress of 100 kPa was maintained over the duration of the tests. Downward seepage flow was imposed by maintaining a constant difference in total head. A value of hydraulic gradient up to 8 was applied for a total test duration varying between 50 and 880 h. Although the author mentioned that the hydraulic system of the test device was designed to reduce head losses to a minimum, it was not reported if the difference in total head used to calculate the hydraulic gradient took into account those head losses. Permeability values were calculated for the base soil-filter system at different time intervals throughout the test. Migration of base soil particles through the filter was quantified only at the end of the test.
Between 1981 and 1985, the Soil Conservation Service (now Natural Resources Conservation Service (NRCS) conducted a series of tests to improve the understanding of soil retention criteria. Basically, only one rigid-wall permeameter was used in testing, with a diameter of 100 mm. Variations in the test procedure, using this test device, yielded the slot test (Sherard et al., 1984a), the slurry test (Sherard et al., 1984b) and the No Erosion Filter Test (NEF) (Sherard and Dunnigan, 1989). The test specimens had a length between 50 and 100 mm (base soil) and 125 to 175 mm (filter). Tests were performed without applying any surcharge load. In all tests, the pressure of the water supply system was used to impose a unidirectional seepage flow in downward direction for about 5 to 10 minutes; however, values of hydraulic gradient were not measured. For most tests, water from the local supply system was used; but for tests on dispersive clays, distilled water was used. No measurement was taken of flow rate, so it was not possible to estimate permeability values of the base soil-filter system.

A systematic filter study was carried out by Tomlinson and Vaid (2000) using a rigid-wall test device with capacity for axial loading. Test specimens were 100 mm in diameter, with a length which varied between 20 to 30 mm for the base soil, and 37 mm for the filter. In contrast to previous experimental studies on soil, artificial glass bead materials were used in this study. Specimens were consolidated under a constant vertical stress between 50 and 400 kPa, and then subjected to multistage seepage flow in the downward direction. This was accomplished by maintaining a difference in total head. The study did not report if head loss across the specimen was taken into account in the determination of hydraulic gradient, however, the values reported varied approximately between 0 and 60. Water from the water supply system was used for
testing. As part of the systematic methodology that the authors proposed, the values of flow rate and mass loss from the base soil were recorded at discrete intervals over the duration of a test.

2.2 Select companion studies of instability and filtration phenomena

Other test devices have been developed to investigate companion aspects of soil retention criteria, such as the phenomenon of internal stability. Kenny and Lau (1985) used a rigid-wall permeameter to define the threshold between stable and unstable soil gradations. Skempton and Brogan (1994) tested well-graded and gap-graded sandy gravels to verify the Kenny and Lau (1985) criterion for internal stability: a rigid-wall permeameter was used, with standpipe piezometers that were embedded in the specimen at different elevations in order to determine head losses. An ASTM Gradient Ratio test device was used by Moffat (2002) to assess the potential for internal stability, where specimens were subjected to a vertical stress of 25 kPa and to a multistage seepage flow in the downward direction. Bendahmane et al. (2008) developed three modified triaxial cells to perform a parametric study of suffusion and backward erosion under isotropic stress conditions. In this study, a photo sensor was placed below the specimen, so that real-time measurements of eroded materials could be taken. The piping potential of both cohesionless and cohesive materials was investigated by Richards and Reddy (2010), using a true triaxial system. A stress-controlled test device was used by Chang and Zhang (2011) to investigate the onset and progression of suffusion. The eroded soil and the flow rate were measured to estimate the erosion rate and the variation of the permeability during the erosion process.
Similar devices as previously described have also been used to study geotextile filters as an alternative to granular soil filters. Fisher (1994) developed a long-term Gradient Ratio filtration test device to evaluate different combinations of base soil and geotextile filter. Harney (2001) developed and use a flexible wall Gradient Ratio test device that was able to measure head loss at various elevations along the soil/geotextile specimen, as well as values of permeability of the specimen.

In summary, and with reference to all filter studies, various configurations of rigid-wall permeameter have been used to investigate base soil-filter compatibility. In more recent work there is a tendency to use flexible wall permeameters, and to start apply an isotropic confining stress to the test specimen. An improved understanding of the energy losses in the system is also reported. Moreover, incremental hydraulic gradients tend to be used, instead of imposing a constant hydraulic gradient over the duration of the test yet, in spite of these improvements to the test device, the test method remains non-standardized and, in general, testing conditions differ from study to study. Therefore, the systematic test procedure developed by Tomlinson and Vaid (2000) is followed in this study, using a flexible wall permeameter that was designed and commissioned specifically for this work.

2.3 Experimental findings from previous filter compatibility studies

Filter compatibility studies began after Terzaghi (1939) proposed the soil retention criteria (see Equation 2.1) for uniform sand and gravels as base soil materials. Subsequent studies, such as Bertram (1940), Karpoff (1955) and Sherard et al. (1984a) tested similar uniform cohesionless
materials as base soil to validate the criterion proposed by Terzaghi (1939); a few broadly-graded uniform base soils were included in these studies as well. In contrast, Lafleur (1984) and Lafleur et al. (1989) focused only on broadly-graded base soil materials. A brief description of how the findings of these experimental studies have influenced current design guidelines for granular filters is now presented.

Bertram (1940) performed 30 tests in his experimental study, using Ottawa sand (a rounded sand) and crushed quartz (an angular sand). A total of 6 uniform base soils were examined. It was observed that any filter incompatibility started at the beginning of the test and that the movement ceased within three to five minutes. Interpretation of the test data established that the minimum critical grain size ratio $D_{15}/d_{85}$ at the limit of stability is approximately 6; however, it was observed that only 2 out of 30 tests showed a grain size ratio $D_{15}/d_{85}$ of 6.5 at the limit of stability, and the other 28 tests showed grain size ratios $D_{15}/d_{85}$ greater or equal to 8.7 at the limit of stability. Those 2 tests were performed at hydraulic gradient between 18 and 20, and crushed quartz and Ottawa sand were used as a base soil and filter layer, respectively. Two additional tests were carried out using a well-graded base soil and filter materials. Results of these two tests did not permit definitive conclusions regarding the critical grain size ratio.

Karpoff (1955) performed 25 tests on sand and gravels to determine suitable criteria for filter design. A total of 13 tests used a uniformly graded filter, and 12 used a broadly-graded filter. A total of 5 base soil materials (uniform sands and well-graded silts) were examined in the study.
There were three main contributions that were adopted as part of the filtration criteria in the current filter design guidelines; these are as follows:

- “The filter material should pass the 75 mm (3”) screen for minimizing particle segregation and bridging during placement. Also filters must not have more than 5 per cent minus 0.075 mm (No. 200) particles to prevent excessive movement of fines in the filter and into drainage pipes causing clogging.”

- “The gradation curves of the filter and the base material should be approximately parallel in the range of finer sizes, because the stability and proper function of protective filters depend upon skewness of the gradation curve of the filter toward the fines, giving a support to the fines in the base material.”

- “In designing of filters for base materials containing particles larger than 4.75 mm (No. 4) size the base material should be analyzed on the basis of the gradation of material smaller than No. 4 size.”

Karpoff (1955) recommended additional rules for the grain size ratio of $D_{50}/d_{50}$ and $D_{15}/d_{15}$ for uniform and broadly-graded soils. Interestingly, he tested only one broadly-graded base soil and filter specimen with a grain size ratio of $D_{15}/d_{85} = 4$, which showed filter incompatibility.

In contrast with previous studies, Lafleur (1984) used only 3 broadly-graded cohesionless tills as the base soil to perform 9 tests to establish the margin of safety associated with filter criterion. Since this laboratory testing on the James Bay project was intended to reproduce field conditions, “… the specimens were compacted to in situ minimum density conditions; the James Bay specifications were 97% of the standard Proctor maximum dry density for the till core and
70% relative density for the filters.” Laboratory data analysis confirmed the recommendation given by Karpoff (1955) for soil retention criteria regarding the use of particles smaller than the 4.75 mm (No. 4) size for gravelly base soil materials. Furthermore, it was found that the critical piping ratio (or grain size ratio) $D_{15}/d_{85}$ for the onset of failure was about 8.4. This value is in reasonable agreement with that obtained by Bertram (1940) for uniformly graded soils, which tends to confirm the margin of safety for the soil retention rule of $D_{15}/d_{85} \leq 4$ proposed by Terzaghi (1939).

Sherard et al. (1984a) intended to validate Bertram’s (1940) recommendations and also to improve the understanding of sand and gravels filters [Category 4 of the NRCS (1994)]. The total number of tests performed was not reported, however, in the first phase of the laboratory experiments, it is believed that 16 uniform base soils were used. Materials tested were described as uniform fine, medium or coarse sand for the base soils, and uniform gradation coarse sand, uniform gravel, or well-graded sandy gravel for their filter materials. Findings presented in this study were that:

- “… for filters with $D_{15}$ of larger than about 1.0 mm, the ratio $D_{15}/d_{85} \leq 5$ should be continued as the main criterion for judging filters acceptability.”

- “Filter criteria based on the $D_{50}/d_{50}$ of $D_{15}/d_{15}$ ratios are not founded on a sound theoretical or experimental basis and should be abandoned.”

- “It is not necessary that the particle size distribution curve of the filter be generally similar in shape to that of the base soil.”
As part of this work, Sherard et al. (1984b) performed a total of 254 tests in order to expand the soil retention rule for fine-grained clays and silts as base soils [Category 1, 2 and 3 of the NRCS (1994)]. Thirty six different base soil materials were used. It is inferred that 11 of these 36 base soils were relatively widely-graded sandy silts and sandy clays; and the other 25 were uniform fine-grained clay and silts with appreciable cohesion. Filter materials were fairly uniform to moderately-graded sand and gravel. All the results from the tests using the relatively widely-graded base soils showed that the failure-success boundary was at a grain size ratio $D_{15}/d_{85}$ larger than 9. Failure was defined as “a significant quantity of base material passed through the filter in the first 60 sec of flow, and continued at about the same rate.” Moreover, Sherard et al. (1984a) concluded that, “for the well-graded base soils tested (sandy silts and clays and well-graded pervious sands with $d_{85}$ of 1.2-1.9 mm), a filter sufficiently fine to catch the $d_{85}$ size will also catch the finer base particles. Thus, it can be concluded that, for the soils tested, the oldest and most widely used filter criterion, $D_{15}/d_{85} \leq 4$ or 5, is conservative. It can be considered to have a safety factor of about 2, since the experimental failure-success boundary is about $D_{15}/d_{85} = 9$.”

As part of the same study, Sherard and Dunnigan (1989) performed No Erosion Filter (NEF) tests, which examined filters exposed to concentrate leaks in the base soil. The base soils were a wide range of different fine silts and clays and clayey and silty sands. In these tests, permeability of the base soil-filter system was not evaluated, since the main objective was to investigate the role of the filter in preventing erosion. The main contribution of this companion testing was to suggest filter criteria rules for the entire range of base soils that can be used in earth dams. These
base soils were divided into four categories based on the percentage of base soil passing the No. 200 (0.075 mm) sieve. The filter criteria for these four categories are as follows:

- \(D_{15} \leq 9 \times d_{85}\), but not smaller than 0.2 mm (85-100% of base soil passing the No. 200 sieve)
- \(D_{15} = 0.7\) mm (40-80% of base soil passing the No. 200 sieve)
- \(D_{15} \leq (40 - A/40 - 15)(4 \times d_{85} - 0.7\) mm) + 0.7 mm, where \(A\) is the percentage of the base soil passing the No. 200 (0.075-mm) sieve after any regrading (15-40% of base soil passing the No. 200 sieve)
- \(D_{15} \leq 4 \times d_{85}\) (0-15% of base soil passing the No. 200 sieve)

These filtering criteria with a few refinements, which are detailed in McCook and Talbot (2008), support the current guidelines for filter design. As mentioned before, the study of the last filtering criterion \(D_{15} \leq 4 \times d_{85}\) is the subject of this study.

A select number of other studies that have contributed to a better understanding of the filter phenomenon are worth mentioning. For example, Indraratna and Locke (2000) used a mathematical model to describe the time-dependent changes of porosity, permeability, and particle retention capacity of a filter. Foster and Fell (2001) presented an embankment dam assessment for filters that do not satisfy design criteria based on analysis of previous laboratory results. Tomlinson and Vaid (2000) proposed a novel systematic methodology to evaluate filter compatibility. They used uniform artificial glass beads to represent various base soils and filter combinations to determine the critical hydraulic gradient at which the base soil erodes through the filter. Five different base soil materials were used to perform 17 tests with a grain size ratio
$D_{15}/d_{85}$ between 7.3 and 12.3. Various combinations of surcharge loads, hydraulic gradient, rate of hydraulic gradient increase and filter thickness were examined in testing. It was found that base soil-filter specimens with a ratio $D_{15}/d_{85} < 8$ did not fail, which is consistent with what Bertram (1940), Lafleur (1984) and Sherard et al. (1984a and 1984b) found. Specimens with a ratio $D_{15}/d_{85} > 12$, exhibited a spontaneous failure. Specimens with a ratio $D_{15}/d_{85}$ between 8 and 12 failed if a critical gradient was exceeded. In this study, failure is defined as the point at which “the erosion becomes continuous when piping is occurring. This instant is regarded herein as the commencement of piping erosion and the associated gradient as the critical gradient.” Therefore, it was concluded that, the higher the ratio $D_{15}/d_{85}$, the lower the critical hydraulic gradient to trigger piping, and also that the critical hydraulic gradient decreased with increase in surcharge load.

In summary, a brief description has been provided of the key experimental studies which have influenced current design guidelines for granular filter. In reality these design guidelines are founded on four experimental studies, namely those of Bertram (1940), Karpoff (1955), Lafleur (1984) and Sherard et al. (1984a, 1984b and 1989). In total, 39 different base soils, comprising of fine-grained clay and silt materials (25 uniform by graded soils materials, and 14 widely-graded soils), were tested by Sherard et al. (1984b) and Lafleur (1984) to support the filter criteria for base soils with a percentage passing the No. 200 (0.075 mm) greater than 15% [Category 1, 2 and 3 of the (NRCS, 1994)]. Otherwise, 29 types of sand and gravels materials were used as base soils to support the filter criteria for base soils with a percentage passing the No. 200 (0.075 mm) less than 15% [Category 4 of the (NRCS, 1994)]. Only 5 out of these 29 base soils were broadly-
graded base soils; the other 24 were uniform sand and gravels. The experimental studies which used these sand and gravels as base soils were Bertram (1940), Karpoff (1955) and Sherard (1984a). Accordingly, the current design guidelines for granular filters are supported, in general, by experimental studies that used in total 68 different types of base soil gradations.

2.4 Seepage flow regime

The objective of this section is to examine seepage flow theories relating to through porous media, in order to provide a framework for interpretation of laboratory test results found in this study. The applicability of Darcy’s law, which is valid during laminar flow conditions, is described, together with some exceptions to this empirical law. Studies are reviewed to support the understanding of non-Darcian flow conditions in coarse materials, such as sand and gravels.

It has long been recognized that seepage flow in a porous medium may assume one of two states of motion, namely laminar or semi-turbulent to turbulent flow, illustrated in Figure 2.1. A laminar flow regime is described by Darcy’s law.

The transition from laminar flow to semi-turbulent or turbulent flow may be quantified using the Reynolds number (Re) criterion, which was developed initially for flow in pipes. It was generalized for soils by other investigators, including as Fancher et al. (1933) and Taylor (1948). More specifically, an experimental study performed by Cedergren (1977) defined the concept of “relative” permeability, which is used later on in this study to establish the initiation of semi-
turbulent to turbulent flow in coarse materials. This change in flow regime is triggered by a hydraulic gradient, and is evident when permeability values start to decrease.

2.4.1 Darcian flow conditions

According to Darcy’s Law, the discharge velocity \( v \) or the flow rate \( q \) is linearly proportional to the hydraulic gradient \( i \), as long as the flow is laminar and steady through a saturated porous medium, such as soil. Equations 2.2 and 2.3 are expressions of Darcy’s law.

\[
v = k i \quad (2.2)
\]

\[
q = kiA \quad (2.3)
\]

Permeability is denoted by \( k \) and \( A \) is the cross-section area normal to the direction of the flow. Henry Darcy (1853) developed this empirical relation based on the results of one-dimensional water flow tests through packed clean sands at low velocity. Brown et al. (2003) published details of the development of Darcy’s law, including details of the experimental testing.

Equation 2.2 represents the laminar zone (see Figure 2.1), where there is a linear relation between the hydraulic gradient and the discharge velocity (or velocity); however, this relation is nonlinear after the transition zone is passed. Typically, it has been argued for most soils that the velocity is so small that the seepage flow regime may be assumed laminar and Darcy’s law is applicable. However, the findings of some experiments have revealed that, in clays at very low hydraulic gradients, the relation between velocity and hydraulic gradient is non-linear. Similarly, it has been shown that for coarse materials, such as sand and gravels, Darcy’s law is invalid.
when pore sizes and hence seepage velocities are sufficiently large. For this reason, exceptions to Darcy’s law are described in the next section.

### 2.4.2 Non-Darcian flow conditions

Hansbo (1960) tested clays at very low gradients, and found that Darcy’s law was not valid in these fine soils, since the relation between velocity and hydraulic gradient was non-linear. In contrast, Olsen (1966) suggested that Darcy’s law is meaningful in many saturated natural clays and clayey sediments, however, in extremely fine-grained clays, such as montmorillonite, and also in unconfined shallow sediments with high hydraulic gradients, there can be some exceptions.

