A LABORATORY STUDY OF PARTICLE MIGRATION IN COHESIONLESS SOILS

by

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B.Sc, University de Chile, 1996

A THESIS SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF APPLIED SCIENCE

in

THE FACULTY OF GRADUATE STUDIES

Department of Civil Engineering

We accept this thesis as conforming to the required standard

THE UNIVERSITY OF BRITISH COLUMBIA

October 2002

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Vancouver, Canada

Date OCTOBER 11, 2002

DE-6 (2/88)

ABSTRACT

Filters represent a very important component of earth structures. Indeed a large percentage of embankment dam failures have involved piping or soil migration. A key aspect of filter design is evaluating the potential for internal instability.

The onset of internal stability in potential unstable soils is governed by geometric and hydrodynamic constraints. Interpretations of laboratory studies o reconstituted specimens have led, in the last 25 years, to empirical criteria that define a threshold to the onset of instability. The development of those empirical criteria is reviewed. New laboratory data are then presented, and compared with selected data reported in the literature. The new data describes the response of five soil gradations to unidirectional seepage flow, at a low confining stress, from testing in a rigid walled permeameter (a Gradient Ratio device). Test variables examined include the influence of hydraulic gradient, vibration of the specimen, and opening size of the supporting wire mesh screen. The results broadly confirm the relevance of the empirical design criteria which, it is noted, address only geometric constraints to internal stability. In practice, concern exists for the risk posed by seepage flow through a potentially unstable soil: a need remains to better address hydrodynamic influences, and resulting total loss of soil.

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A	Cross section of the permeameter
C_U	Coefficient of uniformity
Dn	Grain size corresponding to n% finer
e	Void ratio
F	Mass fraction smaller than D
f	Frequency
GR	Gradient Ratio
G_S	specific gravity
H	mass fraction measured between particles sizes D and 4D (Kenney & Lau, 1985)
h	Water head
h_{ij}	Head difference between ports i and j
i	Hydraulic gradient
i_{ij}	Hydraulic gradient between ports i and j
k	Hydraulic conductivity
k_{ij}	Hydraulic conductivity between ports i and j
L	Soil specimen length
LVDT	Linear variable differential transformer
Q	volumetric flow rat of water
R_e	Reynolds number
γ_w	Unit weight of water
ρ	Density

ACKNOWLEDGMENTS

First and foremost, I thank my wife for her support, and love throughout this research.

I am genuinely grateful to my supervisor Dr. Jonathan Fannin for his direction, advice, and enthusiastic encouragement. Additionally, I would like to express my sincere gratitude to Mr. Stephen Garner, Head of the Geotechnical area of BC Hydro for his continued support.

I also thank Dr. D. Wijewickreme for his significant comments on the draft of this thesis.

I thank all of the University support staff who influenced my research, and in particular the staff of the Civil Engineering workshop.

This research is funded by through the Professional Partnership program between BC Hydro and UBC Civil Engineering. I wish to express my thanks for the financial support provided. Additional funding, for modifications to the test equipment, came from the Natural Sciences and Engineering Research Council of Canada (NSERC).

Finally, I thank my parents who have always encouraged me to try my best in any activity. This thesis is dedicated to the memory of my father.

1.0 Introduction

1.1 Seepage flow in soils

Seepage OF ground water exerts a force on soil particles, which acts in the direction of flow. The seepage force per unit volume of soil is given by "$i\gamma_w$", where "i" is the hydraulic gradient. The velocity of flow induces a drag force on individual particles, with a potential to dislodge them from the soil matrix and to cause a rearrangement or migration of fines through voids between the larger grains.

An important aspect of groundwater seepage is that of filter stability at the interface of two adjacent soils. The restriction of soil movement from a soil ("base" soil) into or through and adjacent medium (the "filter" soil) as a consequence of seepage flow has been studied for almost a hundred years (Terzaghi, 1922). There is a considerable body of experience with the use of granular soils as a filter material, and that with the use of geotextiles is steadily growing. The filtration process depends entirely on the formation of a stable interface between the base soil and the filter material, termed a "self-filtering layer". To ensure this stable interface, it is necessary to satisfy appropriate design criteria. Such design criteria are empirical and are typically derived from experimental studies on reconstituted laboratory test specimens. Aspects that must be considered in the application of design criteria are those of geometric conditions (constriction size), hydraulic conditions (hydraulic gradient, seepage velocity, and unidirectional or reversing flow), and any potential for vibration conditions (earthquake-induced or other ground movements).