Various investigators suggested that, if the flow rates in granular materials are sufficiently great, then Darcy’s law no longer is valid. That is the case of the experimental studies performed by Lindquist (1933), Hubbert (1956) and Scheidegger (1960), where findings suggested that, if a flow rate increases, the pressure drop is no longer linearly proportional to the velocity, so Darcy’s law is not valid. A good summary of studies related to flow conditions in coarse gravel and rock was performed by Leps (1973). This study also presented formulas for estimating flow velocities in clean gravel or rocks. Holtz et al. (1981, pp. 205) mentioned that “in very clean gravel and open-graded rock fills, flow may be turbulent and Darcy’s law would be invalid”. Hansen et al. (1995) evaluated one-dimensional non-Darcy flow in rockfill materials. Li et al. (1998) published a summary of various studies on non-Darcy flow in rockfill materials.
Therefore, there is a significant body of evidence to support the claim that non-linear flow conditions may occur in coarse-grained soil.

Mitchell et al (2005, pp. 256-258) summarize several studies, mainly focused on fine materials, about the validity of Darcy’s law, that leading to the conclusion “…Darcy’s law is valid, provided that all system variables are held constant.” However, laboratory data reported in this study (see Chapter 5) suggest that for uniformly-graded coarse-grained soil there is a hydraulic gradient at which the flow regimen changed from laminar to semi-turbulent to turbulent flow, in spite of all variables being held constant. Accordingly, it is important to establish a criterion to identify the transition from laminar to semi-turbulent to turbulent flow in order to corroborate the test results. Hence, the Reynolds number criterion is described in the following section.

2.4.3 Criterion to determine flow regimen transition

Experiments performed by Osborne Reynolds determined that the transition from laminar to semi-turbulent to turbulent flow can be quantified using the Reynolds number (Re). This criterion, which was developed initially for flow in pipes, established that velocity is inversely proportional to the pipe’s diameter; therefore, if the velocity is less than a critical value (Re ≤ 2000), the flow is considered laminar. This well-known critical value does not apply to porous media where flow is tortuous. Eventually, the Reynolds number criterion was applied to porous media, such as soils, by other investigators. Fancher et al. (1933) gave a rough but satisfactory criterion of the limit of applicability of Darcy’s law, where the Reynolds number (Re) of “2000” was changed to “1” to be applicable in soils. Therefore, a rough but generally recognized
criterion to identify the change in flow regime from laminar to semi-turbulent to turbulent flow is defined by the following expression:

$$R_e = 1 \geq \frac{vD_a\gamma_w}{\mu g}$$

(2.4)

Where the diameter of the sphere is $D_a$, the unit weight of water is $\gamma_w$, the viscosity is $\mu$, and the gravity is $g$. Taylor (1948) merged the Reynolds and Fancher et al. (1933) studies to set a range of validity of Darcy’s law and claimed that “in soils there is a slow transition from purely laminar flow to mildly turbulent condition, which precludes an accurate expression for the critical point but which also makes a precise expression unnecessary”. Taylor (1948) also admitted that the Reynolds number of unity (Re = 1) in Equation 2.4 is an approximate value and has been conservatively chosen; and also that “its approximate nature is proved by the absence of any account of the void ratio, which surely should have some effect on the relationship.”

In further examinations, different values of Reynolds number were considered to determine the initiation of semi-turbulent flow in porous media. Consider for example Bear (1972), where experimental evidence showed that Darcy’s law is valid as long as the Reynolds number does not exceed some value between 1 and 10. A recent study by Giroud et al. (2012) corroborated that the value of Re $\approx$ 1 proposed by Taylor (1948) should be in fact a value between 0.4 and 0.8. They also suggested that values proposed by Bear (1972) should be reduced to a range varying between 0.5 and 5. In spite of the lack of agreement on the value at which semi-turbulent flow is initiated, it seems that most of studies agreed there is a transition from laminar to semi-turbulent flow. Proof of that is the suggestion for a range to denote the Reynolds number. Taylor (1948)
claimed that the change from laminar to semi-turbulent flow is a slow transition which makes a
precise expression unnecessary. Therefore, in this study, a Reynolds number greater than 1 (Re ≥ 1) will be considered an indicator that the flow has changed from laminar to semi-turbulent flow.

In a companion path of investigation, Cedergren (1977) conducted a series of experiments in a
small flume using open-graded American River (California) crushed river gravels with no fines. He performed permeability tests for a wide range of hydraulic gradients, so it was possible to have nearly laminar flow conditions at small gradients and some turbulence in the flow at larger gradients. Then, he adapted Darcy’s law to semi-turbulent to turbulent flow conditions, in which an “effective” permeability definition was postulated:

\[
q = k' i A = (kC) i A
\]

(2.5)

In this expression, \(k\), \(i\) and \(A\) were previously defined for laminar flow (see Equation 2.2 and 2.3); \(C\) is an experimental factor that varies with hydraulic gradient, and \(k'\) is the “effective” permeability, for semi-turbulent to turbulent flow. Analysis of the results found that the “effective” permeability for semi-turbulent to turbulent flow is related to the laminar (or nearly laminar) Darcy coefficient, so the “relative” permeability \(C = (k' / k)\) approach was defined. Moreover, it was found that, if flow is laminar, there should be no reduction in “effective” permeability - in other words, \(C = 1\). In contrast, if there is flow at 100% turbulence, the larger is the hydraulic gradient, the smaller is the “effective” permeability - in other words, \(C << 1\). For intermediate cases, there is a certain hydraulic gradient at which the flow regime changes from laminar to semi-turbulent to turbulent flow. Figure 2.2 shows a rough guide for estimating values
of “relative” $C$ to be used in seepage quantity and velocity computations for semi-turbulent to turbulent flow.

In summary, seepage flow regime theory has been reviewed in this section, with special attention given to non-Darcian flow. The Reynolds number criterion was described to identify the transition between laminar to semi-turbulent to turbulent flow. This is particularly important for this study, because it may have an impact on the linear relation between the hydraulic gradient and the velocity, so that permeability is affected. Laboratory data of this study (see Chapter 5) exhibit this tendency, regardless of whether all system variables are held constant.

2.5 The research need

Base soil-filter compatibility has typically been evaluated using a rigid-wall permeameter with no control of the stress conditions, and with seepage flow induced across the specimen using a constant hydraulic gradient. Moreover, in the absence of a standardized test method, the procedures used have differed from study to study. Indeed, only in the study performed by Tomlinson and Vaid (2000) was a systematic procedure developed wherein the base soil-filter specimen was first consolidated under axial loading and then subjected to multistage seepage flow induced by an incremental hydraulic gradient. Therefore, it was considered appropriate to continue the investigation of base soil-filter compatibility using the same approach developed by Tomlinson and Vaid (2000), but using an improved stress-controlled device, where isotropic confining stress is applied to consolidate the specimen, which is then subject to multistage seepage flow.
A common aspect of previous base soil-filter compatibility studies is that the onset of filter incompatibility is defined qualitatively, based on the interpretation of only one select test parameter, and that parameter varied from study to study. The intent of this study is to postulate a definition of the onset of filter incompatibility in both a qualitative and quantitative manner, based on a unified interpretation of two independent test parameters. Then, stress-dependency of the onset of filter incompatibility is considered with reference to this unified interpretation, in order to advance upon the insights of Tomlinson and Vaid (2000).

Finally, most of the experimental studies reported in the literature have focused on the soil retention criterion for uniformly-graded materials, yielding a consensus that a margin of safety exists and it is believed conservative. Therefore, a further intent of this study is to obtain a good and reliable series of laboratory test data over a range of stress, and confirm that the margin of safety is meaningful.
Figure 2.1: Zones of laminar and turbulent flow (after Taylor, 1948)

Figure 2.2: Rough guide for estimating reduction in $k$ of coarse aggregate caused by turbulence (Cedergren, 1977)
Chapter 3: The triaxial permeameter test device

3.1 Introduction

To investigate the onset of filter incompatibility under an isotropic stress state, a new stress-controlled flexible wall permeameter was developed. This test device, which is hereafter called “triaxial permeameter”, is an improved version of previous permeameters and was designed and fabricated at the University of British Columbia (UBC). The main objective of this new test device is to have a stress-controlled testing apparatus, which can be used to systematically study the onset of filter incompatibility subjected to multistage seepage flow. Additionally, it was necessary to use a companion rigid-wall permeameter for some tests because difficulties were encountered during reconstitution of some filter incompatible specimens (see Section 5.4). A description of this companion test device is supplied later in this chapter.

3.2 Test device design objectives

The main purpose of this new triaxial permeameter is to overcome some of the challenges encountered using previous test devices, so that better quality data can be obtained. A schematic diagram of the overall you of the triaxial permeameter is shown Figure 3.1. The new test device can:

- achieve isotropic consolidation of the test specimen
- measure volumetric change during specimen consolidation
- impose a unidirectional seepage flow across the specimen
- continuously monitor the confining stress, head loss and axial deformation
• measure flow rate and mass loss at discrete intervals over the duration of a test

A description of these design objectives is given in the following sections of this chapter.

3.3 Triaxial permeameter device description

3.3.1 Triaxial chamber

The triaxial chamber is capable of withstanding the confining pressures applied in this study, and it comprises a top plate and a base frame separated by the chamber itself, which is a transparent acrylic tube. The length of this tube is 44 cm, with an internal diameter of 23 cm and a wall thickness of 1.3 cm. The triaxial chamber is sealed using mounted rubber O-rings on the top plate and base frame. Once these three components are assembled, six rods are tied to the top plate, to provide the required compression and to seal the chamber.

Different ports are located in the base frame as shown in Figure 3.2 and Photo A1 in Appendix A. The cell port supplies pressurized de-aired water to the chamber. It is connected to a total pressure transducer (TPT) and to an air/water interface cylinder, which is connected to a regulated air supply. The inlet port connects one end of the differential pressure transducer (DPT) and a pipette with the top cap. The outlet port connects the base pedestal to the other end of the DPT. The inlet supplies de-aired water to the top cap using two flexible tubes.

The test set up was designed to impose only seepage flow in the downward direction; therefore, after the inflow passes through the specimen, it exits the triaxial chamber through the bottom drain located in the base frame (see Figure 3.1).


3.3.2 Top cap, base pedestal and membrane

The top cap and the base pedestal are located over and under the test specimen, respectively, and were manufactured by the workshop at UBC Civil Engineering. Both of them are made of aluminum and have the same diameter as the specimen; they were designed to provide drainage from either end of the specimen, however, only downward flow was imposed through the specimen.

3.3.2.1 Top cap

Three fittings are located in the top cap; two of them enable the inflow from the water supply system and are called inlets. The third fitting is called the inlet port (see Figure 3.3) and is used to provide drainage when the specimen is consolidated, and also to measure head losses across the specimen when seepage flow is imposed. The loading ram is not fixed to the top cap, since a stainless steel ball is used to couple it with the top cap.

A perforated plate with a hole diameter of 5 mm on 5 mm centre-to centre spacing is assembled inside the top cap. There is a 5 mm gap deep chamber inside between the perforated plate and the top cap to help achieve better flow distribution. A series of three wire mesh screens are located underneath the perforate plate: they have a sequential opening size of 1.14, 0.035 and 0.61 mm in the direction of downstream flow. The 0.035 mm wire mesh prevents that any grains from entering the top cap during consolidation. The wire mesh screen with a screen opening of 0.61
mm was used only to hold 0.035 mm wire mesh screen. This wire mesh screens arrangement was used for all the tests in the triaxial permeameter.

3.3.2.2 Base pedestal

The base pedestal is sealed to the base frame using four tie rods. O-rings prevent leakage in the contact between the base pedestal and the base frame. Two wire meshes with screen openings of 1.14 and 2 mm are placed in contact with the specimen: they mount in a perforated plate with 5 mm diameter holes on 5 mm centre-to centre spacing (see Figure 3.3). It is important to mention that the objective of these wire mesh screens is to retain the grains of the filter and to let the grains of the base soil pass through. Inside the base pedestal, a permeable filter prevents any washed out particles from entering in the outlet port (see Figure 3.1)

With a diameter of 1 cm the bottom drain of the base frame is concentric, and is connected to the base pedestal so that seepage flow can pass through unimpeded. During specimen reconstitution and consolidation, it is closed with a bottom plug that is subsequently opened just before seepage flow is imposed.

3.3.2.3 Membrane

A 4.0”x0.12”x12” flexible membrane manufactured by ELE International was used to encase the specimen and provide reliable protection against leakage. The membrane is sealed to the top cap and base pedestal with O-rings. It is important to mention that some difficulties were encountered at the base soil-filter interface during multistage seepage flow. These difficulties
were related to have large deformation at the interface due to base soil particles migration. As a consequence of these large deformations at the interface, the membrane burst during some tests. Therefore, a coating of silicone adhesive sealant was used (Vaid, 2012), which is called Superflex Clear RTV Silicone and is manufactured by Henkel/Loctite USA (see Photo A2 in Appendix A). This coating was used to provide extra reinforcement of the membrane and to reduce deformations at the interface without compromising the integrity of the specimen.

3.3.3 Water supply system

The water supply system comprises two main components, which are a two-stage filter system and a water de-airing system. Both components act to provide clean and de-aired water, from the laboratory tap water, to the triaxial permeameter, as explained below.

In the first stage of the two-stage filter system, the tap water passes through a sand filter, type ZWMULTIMD and manufactured by Millipore, where all particles larger than 10 μm are removed. Then, in the second stage, the water passes through a carbon filter, model P72017, manufactured by Millipore. In this stage, all particles larger than 3 μm are removed. Therefore, clean water is obtained.

Afterwards, the clean water is stored in a water tank to start the de-airing process. The water tank was manufactured by John Wood and has a capacity of 270 litres. Before the tank is filled with clean water, a vacuum pressure not less than -70 kPa is imposed. Then, after the vacuum regulator indicates a stable vacuum pressure, the water tank is filled with clean water coming
from the two-stage filter system. Thereafter, the vacuum pressure is applied on the water tank for
not less than 24 hours in order to reach values close to 2.5 mg/L of dissolved oxygen content on
the de-aired water.

**3.3.4 Seepage control system**

The seepage flow induced across the specimen is controlled by means of an inflow and outflow
constant head device. The inflow constant head device (inflow CHD) is mounted on a jack stand
(adjustable) and the outflow constant head device (outflow CHD) is a stationary tank, in which
the triaxial permeameter is placed (see Figure 3.1). The seepage control system is a closed water
circuit, where the difference in total head \(\Delta h_t\) between the inflow and the outflow CHD
induces the water to flow from the inflow CHD to the inlet in the triaxial permeameter, then
across the specimen. Finally, the water passes through the outlet of the outflow CHD to the
reservoir. De-aired water is then recirculated by pumping it up to the inflow CHD. The
maximum possible height difference that can be achieved between the inflow and outflow CHD
is 184 cm: it is constrained by the head-space of the geotechnical laboratory, for conditions of
gravity-flow.

**3.3.5 Mass loss collection system**

The mass loss collection system consists of a collection trough located underneath the base frame
(see Figure 3.1). This collection trough comprises two removable pans mounted on a frame,
which rotate around one of the legs of the base frame, to bring a pan into position below the
bottom drain. When in position, there is a gap of less than 2 mm between the pan and the base
frame, which permits seepage water to flow into the outflow CHD while also causing any base soil particle that passes the lower wire mesh screens to settle in the pan. Photo A1 in Appendix A shows a photo of the collection trough.

This arrangement ensures a continual collection of washed-out particles. At any stage of the test, one of the two pans can be removed, its content emptied, and replaced without interference to the seepage flow. The contents of the removed pan are placed in an oven to be dried at a temperature of 110 ± 5° (ASTM D2216 - 10) and then weighed.

### 3.3.6 Instrumentation

Two pressure transducers and one Linear Variable Differential Transformer (LVDT) are used to continuously record data in a test, during the process of specimen consolidation and also during multistage seepage flow. One pipette is used to measure volume change during specimen consolidation.

#### 3.3.6.1 Confining Stress

The confining stress applied inside the triaxial chamber is controlled by the air/water interface cylinder connected to the air pressure supply by a pressure regulator. The air/water interface cylinder is partially filled with de-aired water and is used to pressurize the triaxial chamber, with measurement using total pressure transducer (TPT) (see Figures 3.1 and 3.2). The model of the total pressure transducer is PDCR 130/w/c and it was manufactured by Druck. Its pressure range is between 0 and 700 kPa, and its measurement resolution is ± 0.1 kPa.
3.3.6.2 Axial deformation

The length of the specimen changes in response to consolidation and induced seepage flow. The length achieved during specimen reconstitution is measured using a height measuring device as described in Appendix B; the axial deformation due to consolidation and induced seepage flow is recorded by a Linear Variable Differential Transformer (LVDT), which is mounted to the loading ram, as shown in Figure 3.1. The model of the LVDT is TS 25 and it was manufactured by Novotechnik. Its capacity is 25 mm and its measurement resolution is ± 0.1 mm.

3.3.6.3 Head loss

A difference in total head \(\Delta h_t\) between the inflow and outflow CHD induces the seepage flow across the specimen (see Figure 3.1), yielding an hydraulic gradient across the specimen. Due to energy losses when the flow passes through the tubing and fittings of the triaxial permeameter, the actual difference in total head across the specimen is less than \(\Delta h_t\). Therefore, in order to have reliable measurements of head loss across the specimen\(\Delta h_s\), a differential pressure transducer (DPT) is connected to the top cap through the inlet port, and to the base pedestal through the outlet port (see Figures 3.2 and 3.3 for details). The model of the DPT is PDW/E972-05-01 and it was manufactured by Sensotec. Its has a range between -70 and 70 kPa, and its measurement resolution is ± 0.1 kPa.
3.3.6.4 Volume Change

A graduated pipette is connected to the inlet port to measure any volume change when the specimen is subjected to consolidation. This volume change is used to calculate the void ratio of the specimen. Details of the void ratio calculation are presented in Appendix C: The upper end of the pipette is connected to the vacuum pressure supply by a vacuum regulator. The pipette has a capacity of approximately 90 cm³, with an inside diameter of 1.3 cm, and a length of 70 cm. The volume change of the specimen is determined using readings taken on the graduated pipette. The measurement resolution of the pipette is ± 0.1 cm.

3.3.6.5 Flow rate

In order to deduce the permeability of the test specimen, it is necessary to measure the flow rate. This flow is manually measured with the aid of a beaker, which has a capacity of 2 litres, by intercepting the overflow at the outlet of the outflow CHD (see Figure 3.1) for a specific time interval, typically a period of approximately one minute. The volume of water collected in the beaker is weighed on an electronic balance. The balance was manufactured by A & D Company, with a capacity of 3100 g and a measurement resolution of ± 0.1 g.

It is important to mention that measurement of flow rate in the early period of testing was erratic and influenced by compliance of the outflow CHD, since the water level in the outflow CHD took approximately 70 minutes to abandon a steady-state, once the seepage flow was imposed. To demonstrate this sort of system compliance, a volume of water of 2100 cm³ coming out to the outflow CHD was estimated, based on an average flow rate for the first 70 minutes of 0.5
cm$^3$/sec. During this period of time, the water level only raises a few millimeters height, knowing that the dimensions of the outflow CHD are 60 by 60 cm.

### 3.3.6.6 Mass loss

After the washed out grains are collected in the collection trough, they are dried and weighed. The weigh scale is the same one used to weigh the water collected for the value of flow rate. Therefore, the measurement resolution of the mass loss is close to ± 0.1g.

### 3.3.6.7 Data acquisition system

The data acquisition system (DAS) records the output voltages of the transducers (TPT and DPT) and the LVDT and stores them in real-time. This system comprises: (1) a signal conditioning unit which amplifies the output signals; (2) a 12-bit resolution DAS board with digital input/output; (3) and a desktop computer with the software LabView Signal Express 2011, version 5.0.0. Using this software, the data are recorded at a frequency of 10 Hz and stored as an output file to generate further plots.

### 3.4 Rigid-wall permeameter

Some difficulties were encountered during the reconstitution of some of the more filter incompatible base soil-filter specimens (see Section 4.5), and it was not possible to test them in the triaxial permeameter. Because of these difficulties, they were instead reconstituted and tested in a rigid-wall permeameter. No seepage flow was imposed in tests using the rigid-wall permeameter. Photo A3 in Appendix A shows the configuration of the rigid-wall permeameter.
The rigid-wall permeameter comprises a plexiglass cylinder, which is mounted on the base pedestal of the triaxial permeameter (see Section 3.3.2.2 for details). The length of the plexiglass cylinder is 15 cm, with an internal diameter of 10 cm and a wall thickness of 0.5 cm. There is an O-ring in the lower part of the cylinder which requires only the slightest pressure to seat securely on the base pedestal. The configuration of the wire meshes and perforated plate is the same as the triaxial permeameter (see Section 3.3.2.2 for details). The bottom drain of the base frame is closed with the bottom plug, since seepage flow is not imposed.

The water used in the rigid-wall permeameter was cleaned and de-aired using the same water supply system employed in the triaxial permeameter (see Section 3.3.3). Mass loss measurements were not recorded precisely during these tests; rather they were qualitatively assessed (see Section 3.3.5) from material amassed inside the base pedestal.

3.5 Summary

Two test devices have been described for this filter compatibility study. One of them was the new stress-controlled test device, named the triaxial permeameter. This equipment was described in detail to investigate the onset of filter incompatibility in specimens that were subjected to multistage seepage flow, after consolidating base soil-filter specimens under isotropic confining stress. A measurement of head loss across the specimen is recorded between the upper and lower part of the specimen, so hydraulic gradient values can be determined with more accuracy. Volume change of specimen due to consolidation is also measured, so that initial a void ratio can
be calculated. All these improvements, which were influenced by a review of experimental
studies (see Section 2.1), are believed to provide for a better quality of test data, thereby
enhancing the confidence in understanding the margin of safety of the filtering criterion
proposed by Terzaghi (1939) (See Equation 2.1) and verified by others. These improvements to
the test device also made it possible to examine the stress-dependency at the onset of filter
compatibility. Difficulties encountered during reconstitution of some specimens were overcome
by the use of a companion test device, a rigid-wall permeameter. The tests performed using the
rigid-wall permeameter are complementary to the tests carried out on the triaxial permeameter.
Figure 3.1: A schematic diagram of the triaxial permeameter
Figure 3.2: A plan view of the base frame

Figure 3.3: Details of the top cap and the upper part of the base pedestal
Chapter 4: Materials and testing procedure

4.1 Introduction

This chapter presents the test materials and test program for this study, and also the procedure to operate the new triaxial permeameter. The specimens used in this study consist of a two-layer system, where the lower layer is called the “filter” and the upper layer is called the “base soil”. Basically, the testing procedure is divided into three main phases: (1) specimen reconstitution; (2) specimen consolidation; and (3) multistage seepage flow. Details of these three phases are presented in the following sections. In addition, the testing procedure for the companion rigid-wall permeameter series of tests is described later in this chapter.

4.2 Test materials

Test specimens were reconstituted using artificial glass beads manufactured by Potter Industries Inc. Their chemical composition is soda-lime silica glass. The reason why these materials were used in this study is the physical characteristics of the grains, and more specifically the roundness and sphericity, which lend themselves to fundamental studies of physical relations. It is important to mention that the term base soil was used in this study, in spite of artificial glass beads materials were used. Photos A4 and A5 in Appendix A depict shape of the glass beads, for which the images were taken by optical the microscope using the same scale.
In total, three size ranges of glass beads were used for the base soil: 0.12 to 0.18 mm, 0.15 to 0.21 mm and 0.25 to 0.30 mm; and five glass beads size ranges were used for the filter: 1.2 to 1.7 mm, 1.4 to 2.0 mm, 2.0 to 2.8 mm, 2.8 to 3.2 mm and 2.1 to 3.3 mm (see Figure 4.1).

4.3 Testing Program

Three different size ranges of base soil materials were matched with five different filter materials in order to yield a wide range of grain size ratio $D_{15}/d_{85}$ between 7.5 and 13.3 (see Table 4.1). Using the triaxial permeameter test device, an isotropic confining stress of 50, 150 and 300 kPa was applied only to the specimens with a grain size ratio $D_{15}/d_{85}$ of 7.5 and 8.7, followed by an induced seepage flow. For the test with a grain size ratio $D_{15}/d_{85}$ larger than 9, the rigid-wall permeameter was used. In all cases, the diameter of the specimen was 100 mm, with a total length of 100 mm, which is divided into two layers (base soil and filter) of 50 mm thickness.

The use of the triaxial permeameter, and consequently the tests performed on this test device are emphasized in this study. The tests performed in the rigid-wall permeameter are considered companion tests, although they are believed important to a confident understanding of the margin of safety in soil retention.

4.4 Test procedure for the triaxial permeameter

In the following sections, details of the operation of the triaxial permeameter are explained.
4.4.1 Preparation of the triaxial permeameter

All components of the triaxial permeameter are disassembled.

- All components are cleaned using pressurized air to remove loose particles, including those used for specimen reconstitution in order to avoid contamination of particles from previous tests.
- Special attention is given to the O-rings located in the top plate, base frame and base pedestal. After the O-rings are cleaned and dried, silicone grease is used to lubricate them.
- The base pedestal is adjusted to the base frame, once these two components are cleaned and lubricated.
- All valves and pressure regulators are also checked in order to verify that they are working correctly.

Saturation of the permeable filter and the finest wire mesh screen is carried out.

- Both are submerged in different beakers, and then boiled. After that, they are left to partially cool and finally are subjected to vacuum (between -70 and -80 kPa) for not less than 12 hours. A supply of water is also prepared using the same procedure. This supply of water is used to purge the fittings and tubes of the triaxial permeameter to avoid any air bubbles on the system.
- De-aired water is also prepared, as mentioned in Section 3.3.3.
4.4.2 Triaxial permeameter assembly

The test device is then placed inside the outflow CHD and located over the collection trough, which should be able to rotate freely around one of the legs of the base frame.

- All valves of the triaxial permeameter are opened.
- The outflow CHD is filled with de-aired water up to a few centimeters above the elevation of the base pedestal.
- The bottom drain in the base frame is closed with the bottom plug.
- The air/water interface cylinder is filled with the water supply prepared the day before, and used to purge air bubbles from all ports (see Figure 3.2).

After there is no evidence of air bubbles in the system, a wet-wet connection is made between all ports and flexible tubes.

- The TPT and the DPT are connected to the cell port, and to the inlet and outlet port, respectively. Connections are also made between the cell port and the air/water interface cylinder, and between the inlet port and the pipette (see Figure 3.2). Then, using a syringe filled with de-aired water, a wet-wet connection is made between one end of a flexible tubing and the inlet port, located inside the triaxial chamber. The other end of the flexible tubing is connected to the top cap later. Thereafter, all valves located in the three ports are closed.
- The inflow CHD is filled with de-aired water and connected to the inlet located in the base frame. Two flexible tubings are also connected to the fitting of the inlet, located inside the triaxial chamber. After that, the inlet valve is opened and water is allowed to
flow through the inlet for some minutes until any entrapped air is flushed out, whereupon the inlet valve is closed.

Once all tubes are connected, the top cap and base pedestal are assembled.

- The permeable filter is inserted, under saturated conditions, into the outlet port located inside the base pedestal.
- The top cap, the perforated top plate and the wire mesh screens are then submerged separately in the outflow CHD, and after all air bubbles have been removed from each component, the perforated top plate and the wire mesh screens are inserted inside the top cap. After that, the top cap is placed inside a submerged container, in which it is transported later to the top cap reservoir.
- Perforated base plate and the wire mesh screens are placed on the base pedestal (see photo A6 in Appendix A). Then, the membrane is placed around the base pedestal and sealed with an O-ring, which is deployed using an O-ring expander.

In order to reconstitute the specimen, a membrane expander is mounted over the base pedestal.

- The membrane is stretched over the expander. Two ring clamps are placed around the expander, special attention being paid to avoid pinching the membrane when the expander is closed and tightened. Then, a small vacuum between -18 to -20 kPa is applied to the membrane expander (see Photo A7 in Appendix A).
The top cap reservoir is placed over the membrane expander, and the water level inside the membrane expander is raised up to the top of the top cap reservoir (see Photo A8 in Appendix A).

Then, the two-layer specimen (filter and base soil) is reconstituted using the water pluviation technique. Details of the specimen reconstitution are presented in Appendix B:

- After specimen reconstitution, the container with the top cap is submerged inside the top cap reservoir. Under water, the top cap is placed very gently over the base soil layer (see Photo A9 in Appendix A).
- Then, the expander is systematically tapped in a gentle manner in order to improve the base soil-filter interface.
- De-aired water from the top cap reservoir is emptied. Then, again very gently, the top cap reservoir is removed from the expander.
- A gentle pressure is applied to the top cap while the membrane is flipped around it, then the membrane is sealed with an O-ring, which is placed using an O-ring expander.
- Inlet and inlet port flexible tubing are connected to the top cap in a gentle manner.

The specimen reconstitution phase is now complete, and the specimen is ready to be consolidated as described in the next section.
4.4.3 Specimen consolidation

In the consolidation phase, the specimen is subjected to consolidation under vacuum pressure first, so as to provide sufficient strength to remove the expander. Then, the confining stress is applied, the vacuum pressure is released, and the consolidation of the specimen continues. Details of both consolidation phases are described in Appendix C:

- A vacuum pressure is applied very slowly through the pipette, which is connected to the inlet port. A manual vacuum regulator should be set approximately from -18 to -20 kPa. Readings in the pipette are taken before and after this pressure is applied, to measure the volume change due to consolidation. Furthermore, dial gage readings are also taken on the top cap to measure the change in length of the specimen (see Appendix B: for details).
- Then, the membrane expander is carefully removed (see Photo A10 in Appendix A).
- The triaxial chamber is filled with de-aird water. After that, the top plate is adjusted over the triaxial chamber and the loading ram is brought into contact with the top cap.
- Data acquisition system is turned on to measure transducers (DPT and TPT) and LVDT readings before applying confining stress.
- With all valves closed, except for the one connected to the pipette, the chamber pressure is increased until the difference between the vacuum pressure and the chamber pressure is slightly positive. Then, the vacuum pressure is turned off, and consolidation to the final confining stress is performed.
• A counter-balance weight is used to prevent lifting of the loading ram when confining stress is applied in the triaxial chamber. The counter-balance weight, which varies according to the confining stress applied, is placed on the upper part of the loading ram.

• The volume change of the specimen is measured again by means of the pipette.

After the specimen is consolidated, the following tasks are performed.

• The inlet port valve, which is connected to the pipette, is closed.

• After placing a large petri dish underneath the base frame, the bottom drain is opened.
  This allows those base soil particles which have passed through the filter because of specimen reconstitution and consolidation, to be collected, dried and weighed.

• The collection trough is rotated, so one of the pans is under the bottom drain of the base frame.

• Inflow CHD is placed at the same water level than the outflow CHD.

The specimen is left overnight, and the multistage seepage flow performed the following day, as described below.

**4.4.4 Multistage unidirectional seepage flow**

The purpose of the last phase of the test procedure is to evaluate the soil retention or filtering criterion under the influence of a unidirectional seepage flow. After specimen consolidation is completed and the inlet valve is opened, a unidirectional seepage flow is imposed in a downward direction in a multistage manner. Photo A11 in Appendix A shows the triaxial permeameter set
up during this phase. At the beginning of every stage, the difference in total head ($\Delta h_t$) is increased very slowly by raising the inflow CHD to achieve an incremental gain in height of about 2 cm. After an elapsed turn off seven minutes of seepage flow, the flow rate is manually measured in a beaker at the outlet of the outflow CHD over a period of one minute. After nine minutes, in the same stage and under the same difference in total head, the collection trough is rotated, in order to recover any mass loss. After ten minutes the stage is complete and another stage begins when the difference in total head is increased by another 2 cm, and the procedure is repeated.

4.5 Test procedure for the rigid-wall permeameter

As mentioned before, the use of a rigid-wall permeameter was found necessary to accomplish the testing program. In contrast with the testing procedure developed for the triaxial permeameter, the procedure for the rigid-wall permeameter involved only the specimen reconstitution phase. The reason for performing tests using the rigid-wall permeameter is to evaluate the margin of safety of the filtering criterion at large grain size ratios $D_{15}/d_{85}$ than 9, which were observed to be inherently incompatible.

Before any test is performed in the rigid-wall permeameter, all components of the permeameter are disassembled to be cleaned. Special attention was placed on the O-ring located at the lower part of the transparent cylinder, where silicone grease is spread to minimize the risk of leakage. Furthermore, a supply of water is prepared using the procedure described in Section 4.4.1. The configuration of the test device is illustrated in Photo A3 in Appendix A.
Before filling the rigid-wall permeameter with clean and de-aired water, the bottom drain in the base frame is closed with the bottom plug, the perforated plate and the two wire mesh screens are placed on the upper part of the base pedestal, and finally the transparent cylinder is mounted on the base pedestal. Then, the two-layer specimen (filter and base soil) is reconstituted using the water pluviation technique in the same manner that is performed on the triaxial permeameter. Details of the specimen reconstitution are presented in Appendix B. As soon as the base soil material started to be water-pluviated, the time is recorded and the test begins: The procedure is based on the premise that grains of base soil immediately begins to enter the interstices of the filter material under the influences of gravity alone.

4.6 Summary

The testing program, including the test materials, was described in this chapter. In order to evaluate the onset of filter incompatibility and explore the margin of safety of the filtering criterion rule proposed by Terzaghi (1939), grain size ratios $D_{15}/d_{85}$ between 7.5 and 13.3 are tested, using three different uniform base soils. The procedure to operate this new triaxial permeameter was also described. Systematic controls over the three phases of the testing procedure were performed in order to assure a good quality of the data results. These three phases are: specimen reconstitution, specimen consolidation and multistage seepage flow. Complementary tests were performed on the rigid-wall permeameter, so the testing procedure of this test device was described as well. Tests in this equipment have only one phase, which was
specimen reconstitution. In the following chapter, the results and analysis of the entire test program already presented are described.
Table 4.1: Laboratory testing program

<table>
<thead>
<tr>
<th>D_{15}/d_{85}</th>
<th>Confining stress (kPa)</th>
<th>Filter Size (mm)</th>
<th>D_{15} (mm)</th>
<th>Base soil Size (mm)</th>
<th>d_{85} (mm)</th>
<th>Test code</th>
<th>Test device</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.5</td>
<td>50</td>
<td>1.2 - 1.7</td>
<td>1.275</td>
<td>0.12 - 0.18</td>
<td>0.171</td>
<td>7.5 - 50</td>
<td>Triaxial permeameter</td>
</tr>
<tr>
<td>7.5</td>
<td>150</td>
<td>1.2 - 1.7</td>
<td>1.275</td>
<td>0.12 - 0.18</td>
<td>0.171</td>
<td>7.5 - 150</td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td>300</td>
<td>1.2 - 1.7</td>
<td>1.275</td>
<td>0.12 - 0.18</td>
<td>0.171</td>
<td>7.5 - 300</td>
<td></td>
</tr>
<tr>
<td>8.7</td>
<td>50</td>
<td>1.4 - 2.0</td>
<td>1.490</td>
<td>0.12 - 0.18</td>
<td>0.171</td>
<td>8.7 - 50</td>
<td></td>
</tr>
<tr>
<td>8.7</td>
<td>150</td>
<td>1.4 - 2.0</td>
<td>1.490</td>
<td>0.12 - 0.18</td>
<td>0.171</td>
<td>8.7 - 150</td>
<td></td>
</tr>
<tr>
<td>8.7</td>
<td>150</td>
<td>1.4 - 2.0</td>
<td>1.490</td>
<td>0.12 - 0.18</td>
<td>0.171</td>
<td>8.7 - 150 (R)</td>
<td></td>
</tr>
<tr>
<td>8.7</td>
<td>300</td>
<td>1.4 - 2.0</td>
<td>1.490</td>
<td>0.12 - 0.18</td>
<td>0.171</td>
<td>8.7 - 300</td>
<td></td>
</tr>
<tr>
<td>9.8 (*)</td>
<td>0</td>
<td>2.8 - 3.2</td>
<td>2.860</td>
<td>0.25 - 0.30</td>
<td>0.293</td>
<td>9.8 - RW</td>
<td>Rigid-wall permeameter</td>
</tr>
<tr>
<td>10.5</td>
<td>0</td>
<td>2.0 - 2.8</td>
<td>2.120</td>
<td>0.15 - 0.21</td>
<td>0.201</td>
<td>10.5 - RW</td>
<td></td>
</tr>
<tr>
<td>11.3</td>
<td>0</td>
<td>2.1 - 3.3</td>
<td>2.280</td>
<td>0.15 - 0.22</td>
<td>0.201</td>
<td>11.3 - RW</td>
<td></td>
</tr>
<tr>
<td>12.4</td>
<td>0</td>
<td>2.0 - 2.8</td>
<td>2.120</td>
<td>0.12 - 0.18</td>
<td>0.171</td>
<td>12.4 - RW</td>
<td></td>
</tr>
<tr>
<td>13.3</td>
<td>0</td>
<td>2.1 - 3.3</td>
<td>2.280</td>
<td>0.12 - 0.18</td>
<td>0.171</td>
<td>13.3 - RW</td>
<td></td>
</tr>
</tbody>
</table>

(*) An axial load smaller than 3 kPa was applied.
Figure 4.1: Grain size distribution curves made of artificial glass beads
Chapter 5: Test results

5.1 Introduction

The results of the testing program are presented and discussed in this chapter. The main objectives of these tests include: (1) the demonstration of the repeatability of the test method; (2) the definition of the onset of filter incompatibility; (3) the study of the effect of confining effective stress ($\sigma'_3$) and hydraulic gradient ($i$) at the onset of filter incompatibility; and, (4) the evaluation of the margin of safety of the filtering criterion rule ($D_{15}/d_{85} \leq 4$) for uniform cohesionless base soils. To achieve these objectives, laboratory test parameters were obtained for interpretation and further analysis.