1.2 Soil Migration due to seepage

Resistance to soil migration is typically derived from cohesion of the fines or an impediment to the movement resulting from the structure of the pore size constrictions. Hence, the gradation of the soil exerts a major influence on the resistance to migration. This resistance has also been called "inherent stability" (U.S. Army Corps of Engineers, 1941).

Different terms have been proposed to distinguish between forms of soil migration (see Figure 1.1). Some of the terms that have been used in geotechnical practice are:

- internal, external, or contact "suffusion";
- internal, external, or contact "erosion";
- "piping"; and,
- "suffosion".

The term *"suffusion"* refers to the phenomenon where seepage water removes fine particles without destroying the soil structure (Kezdi, 1979). Hence, i*nternal suffusion* has been used to describe a movement of fines within the soil that affects only the local permeability. In contrast, *external suffusion* occurs at the free surface where fine particles are carried from one layer (e.g. base soil) into an adjacent layer (e.g. filter).

The term *"erosion"* has been used where the structure or skeleton of the soil is affected. In addition to migration of fine particles, there is also a movement of larger particles from the soil matrix. *Internal erosion* may lead to formation of an open conduit through the soil (piping). This may also occur when cavities already exist in the skeleton, leading to high seepage velocities. *External erosion* may occur at the ground surface of an earth structure, due to high exit gradients and the resultant seepage force. *Contact erosion* is similar to contact suffusion, but there is migration of particles belonging to the soil "skeleton".

As mentioned, the term *"piping"* has been used in the same sense as internal erosion and usually relates to erosion around a specific interface locality such as tube, cable, instrument or other matrix features that promotes arching, or leaves free spaces within the surrounding soil.

The term *"suffosion"* also has been used in the same sense of erosion. Therefore, it could be defined as migration of the fine fraction simultaneously with the coarse fraction ("skeleton"). On the other hand, and somewhat confusingly, it has been also used to define the transport of small particles from a soil (Kenney and Lau, 1985).

1.3 Seepage flow in soil structures

When dealing with seepage flow of water, it is important to ensure that each of the material layers or zones of earth-fill is internally stable under the expected field conditions (see Figure 1.2). Moreover, it is necessary to check the interface compatibility of those materials by considering the behaviour of each one separately and then the interface. It is for these reasons that empirical techniques have been developed both to assess the internal stability of a soil as well as to verify the filter requirements of adjacent soils.

In filtration design, the empirical criteria are usually intended to satisfy two limiting conditions:

1. Soil particles from the base layer should not pass through the adjacent filter material, to ensure soil retention; and,
2. The permeability of the filter should exceed that of the base soil, to ensure a decrease in pore water pressures.

It is very important to consider the internal characteristics of the soil. Filter, transition and base soil must be stable under the conditions imposed by the application. Most of the filter criteria do not consider the complex behaviour of soil in the field. It is of fundamental importance to have a method, that can be used with confidence, to predict whether or not a soil would be internally stable under given field conditions.

The most important factor governing the behaviour of filters is the grain size difference between the filter and the base soil in contact with that filter. A common design criterion is the grain-size ratio, used as an index of compatibility. The grain-size ratio commonly used is D_{15}/d_{85}, where D_{15} is the size such that 15 % of the particles of the filter are of a smaller diameter. Similarly, d_{85} is the size such as 85 % of the particles of the base are of a smaller diameter.

Other factors that may influence interface stability against soil migration are:

- the magnitude of confining stress;
- density of the materials;
- filter thickness;
- magnitude of hydraulic gradient;
- change in seepage flow conditions; and
- soil structure.

Many of these factors are very difficult to determine, and depend on construction factors such as segregation potential and method of placement of the materials. Consequently, it is difficult to accurate by anticipate the distribution of soil permeability in different zones of an earth structure, such as an embankment dam, and hence the maximum hydraulic gradients. For this reason, caution must be exercised in selecting appropriate parameters for use in any simulation of filter-base soil interaction using laboratory tests (see Figure 1.3), to ensure the field soil structure is adequately represented.