Initially, it was planned to use only the triaxial permeameter test device for the testing program, but this could only be used for base soil-filter specimens with grain size ratio $D_{15}/d_{85}$ smaller than 9. As explained previously (see Section 4.5), some difficulties were encountered during reconstitution of base soil-filter specimens with grain size ratio $D_{15}/d_{85}$ larger than 9. Therefore, a rigid-wall permeameter was used to overcome these specimen reconstitution difficulties. Twelve tests were performed in this study (see Table 4.1).

5.2 Test parameters

The results presented herein are based on direct measurements or calculations made from direct measurements during multistage seepage flow phase. Direct measurements such as head loss across the specimen ($\Delta h_s$) and seepage-induced axial deformation ($\Delta l$) were used to calculate the hydraulic gradient ($i$), which was combined with the measured flow rate ($q$) to compute the
discharge velocity ($v$). Then, permeability ($k_{bf}$) of the base soil-filter specimen was calculated, and seepage-induced cumulative total mass loss ($m_t$) was measured. A description of the test parameters is presented in the following paragraphs.

5.2.1 Head loss

The head loss ($\Delta h_s$) across the specimen was measured directly and continuously, as previously described (see Section 3.3.6.3). The difference in total head ($\Delta h_t$), which is controlled by means of the inflow and the outflow CHD (see Section 3.3.4 for details), is reduced due to energy losses of the system, as presented in Equation 5.1.

$$\Delta h_s = \Delta h_t - \textit{energy losses}$$

(5.1)

Energy losses of the system are attributed to friction in the tubes and fittings of the triaxial permeameter when water flow was passing through. In general, energy losses reduced the difference in total head ($\Delta h_t$) by approximately 35% to 55%; hence, the higher the difference in total head, the greater the energy losses.

5.2.2 Seepage-induced axial deformation

During the multistage seepage flow, the length of the specimen started to diminish due to a potential combination of seepage-induced consolidation and mass loss of the base soil layer. This reduction in lengths is called hereafter the seepage-induced axial deformation ($\Delta l$), and was measured directly and continuously (see Section 3.3.6.2). It was noticed that, in some cases, the axial deformation was not uniform across the area of the specimen, as evident form tilting of the
top cap. This was attributed to a non-uniform spatial distribution of mass loss in the specimen, which developed at the contact surface between the base soil and the filter layer.

5.2.3 Hydraulic gradient

The hydraulic gradient \(i\) was calculated by the Equation 5.2:

\[
i = \frac{\Delta h_s}{L - \Delta l}
\]  

(5.2)

Where, \(\Delta h_s\) is the head loss across the specimen, \(L\) is the length of the specimen after consolidation (see Table 5.1), and \(\Delta l\) is the seepage-induced axial deformation.

5.2.4 Flow rate

The flow rate \(q\) was measured directly at discrete intervals during all stages of the multistage seepage flow phase. Details of how it was measured were described previously (see Section 3.3.6.5).

5.2.5 Discharge velocity

The discharge velocity \(v\) was calculated during all stages at discrete intervals using the Equation 5.3.

\[
v = \frac{q}{A_c}
\]  

(5.3)

Where, \(q\) is the flow rate and \(A_c\) is the corrected area of the specimen after consolidation (see Equation 5.4).
The initial volume of the specimen prior to consolidation is denoted by \( V_o \), and the change of volume \( \Delta V \), which was measured with a pipette (see Section 3.3.6.4), and the total length of the specimen \( L \) were obtained after the specimen was consolidated (see Appendix B and C for details). It is important to mention that the across-sectional area of the specimen was not corrected during the multistage seepage flow phase. The reason was due to the pipette cannot measure the volume change of the specimen while the seepage flow is imposed.

5.2.6 Permeability

The permeability of the base soil-filter specimen \( (k_{bf}) \) was calculated using Darcy’s law, as mentioned previously (see Equation 2.2), which is rearranged for simplicity in Equation 5.5:

\[
k_{bf} = \frac{v}{i}
\]

(5.5)

Where, \( v \) is the discharge velocity and \( i \) is the average hydraulic gradient for a specific time interval over which the flow rate was measured (see Section 3.3.6.5). This time interval was approximately one minute.

Theoretical values of permeability for the two-layer system \( (k_v) \) were calculated to compare them with experimental values obtained in this study.

\[
k_v = \frac{L_b + L_f}{\frac{L_b}{k_b} + \frac{L_f}{k_f}}
\]

(5.6)
Where $L_b$ and $L_f$ are the length of the base soil and the filter layer, respectively (see Appendix B for details). Permeability values of the base soil and the filter layer are denoted by $k_b$ and $k_f$.

Length of the base soil ($L_b$) and the filter ($L_f$) were recorded after specimen reconstitution (see Table 5.1), because at the end of that phase both lengths were known with certainty. Upon completion of consolidation, the individual length of base soil and filter was unknown; however, the total length of the specimen ($L$) was known with reliability, and was used for further calculations. It is important to mention that these slight variations in length of the base soil and the filter due to consolidation did not have a significant impact on the theoretical permeability ($k_v$) (see Table 5.1), since it is mainly controlled by the permeability of the base soil ($k_b$).

5.2.7 Mass loss

Mass loss ($m$) was defined by the finer fraction (base soil) that washed through the filter into the collection trough (see Section 3.3.5). It is important to mention that base soil particles may be retained within the interstices of filter during any phase of the test and it was not possible to account for this component of mass loss. The seepage-induced mass loss ($m$) was recorded at all stages during the multistage seepage flow phase (see Section 4.4.4), and used to calculate the seepage-induced cumulative total mass loss ($m_t$) expressed as a percentage:

$$m_t = \frac{\sum m}{M_{bc}} \times 100$$

(5.7)

Where the dry mass of the base soil after consolidation is denoted by $M_{bc}$ (see Table 5.2).
5.3 Triaxial permeameter test data

A total of 7 tests were conducted using grain size ratios $D_{15}/d_{85}$ smaller than 9, as shown in Table 4.1. In these tests, specimens were subjected to a multistage seepage flow, after consolidation under one of three different isotropic confining stresses (50, 150 and 300 kPa). Isotropic consolidation of the specimens was performed with single drainage in the upward direction through the inlet port in the top cap (see Figure 3.3). The grain size ratios $D_{15}/d_{85}$ used in these series of tests were 7.5 and 8.7, with 3 and 4 tests respectively. A summary of the results of each test is described in the following paragraphs.

5.3.1 Test 7.5-50

Test 7.5-50 was performed on the “0.12 to 0.18 mm” base soil and the “1.2 to 1.7 mm” filter, to yield a grain size ratio $D_{15}/d_{85}$ of 7.5. After specimen reconstitution, an isotropic confining stress ($\sigma'_3$) of 50 kPa was applied to consolidate the specimen. At this point, the mass loss due to reconstitution and consolidation ($m_{rc}$) (see Appendix C for details) was less than 0.1% (see Table 5.2), and the void ratio was 0.60 (see Table 5.1). The consolidation settlement and the length of the specimen after consolidation ($L$) were 2.7 mm and 98.0 mm, respectively (see Table 5.1).

Since it was considered representative of the typical behaviour for this series of tests ($D_{15}/d_{85} = 7.5$), the test is described in detail. Figure 5.1 shows the relation between the total seepage-induced axial deformation ($\Delta l$) and the hydraulic gradient ($i$). Axial deformation of 1.0 mm was recorded at the maximum hydraulic gradient ($i$) of 7.8. The variation of hydraulic gradient with
time (see Figure 5.2) shows that it ranged from 0 to 5.8 at a rate of increase of $\Delta h_t = 2$ cm every 10 minutes, and from 5.8 to 7.8 at a rate of $\Delta h_t = 10$ cm every 10 minutes. The rate increase of five was imposed to evaluate its influence on the specimen response. The maximum hydraulic gradient ($i$) of 7.8 (see Figure 5.2) is associated with a difference in total head across the system ($\Delta h_t$) and a head loss across the specimen ($\Delta h_s$) of 164 and 76 cm, respectively. The difference of 88 cm is attributed to energy losses in the system.

Figure 5.3 shows the relation between the discharge velocity ($v$) and the hydraulic gradient ($i$). It is evident that this relation is linear at the initial stage of seepage flow ($i \leq 2.5$). However, at hydraulic gradient values ($i$) larger than 2.5 this relation is non-linear, implying Darcy’s law is not satisfied. This non-linearity is tentatively attributed to a change in flow regime from laminar to semi-turbulent to turbulent flow. A comprehensive analysis of these test data is provided in Chapter 6.

A permeability ($k_{bf}$) value of approximately 0.036 cm/sec was computed for the base soil-filter specimen (see Figure 5.4). This experimentally-derived value was compared with a theoretical value (see Equation 5.6) to verify that the experimental findings are within an appropriate range. The length of the base soil ($L_b$) and the filter ($L_f$) were 51.6 and 49.1 mm, respectively (see Table 5.1). The 0.12 to 0.18 mm base soil ($k_b$) has a permeability measured at 0.02 cm/sec (see Appendix D for details). The permeability of the filter ($k_f$) was deduced, from Mitchell et al (2005, pp. 254), to be 0.45 cm/sec, based on its grain size range between 1.2 and 1.7 mm. Thus, a
theoretical permeability ($k_v$) of 0.038 cm/sec was calculated (see Table 5.1). The values of $k_{bf} = 0.036$ cm/sec and $k_v = 0.038$ cm/sec are in excellent agreement. As noted previously, the relation between the discharge velocity ($v$) and the hydraulic gradient($i$) is non-linear at $i \geq 2.5$, which anticipated a reduction in permeability values, as observed in Figure 5.4. Recall that measurement of flow rate in the early period of testing (approximately 70 minutes) is influenced by compliance of the outflow CHD, as explained before (see Section 3.3.6.5).

The cumulative total mass loss ($m_t$) due to induced seepage flow (see Figure 5.5) reveals no evidence of significant loss during the test. A loss of $m_t = 103$ g/m$^2$ was recorded at the end of the final stage ($i = 7.8$) of seepage flow phase, which represents just 0.1% of the base soil layer mass after specimen consolidation (see Table 5.2) and deemed negligible.

In summary, Test 7.5-50 is considered filter compatible based on a negligible cumulative total mass loss at the end of the multistage seepage flow phase. The observed decrease in permeability ($k_{bf}$) is tentatively attributed to a change in the flow regime from laminar to semi-turbulent to turbulent flow at hydraulic gradient ($i$) values larger than 2.5. The response of the specimen appears independent of the increase in hydraulic gradient.

### 5.3.2 Test 7.5-150

Test 7.5-150 was consolidated under an isotropic confining effective stress $\sigma'_3 = 150$ kPa. At this point, the mass loss due to reconstitution and consolidation ($m_{rc}$) (see Appendix C for details) was 0.1% (see Table 5.2), and the void ratio was 0.59 (see Table 5.1). The consolidation
settlement and the length of the specimen after consolidation \((L)\) were 2.1 mm and 101.4 mm, respectively (see Table 5.1).

Figure 5.1 shows the relation between the total seepage-induced axial deformation \((\Delta l)\) and the hydraulic gradient \((i)\). Axial deformation of 0.4 mm was recorded at the maximum hydraulic gradient \((i)\) of 7.9. The variation of hydraulic gradient with time (see Figure 5.2) shows that it ranged from 0 to 5.9 at a rate of increase of \(\Delta h_t = 2\) cm every 10 minutes, and from 5.9 to 7.9 at a rate of \(\Delta h_t = 10\) cm every 10 minutes. The rate increase of five was imposed to evaluate its influence on the specimen response. The maximum hydraulic gradient \((i)\) of 7.9 (see Figure 5.2) is associated with a difference in total head across the system \((\Delta h_t)\) and a head loss across the specimen \((\Delta h_s)\) of 164 and 77.3 cm, respectively. The difference of 86.7 cm is attributed to energy losses in the system.

Figure 5.3 shows the relation between the discharge velocity \((v)\) and the hydraulic gradient \((i)\). It is evident that this relation is linear at the initial stage of seepage flow \((i \leq 2.5)\). However, at hydraulic gradient values \((i)\) larger than 2.5 this relation is non-linear, implying Darcy’s law is not satisfied. This non-linearity is tentatively attributed to a change in flow regime from laminar to semi-turbulent to turbulent flow.

A permeability \((k_{bf})\) value of approximately 0.034 cm/sec was computed for the base soil-filter specimen (see Figure 5.4). The values of \(k_{bf} = 0.034\) cm/sec and \(k_v = 0.038\) cm/sec are in
excellent agreement. As noted previously, the relation between the discharge velocity \( v \) and the hydraulic gradient \( i \) is non-linear at \( i \geq 2.5 \), which anticipated a reduction in permeability values, as observed in Figure 5.4. Recall that measurement of flow rate in the early period of testing (approximately 70 minutes) is discarded, as explained before (see Section 3.3.6.5).

The cumulative total mass loss \( m_t \) due to induced seepage flow (see Figure 5.5) reveals no evidence of significant loss during the test. A loss of \( m_t = 90 \) g/m\(^2\) was recorded at the end of the final stage \(( i = 7.9 \) of seepage flow phase, which represents just 0.1% of the base soil layer mass after specimen consolidation (see Table 5.2) and deemed negligible.

In summary, Test 7.5-150 is considered filter compatible based on a negligible cumulative total mass loss at the end of the multistage seepage flow phase. The observed decrease in permeability \( k_{bf} \) is tentatively attributed to a change in the flow regime from laminar to semi-turbulent to turbulent flow at hydraulic gradient \( i \) values larger than 2.5. The response of the specimen appears independent of the increase in hydraulic gradient.

### 5.3.3 Test 7.5-300

Test 7.5-300 was consolidated under an isotropic confining effective stress \( \sigma'_3 = 300 \) kPa. At this point, the mass loss due to reconstitution and consolidation \( m_{rc} \) (see Appendix C for details) was less than 0.1% (see Table 5.2), and the void ratio was 0.61 (see Table 5.1). The consolidation settlement and the length of the specimen after consolidation \( L \) were 2.5 mm and 100.7 mm, respectively (see Table 5.1).
Figure 5.1 shows the relation between the total seepage-induced axial deformation ($\Delta l$) and the hydraulic gradient ($i$). Axial deformation of 1.8 mm was recorded at the maximum hydraulic gradient ($i$) of 7.8. The variation of hydraulic gradient with time (see Figure 5.2) shows that it ranged from 0 to 5.9 at a rate of increase of $\Delta h_t = 2$ cm every 10 minutes, and from 5.9 to 7.8 at a rate of $\Delta h_t = 10$ cm every 10 minutes. The rate increase of five was imposed to evaluate its influence on the specimen response. The maximum hydraulic gradient ($i$) of 7.8 (see Figure 5.2) is associated with a difference in total head across the system ($\Delta h_t$) and a head loss across the specimen ($\Delta h_s$) of 164 and 77.0 cm, respectively. The difference of 87.0 cm is attributed to energy losses in the system.

Figure 5.3 shows the relation between the discharge velocity ($v$) and the hydraulic gradient ($i$). It is evident that this relation is linear at the initial stage of seepage flow ($i \lesssim 2.5$). However, at hydraulic gradient values ($i$) larger than 2.5 this relation is non-linear, implying Darcy’s law is not satisfied. This non-linearity is tentatively attributed to a change in flow regime from laminar to semi-turbulent to turbulent flow.

A permeability ($k_{bf}$) value of approximately 0.034 cm/sec was computed for the base soil-filter specimen (see Figure 5.4). The values of $k_{bf} = 0.034$ cm/sec and $k_v = 0.038$ cm/sec are in excellent agreement. As noted previously, the relation between the discharge velocity ($v$) and the hydraulic gradient ($i$) is non-linear at $i \geq 2.5$, which anticipated a reduction in permeability
values, as observed in Figure 5.4. Recall that measurement of flow rate in the early period of testing (approximately 70 minutes) is discarded, as explained before (see Section 3.3.6.5).