1.4 Purpose and scope of the study

The primary purpose of this study is to review and improve the confidence in use of criteria proposed by Kezdi (1979) and Kenney & Lau (1985, 1986) to evaluate the internal stability of soils. There have been some concerns, based on performance monitoring of field structures, that refined criteria might be required. A modified Gradient Ratio test device was used to perform a series of permeameter tests using both glass beads and a cohesionless soil obtained from a borrow pit for the Bennett Dam, British Columbia. A multi-stage test procedure was developed to assess whether a soil is internally stable or not. Results of this study were compared with the findings of others for similar conditions (notably Kenney and Lau, 1985 and Honjo et al., 1996). Emphasis was placed on the repeatability and consistency of these experimental studies, and the use of empirical design criteria in geotechnical practice.

1.5 Organization of the thesis

In Chapter 2, a review is given of selected filtration studies reported in the literature. Analysis examines the differences between various laboratory tests, and factors that influence the filtration behaviour of soils. Chapter 3 describes the permeameter (Gradient Ratio Test device) used in this laboratory study. Properties of the materials used in testing are reported in Chapter 4. In addition, specimen reconstitution techniques and the test procedure are described in Chapter 4. Chapter 5 presents results obtained using the permeameter, as is followed by an analysis and interpretation of the experimental data in Chapter 6. A series of conclusions and recommendations are presented in Chapter 7.

2.0 Literature Review

2.1 Filter specifications for cohesionless soils

Regulatory practices and associated design guidance tend to varying with country, leading to variations in the specification of filters in soil structures. Most of these specifications are based in the first works of Terzaghi (1939), who proposed that the gradation between two soils meet the following criteria:

$D_{15}/d_{85} < 4$	Soil retention criterion (piping)	[2.1]
$D_{15}/d_{15} > 4$	Permeability criterion	[2.2]

where d_{15} and d_{85} are the 15 and 85 percent size of the base material respectively, and D_{15} is the 15 percent size of the filter material (see Figure 2.1).

Other gradation requirements that have subsequently been used include:

- Gradation of the filter should be approximately parallel to the gradation curve of the protected soil, especially in the fines range (Waterways Experimental Station 1941, 1948).
- Filter should not have particles larger than 75 mm so as to minimize segregation.
- Filter should not contain more than 5 percent fines, and the fines should be non-plastic (Karpoff, 1955).

- If the base ranges from gravel to silt, the base material should be analyzed on the basis of the gradation smaller than 4.75 mm (Karpoff, 1955).

Based on these concepts, different regulatory agencies use different granular filter criteria. For ilustartive purposes, two examples of these criteria are reported below:

(1) The United States Bureau of Reclamation criteria (1987) were developed with reference to Terzaghi (1939), supplemented by controlled laboratory tests and studies performed by Bertram (1940), and others. A summary of these criteria is given in Table 2.1.

(2) The Geotechnical Engineering Office, Civil Engineering Department, Hong Kong, recommends the approach summarized in Table 2.2, based on requirements of stability, permeability, and segregation.

In these filter criteria, requirements for the stability and permeability are given greatest importance. Careful consideration of broadly graded and gap graded soils is recommended, due to the potential for internal stability in these materials. The maximum content of fines is restricted to no more than 5 % and the plasticity of the fines is also limited. There is no restriction on the width of the filter, or the number of filter layers if it is used a multi-layer system of filter. These requirements are further considered in ICOLD (1994), wherein the internal stability of a soil is also recognized as an important subject when analyzing soil filters.

2.2 Filter criteria for geotextiles

Geotextiles are typically used in place of, or in combination with, a graded granular filter. The basic function of the geotextile filter is to achieve the formation of a stable interface with the base soil when there is a flow of water from the base to the filter. A very strong preference exists for the use of nonwoven geotextiles in filtration applications. Experience suggests that care is necessary during installation to prevent undue exposure to ultraviolet light or damage to the geotextile itself.