The cumulative total mass loss \( m_c \) due to induced seepage flow (see Figure 5.5) reveals no evidence of significant loss during the test. A loss of \( m_c = 287 \text{ g/m}^2 \) was recorded at the end of the final stage \((i = 7.8)\) of seepage flow phase, which represents just 0.4% of the base soil layer mass after specimen consolidation (see Table 5.2) and deemed negligible.

In summary, Test 7.5-300 is considered filter compatible based on a negligible cumulative total mass loss at the end of the multistage seepage flow phase. The observed decrease in permeability \( k_{bf} \) is tentatively attributed to a change in the flow regime from laminar to semi-turbulent to turbulent flow at hydraulic gradient \((i)\) values larger than 2.5. The response of the specimen appears independent of the increase in hydraulic gradient.

5.3.4 Test 8.7-50

Test 8.7-50 was performed on the “0.12 to 0.18 mm” base soil and the “1.4 to 2.0 mm” filter, to yield a grain size ratio \( D_{15}/d_{85} \) of 8.7. After specimen reconstitution, an isotropic confining stress \((\sigma'_3)\) of 50 kPa was applied to consolidate the specimen. At this point, the mass loss due to reconstitution and consolidation \((m_{rc})\) (see Appendix C for details) was 5.1% (see Table 5.2), and the void ratio was 0.59 (see Table 5.1). The consolidation settlement and the length of the specimen after consolidation \((L)\) were 2.4 mm and 99.5 mm, respectively (see Table 5.1).
Since it was considered representative of the typical behaviour for this series of tests ($D_{15}/d_{85} = 8.7$), the test is described in detail. Figure 5.6 shows the relation between the total seepage-induced axial deformation ($\Delta l$) and the hydraulic gradient ($i$). Axial deformation of 14.4 mm was recorded at the maximum hydraulic gradient ($i$) of 10.2. The variation of hydraulic gradient with time (see Figure 5.7) shows that it ranged from 0 to 10.2 at a constant rate of increase of $\Delta h_t = 2 \text{ cm}$ every 10 minutes. The maximum hydraulic gradient ($i$) of 10.2 (see Figure 5.7) is associated with a difference in total head across the system ($\Delta h_t$) and a head loss across the specimen ($\Delta h_s$) of 162 and 88.2 cm, respectively. The difference of 73.8 cm is attributed to energy losses in the system.

Figure 5.8 shows the relation between the discharge velocity ($v$) and the hydraulic gradient ($i$). It is evident that this relation is linear at the initial stage of seepage flow ($i \leq 1.6$). However, at hydraulic gradient values ($i$) larger than approximately 1.6 this relation is non-linear, implying Darcy’s law is not satisfied. This non-linearity is tentatively attributed to: (i) base soil particle migration into the filter, as implied by the seepage-induced axial deformation (see Figure 5.6); and (ii) a change in flow regime from laminar to semi-turbulent to turbulent flow. A comprehensive analysis of these test data is provided in Chapter 6.

A permeability ($k_{bf}$) value of approximately 0.033 cm/sec was computed for the base soil-filter specimen (see Figure 5.9). This experimentally-derived value was compared with a theoretical value (see Equation 5.6) to verify that the experimental findings are within an appropriate range. The length of the base soil ($L_b$) and the filter ($L_f$) were 52.8 and 49.1 mm, respectively (see
Table 5.1). The 0.12 to 0.18 mm base soil \( (k_b) \) has a permeability measured at 0.02 cm/sec (see Appendix D for details). The permeability of the filter \( (k_f) \) was deduced, from Mitchell et al (2005, pp. 254), to be 0.5 cm/sec, based on its grain size range between 1.4 and 2.0 mm. Thus, a theoretical permeability \( (k_p) \) of 0.038 cm/sec was calculated (see Table 5.1). The values of \( k_{bf} = 0.033 \text{ cm/sec} \) and \( k_p = 0.038 \text{ cm/sec} \) are in excellent agreement. As noted previously, the relation between the discharge velocity \( (v) \) and the hydraulic gradient \( (i) \) is non-linear at \( i \geq 1.6 \), which is anticipated to yield reduction in permeability values, as observed in Figure 5.9. Recall that measurement of flow rate in the early period of testing (approximately 70 minutes) is discarded, as explained before (see Section 3.3.6.5).

The cumulative total mass loss \( (m_t) \) due to induced seepage flow (see Figure 5.10) reveals evidence of significant loss during the test, in contrast to tests with a grain size ratio \( D_{15}/d_{85} \) of 7.5. A loss \( m_t = 15310 \text{ g/m}^2 \) was recorded at the end of the final stage \( (i = 10.4) \) of the seepage flow, which represents 19% of the base soil layer mass after specimen consolidation (see Table 5.2) and is deemed significant. It is also observed that, at hydraulic gradients larger than about 1.6, large quantities of seepage-induced mass loss are associated with large axial deformations (see Figure 5.6 and 5.10).

In summary, Test 8.7-50 is considered filter incompatible based on a significant cumulative total mass loss \( (m_t = 19\%) \) at the end of the multistage seepage flow phase. The observed decrease in permeability \( (k_{bf}) \) is tentatively attributed to base soil particle migration into the filter and a

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change in flow regime from laminar to semi-turbulent to turbulent flow at hydraulic gradient \((i)\) values larger than about 1.6. The response of the specimen appears dependent of the increase in hydraulic gradient.

### 5.3.5 Test 8.7-150

Test 8.7-150 was consolidated under an isotropic confining effective stress \(\sigma'_3 = 150 \text{ kPa}\). At this point, the mass loss due to reconstitution and consolidation \((m_{rc})\) (see Appendix C for details) was 1.6\% (see Table 5.2), and the void ratio was 0.58 (see Table 5.1). The consolidation settlement and the length of the specimen after consolidation \((L)\) were 4.3 mm and 99.0 mm, respectively (see Table 5.1).

Figure 5.6 shows the relation between the total seepage-induced axial deformation \((\Delta l)\) and the hydraulic gradient \((i)\). Axial deformation of 23.4 mm was recorded at the maximum hydraulic gradient \((i)\) of 10.7. The variation of hydraulic gradient with time (see Figure 5.7) shows that it ranged from 0 to 10.7 at a constant rate of increase of \(\Delta h_t = 2 \text{ cm}\) every 10 minutes. The maximum hydraulic gradient \((i)\) of 10.7 (see Figure 5.7) is associated with a difference in total head across the system \((\Delta h_s)\) and a head loss across the specimen \((\Delta h_s)\) of 148 and 80.5 cm, respectively. The difference of 67.5 cm is attributed to energy losses in the system.

Figure 5.8 shows the relation between the discharge velocity \((v)\) and the hydraulic gradient\((i)\). It is evident that this relation is linear at the initial stage of seepage flow\((i \leq 1.6)\). However, at hydraulic gradient values \((i)\) larger than approximately 1.6 this relation is non-linear, implying
Darcy’s law is not satisfied. This non-linearity is tentatively attributed to: (i) base soil particle migration into the filter, as implied by the seepage-induced axial deformation (see Figure 5.6); and (ii) a change in flow regime from laminar to semi-turbulent to turbulent flow.

A permeability \(k_{bf}\) value of approximately 0.032 cm/sec was computed for the base soil-filter specimen (see Figure 5.9). The values of \(k_{bf} = 0.032 \text{ cm/sec} \) and \(k_v = 0.038 \text{ cm/sec} \) are in excellent agreement. As noted previously, the relation between the discharge velocity \((v)\) and the hydraulic gradient\((i)\) is non-linear at \(i > 1.6\), which is anticipated to yield reduction in permeability values, as observed in Figure 5.9. Recall that measurement of flow rate in the early period of testing (approximately 70 minutes) is discarded, as explained before (see Section 3.3.6.5).

The cumulative total mass loss \(m_t\) due to induced seepage flow (see Figure 5.10) reveals evidence of significant loss during the test, in contrast to tests with a grain size ratio \(D_{15}/d_{85}\) of 7.5. A loss \(m_t = 26457 \text{ g/m}^2\) was recorded at the end of the final stage \((i = 10.7)\) of the seepage flow, which represents 32.3% of the base soil layer mass after specimen consolidation (see Table 5.2) and is deemed significant. It is also observed that, at hydraulic gradients larger than about 1.6, large quantities of seepage-induced mass loss are associated with large axial deformations (see Figure 5.6 and 5.10).

In summary, Test 8.7-150 is considered filter incompatible based on a significant cumulative total mass loss \(m_t = 32.3\%) at the end of the multistage seepage flow phase. The observed
decrease in permeability \( (k_{bf}) \) is tentatively attributed to base soil particle migration into the filter and a change in flow regime from laminar to semi-turbulent to turbulent flow at hydraulic gradient \( (i) \) values larger than about 1.6. The response of the specimen appears dependent of the increase in hydraulic gradient.

5.3.6 Test 8.7-150(R)

Test 8.7-150(R) was isotropically consolidated under a 150 kPa. It was compared with Test 8.7-150 to examine issues of repeatability and to demonstrate that laboratory test results were reliable. Repeatability of the findings was considered with reference to void ratio, permeability values and mass loss of the base soil through the filter.

The mass loss due to reconstitution and consolidation \( (m_{rc}) \) (see Appendix C for details) was 3.0% (see Table 5.2), and the void ratio was 0.57 (see Table 5.1). The consolidation settlement and the length of the specimen after consolidation \( (L) \) were 3.6 mm and 99.2 mm, respectively (see Table 5.1).

Figure 5.6 shows the relation between the total seepage-induced axial deformation \( (\Delta l) \) and the hydraulic gradient \( (i) \). Axial deformation of 11.2 mm was recorded at the maximum hydraulic gradient \( (i) \) of 5.6. The variation of hydraulic gradient with time (see Figure 5.7) shows that it ranged from 0 to 5.6 at a constant rate of increase of \( \Delta h_t = 2 \) cm every 10 minutes. The maximum hydraulic gradient \( (i) \) of 5.6 (see Figure 5.7) is associated with a difference in total head across the system \( (\Delta h_t) \) and a head loss across the specimen \( (\Delta h_s) \) of 88 and 48.9 cm,
respectively. The difference of 39.1 cm is attributed to energy losses in the system. On the other hand, in the attempt to raise the hydraulic gradient from 5.6 to 5.7, the membrane burst at the base soil-filter interface. Bursting was attributed to loss of base soil particles into and/or through the filter, which led to deformation of the membrane at the interface, in spite of the fact that the membrane was reinforced (see Section 3.3.2.3 for details). Photo A12 in Appendix A shows the specimen before the membrane burst.

Figure 5.8 shows the relation between the discharge velocity \( (v) \) and the hydraulic gradient \( (i) \). It is evident that this relation is linear at the initial stage of seepage flow \( (i \leq 1.6) \). However, at hydraulic gradient values \( (i) \) larger than approximately 1.6 this relation is non-linear, implying Darcy’s law is not satisfied. This non-linearity is tentatively attributed to: (i) base soil particle migration into the filter, as implied by the seepage-induced axial deformation (see Figure 5.6); and (ii) a change in flow regime from laminar to semi-turbulent to turbulent flow.

A permeability \( (k_{bf}) \) value of approximately 0.033 cm/sec was computed for the base soil-filter specimen (see Figure 5.9). The values of \( k_{bf} = 0.033 \) cm/sec and \( k_v = 0.038 \) cm/sec are in excellent agreement. As noted previously, the relation between the discharge velocity \( (v) \) and the hydraulic gradient \( (i) \) is non-linear at \( i > 1.6 \), which is anticipated to yield reduction in permeability values, as observed in Figure 5.9. Recall that measurement of flow rate in the early period of testing (approximately 70 minutes) is discarded, as explained before (see Section 3.3.6.5).
The cumulative total mass loss ($m_t$) due to induced seepage flow (see Figure 5.10) reveals evidence of significant loss during the test, in contrast to tests with a grain size ratio $D_{15}/d_{85}$ of 7.5. A loss $m_t = 14437$ g/m$^2$ was recorded at the end of the final stage ($i = 5.6$) of the seepage flow, which represents 17.6% of the base soil layer mass after specimen consolidation (see Table 5.2) and is deemed significant. It is also observed that, at hydraulic gradients larger than about 1.6, large quantities of seepage-induced mass loss are associated with large axial deformations (see Figure 5.6 and 5.10).

In summary, Test 8.7-150(R) is considered filter incompatible based on a significant cumulative total mass loss ($m_t = 17.6\%$) at the end of the multistage seepage flow phase. The observed decrease in permeability ($k_{bf}$) is tentatively attributed to base soil particle migration into the filter and a change in flow regime from laminar to semi-turbulent to turbulent flow at hydraulic gradient ($i$) values larger than about 1.6. The response of the specimen appears dependent of the increase in hydraulic gradient.

Therefore, both specimens have good agreement in the void ratio ($e \sim 0.58 – 0.57$), permeability values ($k_{bf} \sim 0.032 – 0.033$ cm/sec) and the hydraulic gradient ($i > 1.6$) when the relation between the discharge velocity ($v$) and the hydraulic gradient($i$) becomes non-linear, but they do not have agreement in the cumulative total mass loss at the end of the multistage seepage flow; however, it is concluded that the test procedure adopted is repeatable and the test results are reliable.
5.3.7 Test 8.7-300

Test 8.7-300 was consolidated under an isotropic confining effective stress $\sigma'_3 = 300$ kPa. At this point, the mass loss due to reconstitution and consolidation ($m_{rc}$) (see Appendix C for details) was 1.8% (see Table 5.2), and the void ratio was 0.54 (see Table 5.1). The consolidation settlement and the length of the specimen after consolidation ($L$) were 4.0 mm and 98.6 mm, respectively (see Table 5.1).

Figure 5.6 shows the relation between the total seepage-induced axial deformation ($\Delta l$) and the hydraulic gradient ($i$). Axial deformation of 12.8 mm was recorded at the maximum hydraulic gradient ($i$) of 2.6. The variation of hydraulic gradient with time (see Figure 5.7) shows that it ranged from 0 to 2.6 at a constant rate of increase of $\Delta h_t = 2$ cm every 10 minutes. The maximum hydraulic gradient ($i$) of 2.6 (see Figure 5.7) is associated with a difference in total head across the system ($\Delta h_t$) and a head loss across the specimen ($\Delta h_s$) of 46 and 23.7 cm, respectively. The difference of 22.3 cm is attributed to energy losses in the system. On the other hand, in the attempt to raise the hydraulic gradient from 2.6 to 2.7, the membrane burst at the base soil-filter interface due to deformation of the membrane at the interface, in spite of the fact that the membrane was reinforced (see Section 3.3.2.3 for details).

Figure 5.8 shows the relation between the discharge velocity ($v$) and the hydraulic gradient($i$). It is evident that this relation is linear at the initial stage of seepage flow ($i \leq 1.6$). However, at hydraulic gradient values ($i$) larger than approximately 1.6 this relation is non-linear, implying
Darcy’s law is not satisfied. This non-linearity is tentatively attributed to: (i) base soil particle migration into the filter, as implied by the seepage-induced axial deformation (see Figure 5.6); and (ii) a change in flow regime from laminar to semi-turbulent to turbulent flow.

A permeability \( (k_{bf}) \) value of approximately 0.032 cm/sec was computed for the base soil-filter specimen (see Figure 5.9). The values of \( k_{bf} = 0.032 \) cm/sec and \( k_{v} = 0.038 \) cm/sec are in excellent agreement. As noted previously, the relation between the discharge velocity \( (v) \) and the hydraulic gradient\( (i) \) is non-linear at \( i > 1.6 \), which is anticipated to yield reduction in permeability values, as observed in Figure 5.9. Recall that measurement of flow rate in the early period of testing (approximately 70 minutes) is discarded, as explained before (see Section 3.3.6.5).

The cumulative total mass loss \( (m_t) \) due to induced seepage flow (see Figure 5.10) reveals evidence of significant loss during the test, in contrast to tests with a grain size ratio \( D_{15}/d_{85} \) of 7.5. A loss \( m_t = 12900 \) g/m² was recorded at the end of the final stage \( (i = 2.6) \) of the seepage flow, which represents 15.4% of the base soil layer mass after specimen consolidation (see Table 5.2) and is deemed significant. It is also observed that, at hydraulic gradients larger than about 1.6, large quantities of seepage-induced mass loss are associated with large axial deformations (see Figure 5.6 and 5.10).

In summary, Test 8.7-300 is considered filter incompatible based on a significant cumulative total mass loss \( (m_t = 15.4\%) \) at the end of the multistage seepage flow phase. The observed
decrease in permeability ($k_{bf}$) is tentatively attributed to base soil particle migration into the filter and a change in flow regime from laminar to semi-turbulent to turbulent flow at hydraulic gradient ($i$) values larger than about 1.6. The response of the specimen appears dependent of the increase in hydraulic gradient.

5.4 Rigid-wall permeameter test data

In total, five tests were conducted using grain size ratios $D_{15}/d_{85}$ larger than 9 (see Table 4.1). Recall that, in these tests, experience proved it was extremely difficult to consolidate the specimens and subject them to multistage seepage flow, so the flexible-wall permeameter (triaxial permeameter) was exchanged for a rigid-wall permeameter that mounted on the same bottom pedestal (see Section 3.4). Base soil-filter specimens with grain size ratios $D_{15}/d_{85}$ of 9.8, 10.5, 11.3, 12.4 and 13.3 were tested. A summary of the results of each test is described in the following paragraphs.

5.4.1 Test 9.8-RW

This test was performed using a grain size ratio $D_{15}/d_{85}$ of 9.8. Initially, it was intended to test the specimen in the triaxial permeameter under confining stress and with multistage seepage flow, however after two failed attempts to consolidate the specimen under vacuum, a decision was taken to reconstitute it and test it in the rigid-wall permeameter.