In a similar fashion to granular filters, geosynthetics filters must be properly designed and satisfy the following criteria.

- Soil retention;
- Permeability; and,
- Tensile strength (durability and construction survivability).

A verification of geotextile filter criteria was undertaken by Fannin et al. (1994), using a Gradient Ratio device, manufactured at the University of British Columbia (UBC). This Gradient Ratio device was further modified by Hameiri (2000) to perform on lass beads tests that included vibration and reversing flow, and used by Hawley (2001) to examine the filtration performance of geotextile with problematic soils in cyclic flow. The same permeameter was used in this study on granular filters.

From these studies and the work of many others, it has been shown that soil particles larger than the geotextile pores tend to form a bridging network at the soil/geotextile interface. If the soil is stable, then behind this zone, the soil remains undisturbed. As such, the geotextile acts as a catalyst in the formation of a bridging zone, but it is the soil itself that must form a stable zone. As a result, the internal stability of the soil agan becomes an important consideration in the design of filters.

2.3 Internal stability of granular soils

Seepage flow exerts a force on the grains of a soil. When this force is greater than those acting to restrain the particles, then migration can occur within the soil. Flow mass then concentrate in these zones, and therefore local permeabilities can differ significantly from the global permeability. A soil is defined as internally stable when it has the ability to generate a stable layer in its skeleton. If this layer is located at the boundary, it is called a "self-healing" layer and the material could be termed a "self-filtering" system. In contrast, an internally unstable soil has fine grains that are able to move freely inside the "skeleton" of the soil. There are different conditions, or factors, influencing how much and when these fine particles will move. These factors include:

- Hydraulic conditions
    - Seepage velocity
    - Hydraulic gradient
    - Flow direction
- Geometrical conditions

    - Porosity
    - Constriction size
    - Shape of the distribution size curve
- Other conditions
    - Vibration
    - Air content
    - Temperature

Most empirical design criteria have been developed with a focus on the geometrical conditions. Filter studies on uniformly graded, broadly graded, and gap graded soils provide the data necessary to establish an appropriate "filter rule" and therefore, to define whether or not a soil is potentially unstable. Kenney and Lau (1985) proposed a method to predict the behaviour of soils under seepage based on the grain size distribution curve. In addition, Honjo et al. (1996) found limit values for the maximum gap ratio and slope on gap-graded soils. Prior to these observations, Kézdi (1979) proposed a theoretical approach that involves dividing the soil into components for purposes of stability assessment.

In addition to the above, one must be careful when using these methods since hydraulic conditions in the field may be significantly different from those in laboratory tests performed to support development of the methods. Therefore, it is very important to establish an appropriate threshold to the onset of instability and consider the hydraulic conditions in design.

2.4 Review of laboratory tests on filtration

Terzaghi (1939) established the requirements for a filter material in order to protect a cohesionless base soil. Based on the results of tests made with the purpose of ascertaining the required grain-size, Terzaghi found that any sand will serve as a suitable filter if its grain size curve intersects the 15 percent line between the points a and b, as previously shown in Figure 2.1. Bertram (1940) reported the results of tests to verify the soil retention criterion proposed by Terzaghi. This first systematic study established many key considerations for laboratory study of filtration phenomena. Specifically, the importance of using distilled de-aired water was identified, to prevent any air in the water supply coming out of solution in the soil specimen during the test and to avoid any decrease in permeability due to suspended solids in laboratory tap water. Details of Bertram's constant head test apparatus are shown in Figure 2.2. Tests were either with 2 h (at $i = 18$ to 20) or 4 h (at $i = 6$ to 8) in duration, with unidirectional flow imposed in a downward direction (and for selected cases in an upward direction) through the base soil located above a filter layer. Any incompatibility between the base and filter soils was found to initiate very quickly, with the visible movement of soil ceasing after three to five minutes. It was also found that the minimum critical ratio between the 15 per cent size of the filter and the 85 per cent size of the base was approximately 6. The findings implied a margin of safety against inadequate retention of the base soil when using the Terzaghi criterion of equation [2.1].