The specimen was prepared and reconstituted using the same procedure described in Section 4.4. While the base soil was being water-pluviated, it was observed that base soil particles were
passing through the filter, and also that some of them remained trapped in the filter (see Photo A13 in Appendix A). After placement of the base soil was complete, particles continued moving into and through the filter under the influence of gravity alone. No seepage flow was imposed on the specimen.

Axial deformation of the base soil was recorded using a dial gage and a reference rod attached to a perforated plate with a 0.035 mm wire mesh screen. The weight of the perforated plate and the rod applied a contact stress of approximately 0.8 kPa to the top of the specimen. The rate of axial deformation was approximately 0.3 cm/min (see Photo A14 in Appendix A). After sufficient data points were recorded, the axial stress was increased to approximately 2.7 kPa by means of additional dead-weight loading: the rate of axial deformation increased to 1.0 cm/min (see Figure 5.11). A photo taken at the end of the test is shown in Appendix A (Photo A15).

The rate of axial deformation appears to diminish slightly with time under constant load. The rate is greater for the higher load that yields a surcharge of 2.7 kPa. Therefore, it was concluded that this base soil-filter specimen is filter incompatible due to particle migration induced by gravity loading and accentuated by axial loading.

5.4.2 Test 10.5-RW

Test 10.5-0 used a grain size ratio $D_{15}/d_{85}$ of 10.5. No axial load or induced seepage flow was applied to the specimen. Filter and base soil materials were water-pluviated, in the same manner described previously. While the base soil was being placed, base soil particles were again
observed to pass through the filter, with some of them becoming entrapped within the filter.

After finishing water pluviation of the base soil, particles continued to move into and through the filter.

Rate of axial deformation was not recorded as for Test 9.8-RW. Instead, changes in base soil layer thickness were visually noted with reference to a graduated rule pasted to the rigid-wall permeameter. Three hours after placement of the base soil, visual observations revealed the rate of base soil particle migration through the filter had diminished with time, but had not stopped. The test was terminated at this point in time, and the total mass loss of the base soil that passed through the filter was then measured.

The initial thickness of the base soil was estimated to be 45 mm. As mentioned previously, base soil particles started to migrate through the filter during water pluviation, so this initial thickness is considered a conservative value. After three hours, the base soil thickness was approximately 35 mm (see Photo A16 in Appendix A). Accordingly, the thickness of the base soil was reduced by approximately 20 - 25 % due to gravity-driven particle migration.

After the test, the base soil and the filter materials were siphoned off very carefully to a flask. These particles were then dried and weighed. A total mass loss of 20% of the initial mass of the base soil passed through the filter during the test, a value that is in good agreement with the relative change in thickness. Therefore, it was concluded that this base soil-filter specimen is
filter incompatible, notwithstanding the lack of complete passage of the base soil through the filter over the duration of the test.

5.4.3 Tests 11.3-RW, 12.4-RW and 13.3-RW

This series of tests was again carried out using the same procedure used for Test 10.5-RW. After the base soil layers were water-pluviated, time measurements began. In general, it was visually observed that over 90% of the base soil layer passed through the filter, but at different values of elapsed time. As noted in the previous two tests, the rate of base soil particle migration into the filter tended to diminish with time, but it did not completely stop.

The time for approximately 90% of the base soil thickness to pass through the filter was 20, 35 and 6 minutes in Tests 11.3-RW, 12.4-RW and 13.3-RW, respectively (see Photos A17 and A18 in Appendix A). It is important to mention that the same filter particle size range (2.1 – 3.3 mm) was used for Tests 11.3-RW and 13.3-RW, but a different base soil size range (0.15 – 0.21 and 0.12 – 0.18 mm, respectively) particles. There is a good agreement in the findings: the larger the grain size ratio D15/d85, the faster the base soil particles migrated through the filter. Note, the time recorded for Test 12.4-RW was 35 minutes. This finding is tentatively attributed to a smaller particle size range (2.0 – 2.8 mm) of the filter material used in this test: so the voids of the filter layer were smaller and it is postulated the base soil particles required more time to pass through the interstices of the filter.
On the basis of these visual observations, it is concluded that Tests 11.3-RW, 12.4-RW and 13.3-RW may be categorized as catastrophically incompatible due to gravitational influences alone, since it was estimated that most of the base soil particles passed through the filter (> 90 % of mass loss).

5.5 Summary

Twelve tests in total were conducted in this study (see Table 4.1). Seven were tested in the triaxial permeameter using grain size ratios $D_{15}/d_{85}$ smaller than 9 (specifically 7.5 and 8.7), including one test to address issues of repeatability. The other five tests had specimens with grain size ratios $D_{15}/d_{85}$ larger than 9 (specifically 9.8, 10.5, 11.3, 12.4 and 13.3) and were tested in the rigid-wall permeameter.

The test series with grain size ratio $D_{15}/d_{85}$ of 7.5 had a mass loss after reconstitution and consolidation of less than 0.1 %. Values of void ratios after consolidation varied from 0.59 to 0.61. The seepage-induced total axial deformation ($\Delta l$) ranged from 0.4 to 1.8 mm, which was associated with a maximum hydraulic gradient ($i$) around 7.9. The relation between the discharge velocity ($v$) and the hydraulic gradient($i$) was linear at the initial stage of seepage flow($i \approx 2$); however, it became non-linear at hydraulic gradient values ($i$) greater than 2.0. This non-linearity is tentatively attributed to a change in the flow regime from laminar to semi-turbulent to turbulent flow. Experimentally-induced values of the permeability of the two-layer system ($k_{bf}$) ranged from 0.034 to 0.036 cm/sec: there are consistent with computed theoretical values of permeability ($k_v \sim 0.038$ cm/sec). A negligible seepage-induced cumulative total mass
loss of less than 0.4 % was recorded at the end of the multistage seepage flow phase. Accordingly, all tests series with a grain size ratio D\textsubscript{15}/d\textsubscript{85} of 7.5 are deemed to be filter compatible.

The test series with grain size ratio D\textsubscript{15}/d\textsubscript{85} of 8.7 had a mass loss \((m_{rc})\) after reconstitution and consolidation between 1.6 and 5.1 %. Values of void ratios after consolidation varied from 0.54 to 0.59. The seepage-induced total axial deformation \((\Delta l)\) ranged from 11.2 to 23.4 mm, and the maximum hydraulic gradient \((i)\) varied between 2.6 and 10.7. The relation between the discharge velocity \((v)\) and the hydraulic gradient \((i)\) was linear at the initial stage of seepage flow \((i \lesssim 1)\); however, it became non-linear at hydraulic gradient values \((i)\) greater than 1.0. This non-linearity is tentatively attributed to: (i) base soil particle migration into the filter, as implied by the seepage-induced axial deformation; and (ii) the change in flow regime from laminar to semi-turbulent to turbulent flow. Experimentally-induced values of the permeability of the two-layer system \((k_{bf})\) ranged from 0.032 to 0.032 cm/sec: there are consistent with computed theoretical values of permeability \((k_p \sim 0.038 \text{ cm/sec})\). A seepage-induced cumulative total mass loss between 15.4 to 32.3 % was recorded at the end of the multistage seepage flow phase. Accordingly, all tests series with a grain size ratio D\textsubscript{15}/d\textsubscript{85} of 7.5 are deemed to be filter incompatible.

It proved very challenging to reconstitute and consolidate specimens with a grain size ratio of D\textsubscript{15}/d\textsubscript{85} larger than 9 in the triaxial permeameter. Instead they were prepared in a rigid-wall
permeameter, and none were actually subject to seepage flow. All base soil-filter combinations were categorized as filter incompatible; the larger the grain size ratios, the more catastrophically incompatible the specimen. Particle migration induced by a surcharge of less than 3 kPa triggered filter incompatibility in Test 9.8-RW (> 90 % of mass loss). In Test 10.5-RW a mass loss of approximately 20 % passed through the filter. Finally, in tests with grain size ratios of $D_{15}/d_{85}$ larger than 11 (Tests 11.3-RW, 12.4-RW and 13.3-RW), gravitational movement of base soil through the filter led to a nearly complete loss of all the base soil.
### Table 5.1: Test parameters before and after specimen consolidation

<table>
<thead>
<tr>
<th>Test code</th>
<th>Before consolidation</th>
<th>After consolidation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Length of base soil, $L_b$ (mm)</td>
<td>Length of filter, $L_f$ (mm)</td>
</tr>
<tr>
<td>7.5 - 50</td>
<td>51.6</td>
<td>49.1</td>
</tr>
<tr>
<td>7.5 - 150</td>
<td>54.3</td>
<td>49.2</td>
</tr>
<tr>
<td>7.5 - 300</td>
<td>52.8</td>
<td>50.4</td>
</tr>
<tr>
<td>8.7 - 50</td>
<td>52.8</td>
<td>49.1</td>
</tr>
<tr>
<td>8.7 - 150</td>
<td>53.4</td>
<td>49.9</td>
</tr>
<tr>
<td>8.7 - 150 (R)</td>
<td>53.8</td>
<td>49.0</td>
</tr>
<tr>
<td>8.7 - 300</td>
<td>52.5</td>
<td>50.1</td>
</tr>
</tbody>
</table>

$^{(1)}$ $k_v$ from Equation 5.6.

### Table 5.2: Mass loss of the base soil passing through the filter

<table>
<thead>
<tr>
<th>Test code</th>
<th>Mass loss after consolidation, $m_{rc}$</th>
<th>Mass of the base soil layer after consolidation, $M_{bc}$</th>
<th>Cumulative total mass loss at the end of the multistage seepage flow phase</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(g)</td>
<td>(g/m²) ($%$)</td>
<td>(g)</td>
</tr>
<tr>
<td>7.5 - 50</td>
<td>0.1</td>
<td>13 ($0.0$)</td>
<td>608.3</td>
</tr>
<tr>
<td>7.5 - 150</td>
<td>0.7</td>
<td>92 ($0.1$)</td>
<td>666.6</td>
</tr>
<tr>
<td>7.5 - 300</td>
<td>0.2</td>
<td>22 ($0.0$)</td>
<td>634.1</td>
</tr>
<tr>
<td>8.7 - 50</td>
<td>33.4</td>
<td>4337 ($5.1$)</td>
<td>626.2</td>
</tr>
<tr>
<td>8.7 - 150</td>
<td>10.3</td>
<td>1323 ($1.6$)</td>
<td>639.6</td>
</tr>
<tr>
<td>8.7 - 150 (R) $^{(1)}$</td>
<td>19.4</td>
<td>2528 ($3.0$)</td>
<td>631.6</td>
</tr>
<tr>
<td>8.7 - 300 $^{(1)}$</td>
<td>12.3</td>
<td>1570 ($1.8$)</td>
<td>657.5</td>
</tr>
</tbody>
</table>

$^{(1)}$ Values of mass loss during multistage seepage flow before flexible membrane burst.
Figure 5.1: Axial deformation ($\Delta l$) versus hydraulic gradient ($\iota$) - $D_{15}/d_{85}=7.5$

Figure 5.2: Hydraulic gradient ($\iota$) versus time ($t$) - $D_{15}/d_{85}=7.5$
Figure 5.3: Discharge velocity ($v$) versus hydraulic gradient ($i$) - $D_{15}/d_{85}=7.5$

Figure 5.4: Permeability values ($k_{ef}$) versus time ($t$) - $D_{15}/d_{85}=7.5$
Figure 5.5: Cumulative total mass loss ($m_t$) versus hydraulic gradient ($i$) - $D_{15}/d_{85}=7.5$

Figure 5.6: Axial deformation ($\Delta l$) versus hydraulic gradient ($i$) - $D_{15}/d_{85}=8.7$
Figure 5.7: Hydraulic gradient ($i$) versus time ($t$) - $D_{15}/d_{85}=8.7$

Figure 5.8: Discharge velocity ($v$) versus hydraulic gradient ($i$) - $D_{15}/d_{85}=8.7$
Figure 5.9: Permeability values \( (k_{bf}) \) versus time \( (t) \) - \( D_{15}/d_{85}=8.7 \)

Figure 5.10: Cumulative total mass loss \( (m_t) \) versus hydraulic gradient \( (i) \) - \( D_{15}/d_{85}=8.7 \)
Figure 5.11: Axial deformation ($\Delta l$) versus time ($t$) - $D_{15}/d_{85}=9.8$ ($i = 0$)
Chapter 6: Analysis and discussion

A synthesis of the test data is presented in order to better understand the nature of filter incompatibility. More specifically, the experimental findings are analyzed in this chapter to define the onset of filter incompatibility in both a qualitative and quantitative manner. Thereafter, the combined influence of isotropic confining stress and hydraulic gradient, on the onset of filter incompatibility, is examined. This is followed by an assessment of the threshold of the margin of safety governing the filtering criterion. Finally, the experimental findings of this study are compared with previous filter compatibility studies reported in the literature.

6.1 Analysis of the test results

The objective of the soil retention or filtering criterion rule is to restrict migration of base soil particles into the filter without affecting the permeability of the base soil-filter system. In principle, some migration of some base soil particles must be accepted, up to a permissible quantity. Therefore, it is important that gradations of base soil and filter materials are compatible, so that the filter pore size distribution is sufficiently small to restrict the passage of base soil particles into and/or through the filter. Therefore, for purposes of analysis, the onset of filter incompatibility is believed triggered if migration of base soil particles into the filter reduces the permeability of the base soil-filter system, and if the mass loss of the base soil that passes through the filter exceeds a permissible quantity. Figure 6.1 depicts this qualitative definition of filter incompatibility.
In this section, the relation between the discharge velocity \(v\) and the hydraulic gradient \(i\) is analyzed to determine if the non-linearity can be attributed with certainty to a change in flow regime. Thereafter, the change in permeability and the magnitude of mass loss are analyzed to assist in the identification of the onset of filter incompatibility.

6.1.1 Seepage flow regime analysis

In the previous chapter, the non-linearity between the discharge velocity \(v\) and the hydraulic gradient \(i\) was tentatively attributed to a change in flow regime from laminar to semi-turbulent to turbulent flow for tests with a grain size ratio \(D_{15}/d_{85}\) of 7.5. In contrast, for a grain size ratio \(D_{15}/d_{85}\) of 8.7, this non-linearity was tentatively attributed to the combined actions of base soil particle migration into the filter and a change in flow regime. Since tests with \(D_{15}/d_{85} = 7.5\) did not show any evidence of base soil particle migration into the filter (see Figures 5.1 and 5.5), only the flow regime transition hypothesis is examined for this series of tests.

The relation between the discharge velocity \(v\) and the hydraulic gradient \(i\) for tests with \(D_{15}/d_{85} = 7.5\) is shown in Figure 6.2, which is an enlargement of Figure 5.3. The experimentally- deduced permeability \(k_{bf}\) of 0.035 cm/sec, which is an average of the values obtained for tests with \(D_{15}/d_{85} = 7.5\), appears in Figure 6.2 as an aid to identify at what point the relation between discharge velocity \(v\) and hydraulic gradient \(i\) becomes non-linear. It is apparent that, at a hydraulic gradient between 2.5 and 3.0, which is associated with a discharge velocity between 0.08 and 0.10 cm/sec, the relation is non-linear for Tests 7.5-50, 7.5-150 and 7.5-300. To confirm that this non-linearity between the discharge velocity \(v\) and the hydraulic gradient \(i\)
can be attributed to a change in flow regime, the Reynolds number was calculated using Equation 2.4 (see Section 2.4.3). Values for the unit weight of water \( (\gamma_w=9.8 \text{ kN/m}^3) \), viscosity of water \( (\mu=0.001 \text{ kg/(ms)} \) at 20°C) and gravity \( (g=9.8 \text{ m/sec}^2) \) were adopted for all Reynolds number calculations such as the ones estimated for Tests b1-50 and 7.5-50.

Consider the base soil discharge velocities were obtained from Test b1-50 (see Figure D.2 in Appendix D), which used a one-layer specimen of only base soil with a particle size range between 0.12 and 0.18 mm. To calculate the Reynolds number \( (\text{Re}) \) for this test, a minimum and a maximum diameter value \( (D_a) \) of 0.12 and 0.18 mm was adopted. As observed in Figure 6.3, even at the largest discharge velocity of 0.24 cm/sec, which is associated with a hydraulic gradient of 10.3 (see Figure D.1), the Reynolds number varied between 0.3 and 0.4, which means that there was a Darcian (i.e. laminar) flow regime in Test b1-50.

In contrast, Test 7.5-50 consisted of a two-layer specimen with a particle size range from 0.12 to 0.18 mm, and from 1.2 to 1.7 mm for the base soil and the filter, respectively. A Reynolds number was calculated for the filter using value for the diameter of the sphere \( (D_a) \) of 1.2 and 1.7 mm (see Figure 6.3). Recall that the discharge velocity in the filter equals that for the base soil, and hence for the system (see Figure 5.3). The Reynolds number is greater than 1.0 at a discharge velocity approximately between 0.09 and 0.06 cm/sec. The range compares well with that obtained from Figure 6.2 \( (\nu=0.08 - 0.10 \text{ cm/sec}) \) for Test 7.5-50. Therefore, it is concluded that the non-linearity between the discharge velocity \( (\nu) \) and the hydraulic gradient \( (i) \) in Figure 6.2 may be confidently be attributed to a change in flow regime, as supported by: (i) the
Reynolds number calculation; and (ii) the fact that seepage-induced axial deformation and cumulative total mass loss values, which are plotted in Figure 5.1 and 5.5, do not suggest a continuously increasing migration of base soil particles into the filter. Extending the same methodology to all tests in the series established a Reynolds number between 1.0 and 1.7.

### 6.1.2 Definition of the onset of filter incompatibility

A quantitative definition of the onset of filter incompatibility is given in this section, based on a change in the permeability \( k_{bf} \) due to particle migration that occurs in combination with a mass loss of base soil particles through the filter, so that the threshold of the margin of safety of the filtering criterion can be assessed. Particle migration of base soil into the filter changes the permeability of the base soil-filter specimen yielding a change in the relation between the discharge velocity \( \nu \) and the hydraulic gradient \( i \). Another way to analyze the permeability values is by using the concept of relative permeability \( C \) proposed by Cedergren (1977) (see Section 2.4.3 for details), which is defined as follows:

\[
C = \frac{k'}{k} = \frac{k'_{bf}}{k_{bf}}
\]  

(6.1)

Where \( k' \) (or \( k'_{bf} \) in this study) is the experimental permeability value at any time during the phase of multistage seepage flow, and \( k \) (or \( k_{bf} \)) is the permeability value for a flow regime that is laminar. If \( C = 1 \), the flow is laminar; however, if \( C < 1 \), there is a condition of non-Darcian flow.
Migration of the base soil through the filter during multistage seepage flow was quantified by the cumulative total mass loss \( m_e \) (see Section 5.2.7 for details). In addition, the cumulative mass loss per unit area \( m_a \) was computed by:

\[
m_a = \frac{\Sigma m}{A_c}
\]  

(6.2)

Where \( m \) is the seepage-induced mass loss during any stage, and the corrected area of the specimen after consolidation (see Equation 5.4) is denoted by \( A_c \).

6.1.2.1 Relative permeability

The relative permeability \( C \) values for tests with grain size ratio \( D_{15}/d_{85} \) of 7.5 and 8.7 were calculated and plotted in Figure 6.4. The values for \( D_{15}/d_{85} \) of 7.5 started to consistently decrease at hydraulic gradients between 2.5 and 3.0, as anticipated in Figure 6.2. This decrease in relative permeability \( C < 1 \) has been attributed to the change in flow regime from laminar to semi-turbulent to turbulent flow, and was confirmed by Reynolds number (Re) criterion.

In contrast, values of relative permeability \( C \) for tests with a grain size ratio \( D_{15}/d_{85} \) of 8.7 started to consistently decrease at hydraulic gradients between 1.6 and 2.0 (see Figure 6.4), which are significantly less than those for tests with a \( D_{15}/d_{85} \) of 7.5. Seepage-induced axial deformation and cumulative total mass loss values, which are plotted in Figures 5.6 and 5.10, suggest a continuously increasing migration of base soil particles into and through the filter. According to Figure 5.6, seepage-induced axial deformation of the base soil started at hydraulic gradients smaller than 1.6, however, the cumulative total mass loss of base soil \( m_e \) that passed
through the filter (see Figure 5.10) started to increase continuously at hydraulic gradients larger than 1.6. Evidence of base soil particle migration is shown in this sequence, where the base soil particles moved into the filter at first, and then, with additional increments of hydraulic gradient, were noted to pass through the filter and be collected. Therefore, the non-linearity between the discharge velocity \( (v) \) and the hydraulic gradient \( (i) \), and consequently, the decrease in relative permeability values for \( D_{15}/d_{85} = 8.7 \) series of tests can be attributed to the combination of migration of base soil particles into the filter at first, and then to a change in flow regime from laminar to semi-turbulent to turbulent flow, as supported previously.

Consider now the relation between discharge velocity \( (v) \) and hydraulic gradient \( (i) \) for tests with grain size ratio \( D_{15}/d_{85} \) of 8.7 (see Figure 6.5, which is a zoom-in of Figure 5.8). The permeability value \( (k_{bf}) \) of 0.033 cm/sec, which is the average for all tests with \( D_{15}/d_{85} = 8.7 \), appears in Figure 6.5 as an aid to identify at what point the relation between discharge velocity \( (v) \) and hydraulic gradient \( (i) \) becomes non-linear. It is apparent that, at a hydraulic gradient between 1.6 and 2.0, the relation is non-linear. These hydraulic gradients are associated with a discharge velocity of 0.06 cm/sec, which was used to calculate the Reynolds number, which ranged between 0.8 and 1.2. The analysis implies that flow is nearly laminar and that a flow regime transition is imminent. Thus, it is postulated that base soil migration into the filter and the change in flow regime induced the non-linearity between discharge velocity \( (v) \) and hydraulic gradient \( (i) \) and, consequently, reduced the relative permeability to values less than 1 \( (C < 1) \).
It is appropriate, now, to consider the implication of the validity of Darcy’s law as depicted in Figure 2.2. Cedergren (1972) sought to characterize Darcy’s law for semi-turbulent to turbulent flow conditions, using a value of relative permeability (C) for which a definition was postulated (see Section 2.4.3 for details). The effective diameter size ($D_{10}$) of the glass beads used for the D$_{15}$/d$_{85}$ = 7.5 specimen are approximately 0.126 mm (~0.005 inches) and 1.25 mm (~0.05 inches) for the base soil and the filter, respectively. Non-Darcian flow conditions were found for D$_{15}$/d$_{85}$ = 7.5 at hydraulic gradients between 2.5 and 3.0 for the base soil-filter specimens. According to Figure 2.2, semi-turbulent to turbulent flow conditions are expected at hydraulic gradients larger than 0.1 for materials with $D_{10}$ = 0.05 inches. Moreover, it is also observed that the smaller the $D_{10}$, the larger the hydraulic gradient to trigger a change in flow regime. These observations are in complete agreement with the findings of this study. Therefore, a considered interpretation of Darcy’s law is necessary to investigate change in flow regime for the coarse materials used in this study.

6.1.2.2 Mass loss

Migration of base soil particles through the filter can be quantified in terms of mass loss per unit area. Figure 6.6 shows the relation between the mass loss per unit area and the hydraulic gradient ($i$) for all tests with grain size ratio D$_{15}$/d$_{85}$ of 7.5 and 8.7, which were categorized as filter compatible and incompatible, respectively. By way of a better appreciation of the behaviour of the mass loss per unit area at low gradients, consider Figure 6.7, which is enlarged of Figure 6.6.
The mass loss per unit area \((m_a)\) is determined at the hydraulic gradients of 1.6 to 2.0 at which \(C < 1\): these values vary between 750 and 1500 g/m\(^2\), and between 850 and 2800 g/m\(^2\), respectively. Lafleur et al. (1989) report an experimental study base soil-geotextile filter compatibility and proposed a value of 2500 g/m\(^2\) for the onset of incompatibility. It is considered conservative in the context of the current study of granular filters, because geotextiles do not have base soil particle storage capacity that exists in the interstices of a filter. Although somewhat subjective, the lower bound \((m_a \sim 750 - 1500\ \text{g/m}^2)\) range is used hereafter for purposes of discussion.

The data of Figure 6.6 are reproduced in Figure 6.8, with four thresholds of mass loss depicted. The maximum mass loss per unit area of 287 g/m\(^2\), and the range between 750 and 1500 g/m\(^2\) estimated for filter compatible and incompatible tests, respectively, appear in Figure 6.8 as an aid to compare with the value of 2500 g/m\(^2\) proposed by Lafleur et al. (1989). If the value of mass loss obtained for a filter compatible test \((\sim 287\ \text{g/m}^2)\) is compared with the value obtained for Lafleur et al. (1989), the difference between them is approximately a factor of 10 times. In contrast, values obtained for the filter incompatible tests \((\sim 750 - 1500\ \text{g/m}^2)\) yields a difference of approximately 1.5 to 3. It appears the distinction between filter compatible and incompatible base soil-filter systems in this study of granular filters is being made on a range of mass loss per unit area consistent with the study of base soil-geotextile filter systems reported by Lafleur et al. (1989).
Accordingly, the onset of filter incompatibility that is triggered by the critical hydraulic gradient \( i_{cr} \) is suggested to occur when the relative permeability is lower than 1 \( (C < 1) \), in combination with a mass loss per unit area that exceeds 750 to 1500 g/m².

**6.1.2.3 Stress-dependency at the onset of filter incompatibility**

The base soil-filter specimens with a grain size ratio \( D_{15}/d_{85} \) of 8.7 (tests 8.7-50, 8.7-150, 8.7-150(R) and 8.7-300) are considered in order to evaluate any evidence for a stress dependency at the onset of filter incompatibility. Base soil-filter specimens with a grain size ratio \( D_{15}/d_{85} \) of 7.5 are not evaluated because they are filter compatible.

Critical hydraulic gradients \( (i_{cr}) \) between 1.6 and 2.0 were obtained for Tests 8.7-50, 8.7-150, 8.7-150(R) and 8.7-300, respectively. The range of critical hydraulic gradient is very narrow, implying that isotropic confining stress has a little influence on the critical hydraulic gradient. Accordingly, no strong evidence was found that the onset of filter incompatibility is stress-dependent.

**6.1.3 Margin of safety of the filtering criterion rule**

Consider the relation between cumulative total mass loss at the end of a test and the grain size ratio \( D_{15}/d_{85} \) (see Figure 6.9). Values of this cumulative total mass loss for grain size ratios \( D_{15}/d_{85} \) smaller than 9, were measured and reported in Table 5.2. Tests with grain size ratios \( D_{15}/d_{85} \) larger than 10 that were tested in the rigid-wall permeameter, for which a reasonable estimate of cumulative mass loss was possible. In these tests, it was observed that the rate of base
soil particle migration through the filter tends to diminish over time, but there was no evidence that it stopped.

From inspection of Figure 6.9, the following limits may be used to define a threshold for the margin safety: for grain size ratios $D_{15}/d_{85} < 8$, the base soil-filter specimen is compatible; for grain size ratios $8 < D_{15}/d_{85} < 9$, the base soil-filter specimen is incompatible if the critical hydraulic is exceeded; and for grain size ratios $D_{15}/d_{85} > 9$, the base soil-filter specimen is incompatible.

6.1.4 Summary of the experimental findings

From analysis of the test results, the experimental findings are summarized as follows:

- Qualitative definition of the onset of filter incompatibility: a base soil-filter specimen is filter incompatible if migration of base soil particles into the filter reduces the permeability of the base soil-filter system, and if the mass loss of the base soil that passes through the filter exceeds a permissible level.

- Quantitative definition of the onset of filter incompatibility: a base soil-filter specimen is filter incompatible if the relative permeability ($C$) is lower than 1, and if the mass loss per unit area exceeds 750 to 1500 g/m$^2$.

- On a threshold to the margin of safety for soil retention: for a grain size ratio $D_{15}/d_{85} < 8$, the base soil-filter specimen is compatible; for a grain size ratio $8 < D_{15}/d_{85} < 9$, the base soil-filter specimen is incompatible if a critical hydraulic gradient ($i_{cr}$) in the
approximate range 1.6 and 2.0 is exceeded; and for a grain size ratio $D_{15}/d_{85} > 9$, the base soil-filter specimen is incompatible.

- There is no strong evidence that the onset of filter incompatibility was influenced by confining stress in the range 50 to 300 kPa.

6.2 Discussion of the experimental findings

One of the challenges of comparing the experimental findings of this study with others reported in the literature is the absence of any standard equipment or test procedure for filter compatibility experiments. In this study, a new flexible-walled (triaxial) permeameter was designed and commissioned to test different grain size ratios of base soil-filter specimens, which were reconstituted using uniformly-graded glass beads. Comparison and discussion of this study’s finding with previous filter compatibility studies are based on three main aspects: (i) definition of the onset of filter incompatibility; (ii) stress-dependency at the onset of filter incompatibility, and, (iii) margin of safety of the filtering criteria rule ($D_{15}/d_{85} \leq 4$).

6.2.1 Filter incompatibility

The question of whether a base soil-filter specimen is filter compatible is a common objective of every study. In reality, studies reported in the literature review have different definitions of filter incompatibility. Table 6.1 provides a summary of those definitions (qualitative and quantitative), and as well as a note on the test parameters used to define filter incompatibility of those studies that have influenced the current filter design.
It is apparent that previous studies define the onset of filter incompatibility based on only one test parameter, either the permeability of the base soil-filter system or the mass loss of the base soil. The intent of this study has been to use both test parameters (relative permeability and mass loss per unit area), in a more unified approach, to define the onset of filter incompatibility in a quantitative manner and thereby with greater confidence.

6.2.2 Stress-dependency

It is believed, the study of Tomlinson and Vaid (2000) is the only one to-date that has evaluated if the onset of filter incompatibility is stress-dependent. They found that increasing effective stress had a minor negative influence on base soil-filter compatibility: it was attributed to an increase in stress causing the arches formed in the filtration zone be more susceptible to collapse. The findings of this showed that the influence of stress had a minor negative impact at the onset of filter incompatibility. Direct comparison between the studies remains challenging because the definition of the onset of filter incompatibility was different for both studies (see Table 6.1), and the findings are based on experiments in a flexible-walled versus a rigid-walled test device.

More specifically, the differences between the Tomlinson and Vaid (2000) studies the current study may be related to use of different equipment and definition of filter incompatibility, the change in flow regime, specimen thickness and measurement of head loss. This aspect deserves further investigation, and identifies the need for a standard approach to experimental investigation of the phenomenon.
6.2.3 Implications for the state-of-practice

The margin of safety related to the filtering criterion proposed for soil retention by Terzaghi (1939) has been evaluated in this study using both the new triaxial permeameter and a rigid-wall permeameter. Other studies, such as Bertram (1940), Sherard et al. (1984a), and Tomlinson and Vaid (2000), have also explored the margin of safety of this criterion. All of them agree on the premise that Terzaghi’s (1939) rule is conservative for uniform base soils.

Bertram (1940) concluded that the minimum critical grain size ratio $D_{15}/d_{85}$ at the limit of stability is approximately 6. However, only 2 out of 30 tests performed on this study suggested a grain size ratio $D_{15}/d_{85}$ of 6 at the limit of stability; the other tests showed a grain size ratio $D_{15}/d_{85}$ greater than 8.7 at the limit of stability. It is important to mention that Bertram (1940) did not take into account stress and did not suggest any margin of safety for the criterion. Moreover, the conclusion was based on testing at a constant hydraulic gradients between 6 and 8, else between 18 and 20, across the specimen.

Sherard et al. (1984a) concluded that “for filters with $D_{15}$ larger than about 1.0 mm, the ratio $D_{15}/d_{85} \leq 5$ should be continued as the main criterion for judging filter acceptability.” Lafleur (1984) found that the critical piping ratio (or grain size ratio) $D_{15}/d_{85}$ above which filter incompatibility occurred was about 8.4 for broadly-graded cohesionless tills. Tomlinson and Vaid (2000) found that, while base soil-filter systems with ratios $D_{15}/d_{85} < 8$ are filter compatible, those with a ratio $D_{15}/d_{85}$ between 8 and 12 were filter incompatible if the critical gradient was reached, and those with a ratio $D_{15}/d_{85} > 12$ resulted in spontaneous failure.
The findings of the current study in the triaxial permeameter suggest that, for grain size ratios $D_{15}/d_{85} < 8$, the base soil-filter specimen is filter compatible, as Bertram (1940) and Tomlinson and Vaid (2000) have also reported. Furthermore, for a grain size ratio $D_{15}/d_{85} > 9$, it was found that unacceptable base soil migration occurred, a finding that is also in agreement with Tomlinson and Vaid (2000). For a grain size ratio $D_{15}/d_{85}$ between 8 and 9, Tomlinson and Vaid (2000) and this study both agree that the base soil-filter specimen is filter incompatible if a critical hydraulic gradient is exceeded. The influence of stress, and the value of critical hydraulic gradient, appears very sensitive to the definition of filter incompatibility.
<table>
<thead>
<tr>
<th>Table 6.1: Comparison of filter incompatibility definitions from previous studies</th>
</tr>
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<tbody>
<tr>
<td><strong>Definition of the onset of filter incompatibility (qualitative)</strong></td>
</tr>
<tr>
<td>Bertram (1940)</td>
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<tr>
<td>Sherard et al. (1984a)</td>
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Figure 6.1: (a) specimen before seepage flow is imposed; (b) filter compatible test; (c) filter incompatible test; (d) catastrophic filter incompatible test.

Figure 6.2: Discharge velocity ($v$) versus low hydraulic gradient ($i$) - $D_{15}/d_{85}=7.5$
Figure 6.3: Discharge velocity ($v$) versus Reynolds number ($Re$).

Figure 6.4: Relative permeability ($C$) values versus hydraulic gradient ($i$).
Figure 6.5: Discharge velocity ($v$) versus low hydraulic gradient ($i$) - $D_{15}/d_{85}=8.7$

Figure 6.6: Mass loss per unit area ($m_a$) versus hydraulic gradient ($i$)
Figure 6.7: Mass loss per unit area ($m_a$) versus hydraulic gradient ($\iota$)

Figure 6.8: Mass loss per unit area ($m_a$) versus low hydraulic gradient ($\iota$)
Figure 6.9: Total mass loss at the end of the test (%) versus grain size ratios ($D_{15}/d_{85}$)
Chapter 7: Concluding remarks and recommendations

In this experimental study, base soil-filter compatibility has been examined in specimens with grain size ratios D$_{15}$/d$_{85}$ between 7.5 and 13.3, using both a triaxial permeameter, and a rigid-wall permeameter. Moreover, the onset of base soil-filter incompatibility has been defined in both a qualitative and quantitative manner using two independent parameters (relative permeability and mass loss). This enabled stress-dependency to be evaluated at the onset of filter incompatibility. Finally, the margin of safety of the filtering (or soil retention) criterion for uniformly-graded materials was investigated.

7.1 Concluding remarks

The following conclusions are made from the literature review, the triaxial permeameter design and operation, and the test results obtained from testing of uniformly-graded glass beads:

- The current design guidelines for a granular filter are mainly founded on four experimental studies: Bertram (1940), Karpoff (1955), Lafleur (1984) and Sherard et al. (1984a, 1984b and 1989). All these studies used a total of 68 different types of base soil materials to support the current filter design guidelines.

- A new stress-controlled (triaxial permeameter) test device was developed to investigate the onset of filter incompatibility subjected to multistage seepage flow, after consolidating base soil-filter specimens under an isotropic confining stress. In addition, the measured of head loss took into account energy losses in order to determine values of...
hydraulic gradient with considerable accuracy. Volume change of specimen due to consolidation was also measured, so that the void ratio of the test specimen could be estimated before imposition of seepage flow.

- A new and systematic test procedure was developed for this new triaxial permeameter. Systematic controls over the three phases of the testing procedure were performed in order to assure a good quality of data. These three phases were: specimen reconstitution, specimen consolidation and multistage seepage flow. Complementary tests were performed on a rigid-wall permeameter.

- The test method and procedure developed for the triaxial permeameter yield repeatable data.

- It is proposed that the onset of filter incompatibility be defined by a value of relative permeability \( C \) less than 1, and a mass loss per unit area that is greater than 750 to 1500 g/m².

- Although the current study finds no strong evidence that the onset of filter incompatibility is stress-dependent, further investigation is believed necessary and should be undertaken in the context of a quantitative (not qualitative) definition of filter incompatibility.
Experimental observations in the stress range 50 to 300 kPa suggested that: for grain size ratios $D_{15}/d_{85} < 8$, the base soil-filter specimen is compatible; for grain size ratios $8 < D_{15}/d_{85} < 9$, the base soil-filter specimen is incompatible if a critical hydraulic gradient ($i_{cr}$) is exceeded; and for grain size ratios $D_{15}/d_{85} > 9$, the base soil-filter specimen is incompatible, and the larger the grain size ratio, the more severe the gravitational segregation without any seepage flow.

In summary, the most significant contributions of this study are: (i) the onset of filter incompatibility is confirmed to be very sensitive to the effect of the grain size ratio $D_{15}/d_{85}$, (ii) filter incompatible specimens were found impossible to test in the triaxial permeameter, so the use of the rigid-wall permeameter is still considered necessary, and (iii) the role of stress at the onset of filter incompatibility should be further investigated for a narrow range of grain size ratios $D_{15}/d_{85}$ between 7.5 and 8.7.

### 7.2 Limitations of current study

The findings of this study are considered applicable to uniformly-graded materials. Although, the repeatability of the test method showed consistent laboratory test results, there is some opportunity to improve upon the new triaxial permeameter that was designed and commissioned specifically for this study. The current limitations of the triaxial permeameter set-up are: (i) there was no measurement of the pore pressure parameter B (see ASTM D2487-11) during consolidation to confirm specimen saturation, (ii) the measurement resolution of the specimen volume change is believed too coarse, if finer materials are to be tested, and (iii) the
measurement of the flow rate and mass loss was only recorded at discrete intervals and not continuously. Recommendations on how to overcome these limitations are listed in the next section.

During the planning of the test program for this study, it was decided to investigate a relatively wide range of grain size ratio $D_{15}/d_{85}$ between 7.5 and 13.3. The findings show the onset of filter incompatibility to occur in a very narrow grain size ratio $D_{15}/d_{85}$ between 7.5 and 8.7, and little was learned from the test at $D_{15}/d_{85} = 13.3$.

7.3 Recommendations

The following recommendations are made from experience gained in this study, with emphasis placed on improvements to the triaxial permeameter and future experimental work.

In further research, it is recommended the triaxial permeameter have the following characteristics:

- Add another total pressure transducer (TPT) to measure pore water pressure within the specimen, under undrained conditions, to establish the pore pressure parameter B, and hence confirm the degree of saturation of the specimen.
- Add another differential pressure transducer (DPT) to improve the volume change measurement resolution during specimen consolidation.
- If measurements of volume change of the specimen during and after multistage seepage flow are required (to calculate void ratios), then add another DPT in order to facilitate
measurement of volume change in the triaxial chamber itself, which can be correlated with mass loss and axial deformation of the specimen.

- Add a flow meter and an optical sensor to characterize the flow rate and the mass loss continuously, during each phase of multistage seepage flow.
- Include tests on real soils (i.e. non-spherical grains) to investigate the influence of grain shape on the onset of incompatibility in uniformly-graded base soil and filter materials.

Additional series of tests are recommended at grain size ratios between $D_{15}/d_{85}$ 7.5 and 8.7 to evaluate the stress-dependency of the onset of filter incompatibility, and also to refine the threshold of the margin of safety of the filtering criterion. Finally, the use of this triaxial permeameter can be expanded to investigate the base soil-geotextile filter compatibility, and also to investigate the phenomenon of seepage-induced internal stability.
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Appendices
Appendix A: Selected photographs of the experimental testing
Photo A1  Plan view of the ports (located on the base frame) and the collection trough

Photo A2  A coating of silicone grease applied over the base soil-filter interface
Photo A3  Rigid-wall permeameter set up

Photo A4  Glass beads (0.12 to 0.18 mm diameter)
Photo A5  Glass beads (1.4 to 2.0 mm diameter)

Photo A6  Base pedestal with perforated plate and wire mesh screens
Photo A7 Membrane expander mounted over the base pedestal

Photo A8 Top cap reservoir mounted over the membrane expander
Photo A9  Container with the top cap submerged inside the top cap reservoir

Photo A10  Test specimen under vacuum pressure
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Photo A12    Base soil-filter specimen before membrane failure (Test 8.7-150(R))
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Photo A14  Axial deformation (Test 9.8-RW)
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Photo A16  Base soil layer thickness (end of Test 10.5-RW)
Photo A17    Base soil layer segregation (Test 11.3-RW)

Photo A18    Base soil layer segregation (Test 13.3-RW)
Appendix B: Specimen reconstitution procedure
B.1 Introduction

The objective of the specimen reconstitution technique is to create a two-layer saturated specimen, with the same homogeneity and density. This two-layer specimen comprises the filter and the base soil. Both layers are saturated and deposited using the water-pluviation technique (Shi, 1993). The filter is situated beneath the base soil layer. Once each layer is placed, top leveling is performed, after which, the reference height measurement is recorded. Details of the specimen reconstitution procedure are described in the following sections.

B.2 Saturation of the sample

The filter and the base soil layers are prepared independently. A dry mass of not less than 700 g of each size of glass beads is weighed and then placed in flasks to be boiled for approximately 30 minutes. Therefore, the total mass of filter \( M_f \) and base soil \( M_b \) is known. After cooling to room temperature, the flasks are placed under vacuum pressure (-70 to -80 kPa) for not less than 12 hours to release any last entrapped air.

B.3 Initial reference height

Before placement of the specimen, the initial reference height measurement is recorded using a height measuring device, made of stainless steel (Shi, 1993). This device comprises four pieces: a perforated plate with a wire mesh screen glued to one side and three detachable rods. The longest detachable rod is used to record the initial reference height \( R_o \). Figure B.1 (a) shows a schematic view of this step. The other two detachable rods are used to record the reference height of the filter and base soil, respectively, once they are water-pluviated.
B.4 Specimen placement

The water-pluviation technique (Shi, 1993) used to reconstitute the specimen is well-suited for uniform sands (Vaid and Negussey, 1988); therefore it can be used for uniform glass beads as well. The flasks containing the saturated glass beads are filled with de-aired water up to the top and turned upside-down inside the top cap reservoir. As the glass beads pluviate downward into the expander, water rises up into the flask in a process of exchange. After the filter material is deposited, top leveling is performed, and then the high reference is recorded. The base soil is then water-pluviated, and the procedure repeated. Figure B.1 (b) shows a schematic view of the water pluviation technique.

B.5 Top leveling

After either the filter or the base soil layer is deposited inside the expander, top leveling is performed in order to create a level surface for each layer. This is accomplished by siphoning off the excess glass beads from the filter ($\Delta m_f$) and the base soil ($\Delta m_b$) layer, as shown in Figure B.1 (c). The excess glass beads are dried and weighed.

B.6 Filter and base soil reference height measurements

After the filter material is deposited and the top surface is leveled by siphoning, the filter reference height is recorded ($R_f$). Similarly, the base soil reference height is recorded ($R_b$), establishing the length of the base layer ($L_b$). Thereafter, the top cap is placed over the specimen and another reference height measurement is recorded ($R_{tc}$). This is used to calculate the length
of the base layer after the top cap \( L_{b(tc)} \) is placed over the base layer. Figure B.1 (d) shows how the reading \( R_{tc} \) is taken.

### B.7 Specimen length calculations

As shown in Figure B.1, once the height measurement device is in position, the dial gage readings are recorded. The length of the specimen after specimen reconstitution is the sum of the length of the filter and the base soil layer. The calculation of the length of both layers is made as follows (Shi, 1993):

\[
L_f = (R_f - R_o) + (l_{rf} - l_{ro})
\]

\[
L_b = (R_b - R_f) + (l_{rb} - l_{rf})
\]

\[
L_{b(tc)} = (R_{tc} - R_f) + (l_{rte} - l_{rf})
\]

Since,

\[
L_{b(tc)} \approx L_b
\]

Therefore,

\[
L_o = L_f + L_{b(tc)}
\]

Where:

- \( L_f \) = length of the filter layer (mm)
- \( R_f \) = dial gage reading after the filter (f) is deposited and leveled (mm)
- \( R_o \) = dial gage reading before the filter is deposited (mm)
- \( l_{rf} \) = length of the detachable rod after the filter layer is deposited (163.1 mm)
- \( l_{ro} \) = length of the detachable rod before any layer is deposited (213.3 mm)
\[ L_b = \text{length of the base soil (b) layer before the top cap is in placed (mm)} \]
\[ R_b = \text{dial gage reading after base soil (b) layer is deposited and leveled (mm)} \]
\[ l_{rb} = \text{length of the detachable rod after the base soil layer is deposited (113.3 mm)} \]
\[ L_{b(tc)} = \text{length of the base soil (b) layer after the top cap (tc) is placed (mm)} \]
\[ R_{tc} = \text{dial gage reading after the top cap (tc) is placed over the base soil layer (mm)} \]
\[ l_{rtc} = \text{length of the rod (including perforated plate) plus the top cap length (126.6 mm)} \]
\[ L_o = \text{length of specimen after reconstitution (mm)} \]

Therefore, the length of the specimen \((L_o)\) can be calculated after the reconstitution. A description of how this length changed due to consolidation is given in Appendix C.
Figure B.1: Specimen reconstitution technique: (a) initial reference height measurement; (b) water pluviation technique; (c) top leveling by siphoning; (d) top cap reference height measurement
Appendix C: Void ratio calculation
C.1 Introduction

The calculation of the void ratio is summarized, based on the geometry (cross section area and length) and total mass of the specimen in the triaxial permeameter. Two phases are distinguished: (1) Phase I is related to specimen reconstitution; and (2) in Phase II, the specimen is consolidated first under vacuum pressure and then, as part of the test procedure, under a cell pressure. It is important to mention that void ratio during and after each stage of multistage seepage flow is not calculated.

C.2 Phase I – Specimen reconstitution

Specimen reconstitution is as previously described in Appendix B. After the two-layer specimen is placed in the expander, the length of the specimen ($L_o$) due to reconstitution is calculated. Since the bottom drain is closed, it is possible that some base soil particles may pass through the filter and be retained inside the base pedestal. It is not possible to estimate this component (if any) of the mass loss due to reconstitution ($\Delta m_r$).

C.3 Phase II – Specimen consolidation due to vacuum pressure

After a vacuum pressure of -18 to -20 kPa is applied on the specimen, the glass beads respond by rearranging themselves into a tighter packing, which causes a decrease in specimen volume, and water is expelled by drainage through the inlet port located on the top cap (see Figures 3.1 to 3.3). This volume discharge is collected and measured in a graduated pipette. The following measurements are made during phase II:
• Change in length of the specimen due to consolidation by vacuum pressure \( (\Delta L_v) \) is recorded using the reference height device (see Figure B.1 (d) in Appendix B):

• Volume change \( (\Delta V_v) \) is recorded.

• Mass loss due to consolidation under vacuum pressure \( (\Delta m_v) \) is not able to be quantifiable due to the fact that the bottom drain is still closed.

C.4 Phase II – Specimen consolidation due to confining stress

Specimens are subjected to different magnitudes of confining stress (e.g. 50, 150 and 300 kPa) in this study. After the consolidation phase is over, the valve connected to the pipette is closed and the bottom on the base pedestal (see Figure 3.1 and 3.2) drain is opened. Any base soil particles can be collected, dried and weighed, as described in Section 3.3.5. The following measurements are made during this phase:

• Change in length of the specimen due to consolidation by the confining stress \( (\Delta L_c) \), using the LVDT mounted on the loading ram (see Figure 3.1).

• Volume change \( (\Delta V_c) \).

• Mass loss due only to consolidation under confining stress \( (\Delta m_c) \) cannot be quantified; nonetheless, it is possible to establish the mass loss \( (m_{rc}) \) due to the combined effect of specimen reconstitution and consolidation.
C.5 Parameters calculated after specimen consolidation

After the process of specimen reconstitution and consolidation is over, values of length, mass loss and volume change of the specimen are obtained. It is important to mention that these values are calculated before multistage seepage flow is applied.

Length of the specimen after consolidation (L)

The initial length of the specimen prior to seepage flow is calculated as follows:

\[ L = L_o - \Delta L_v - \Delta L_c \]

Where:

\[ L \] = length of the specimen after consolidation (mm)
\[ L_o \] = length of the specimen after prior to consolidation (mm)
\[ \Delta L_v \] = change in length of specimen after consolidation due to vacuum pressure (mm)
\[ \Delta L_c \] = change in length of specimen after consolidation due to confining stress (mm)

Mass of the specimen after consolidation (M)

The initial mass of the specimen is calculated as follows:

\[ M = (M_f - \Delta m_f) + (M_b - \Delta m_b - m_{rc}) \]

\[ m_{rc} = \Delta m_r + \Delta m_v + \Delta m_c \]

\[ M_{bc} = M_b - \Delta m_b - m_{rc} \]

Where:

\[ M \] = mass of the specimen after consolidation (g)
As mentioned previously, it is possible to quantify the mass loss after specimen reconstitution and consolidation; however, it is not possible to quantify the mass loss for each phase.

**Void ratio (e)**

The void ratio is estimated for three reasons: (1) to demonstrate the reproducibility of testing; (2) to use as a correlation with permeability values; and (3) to establish if specimens are in a relatively loose or dense state. In order to calculate the void ratio (e), it is necessary to determine first the unit dry weight (\( \gamma_d \)):

\[
\gamma_d = \frac{Mg}{V} = \frac{Mg}{(V_0 - \Delta V_b - \Delta V_c)}
\]

Where:

\( M \) = mass of the specimen after consolidation (g)
\[ g = \text{gravity (9.81 m/s}^2) \]
\[ V = \text{volume of the specimen after consolidation (cm}^3) \]
\[ V_o = \text{volume of the specimen before consolidation (cm}^3) \]
\[ \Delta V_v = \text{volume change of the specimen due to consolidation under vacuum pressure (cm}^3) \]
\[ \Delta V_c = \text{volume change of the specimen due to consolidation under confining stress (cm}^3) \]

Then, void ratio (e) can be calculated:

\[ e = \frac{G \gamma_w}{\gamma_d} - 1 \]

Where:

\[ G = \text{specific gravity of the glass beads (2.5)} \]
\[ \gamma_w = \text{unit weight of water (9.81 kN/m}^3) \]
Appendix D: Preliminary tests
D.1 Introduction

Two identical specimens (Test b1-50 and b2-50) were tested under the same conditions as a preliminary assessment of the triaxial permeameter operation, performance and repeatability. The test procedure involved slight variations to the procedure mentioned in Section 4.4, which is described below. The specimens were reconstituted with only one size range of glass beads (0.12 to 0.18 mm) which is that of the base soil in the main test program.

D.2 Procedure used during commissioning tests

The test procedure was very similar to that described in Section 4.4. There were some differences due only to the fact that a single-layer specimen was reconstituted and tested, and that the system set up was meant to avoid any mass loss during the multistage seepage flow.

During reconstitution, the specimen was deposited inside the expander as a single layer using the water pluviation technique. A 35 μm wire mesh screen was placed on the base pedestal in order to prevent any mass loss due to specimen consolidation and unidirectional seepage flow.

During the multistage seepage flow, the test procedure was the same as described in Section 4.4, with the exception that the rate of increase in total head across the specimen is slightly different. An increment of 2 cm was applied for a difference in total head was between 0 and 10 cm, 50 and 70 cm, and 170 to 184 cm; all other increments were 10 cm (see Figure D.1). These two rates assisted in evaluating the behavior of the one-layer specimen over the entire range of difference in total head that can be imposed using the test set-up, and also in establishing head
losses in the system. Experience gained led to improvements that were made to the test device and also to the test procedure.

D.3 Test results

Both specimens were consolidated under a cell pressure of 50 kPa, and multistage seepage flow then imposed in a downward direction. A void ratio of 0.68 was measured obtained for both specimens, which confirms the reproducibility of the reconstitution procedure. The maximum hydraulic gradients \((i)\) of 10.2 and 10.7 (see Figure D.1) were associated with a difference in total head \((\Delta h_t)\) of 184 cm, and with a head loss \((\Delta h_s)\) of 101 cm and 105 cm, respectively. The resulting difference of \(\Delta h_t - \Delta h_s = 83\) and 79 cm is attributed to energy losses, respectively. Figure D.2 presents the relation between the discharge velocity \((v)\) and hydraulic gradient\((i)\), which is linear. This linearity is attributed to laminar flow through the specimen. Permeability values for this base soil \((k_p)\) material of approximately 0.02 cm/sec were deducted for both tests (see Figure D.3).
Figure D.1: Hydraulic gradient ($i$) versus time ($t$)

Figure D.2: Discharge velocity ($v$) versus hydraulic gradient ($i$)
Figure D.3: Permeability values ($k_b$) versus time ($t$)