The U.S. Army Corps of Engineers (1941) studied filter requirements for underdrains using two permeameters. A small permeameter of 7.6 cm diameter was used to test a thick layer of filter material and thin layer of fine base material (see Figure 2.3). A second permeameter, 21.3 cm in diameter, was used with perforated discs to simulate drainage pipe (see Figure 2.4). In both cases the specimen length was 16.5 cm. The material passing the discs was collected and weighed in each test. The duration of the test was between 2.5 and 5 hours in the small permeameter, and from 15 to 30 minutes in the large permeameter. The base material was a very fine sand and coarse silt, which was believed to be most susceptible to soil migration. A mixture of concrete sands and gravels was used as the filter. The flow was typically downward, and the loss of water head was measured at various points along the permeameter. Regular tap water was used in these tests. The side of the permeameter was tapped with a rubber mallet. No surcharge was applied to the soil specimen.

Relatively stable conditions prevailed when the ratio D_{15}/d_{85} was equal to 5 or less in the small permeameter tests. When this value exceeded 5 the fine base material washed through the filter material. Again, it was attributed to a margin of safety in the Terzaghi filter criterion for soil retention. The introduction of large size particles into the filter material made it easier to the base fine to move through the filter. A further recommendation given was that the grain size curves for filter and base materials should be approximately parallel. In addition, the filter material should be packed densely in order to reduce changes in the gradation due to soil migration. In the large permeameter, most of the weight of material passing through the perforated discs was obtained after tapping. It was also postulated that a well-graded material is less susceptible to running through the drain pipe openings than a uniform material of the same average size.

Karpoff (1955) studied the design criteria for protective filters based on test results published by the US Bureau of Reclamation (1947) and (1955) using a well-graded silt, a uniformly graded fine sand and medium sand, and a well-graded gravelly sand as the base soils. The filter materials were uniform gradations of medium or coarse sand or fine to medium gravel. Details of the permeameter used in this research are shown in Figure 2.5. Few selected details of the test method are reported for comparison in Table 2.3. From these tests, Karpoff made the following observations that have since been widely adopted in filtration criteria:

- "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 (sieve N° 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 the 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."
- "In the designing of filters for base materials containing particles larger than 4.75 mm (sieve N° 4) size the base material should be analyzed on the basis of the gradation of material smaller than N° 4 size."

Karpoff (1955) recommended that these requirements, and additional rules for the ratio D_{50}/d_{50} and D_{15}/d_{15} for uniform and broadly graded soils, should be met in designing a protective filter.

Kézdi (1979) reported tests on the behaviour of internally unstable soils. These soils were considered to comprise two gradation components; based on this premise, it was necessary to examine only these components. One component serves as a filter for the grains of the other. If the two components satisfy the filter rule then the grains of the finer component will not wash out of the soil's skeleton. The division of a soil into two components is shown in Figure 2.6. Arbitrarily, the diameter d_0 is taken as a divisor, which then determines the values of D_{15}' and d_{85}'. Afterward, the same procedure is applied with another d_0 point. This process is repeated until there are sufficient points to confirm that the filter rule is satisfied over the entire gradation range.

Jacques Paré (1982) performed a series of tests to simulate various base/filter combinations encountered in the LG3 dam, Québec. The soils included broadly graded and gap graded particle size distributions. Some of these soils were believed to be internally unstable. He used either de-aired or tap water depending on the water flow obtained in each soil specimen. A water purifier was also used to avoid suspended solids in the water. Paré employed a relatively large permeameter, yielding a specimen 91 cm in length and 61 cm in diameter. A constant level tank allowed for control of the differential water head across the soil specimen. Tests were run under downward or horizontal flow of water, without any surcharge pressure (see Figures 2.7 and 2.8). Pore pressure was monitored either by standpipes or by electrical pressure transducers. Generally, filter tests in both vertical and horizontal directions were performed under five hydraulic gradients, with successive values of 0.5, 1.5, 2.5, 3.5, and 5. The average duration of each hydraulic gradient was 48 hours. The investigation involved several phases. The first phase involved core material in contact with coarse filter, and a fine filter in contact with a coarse transition. In the second phase, the influence of severe conditions was investigated with a pervious trench placed across the base material, parallel to the flow, in order to check the influence of a higher flow velocity on the stability of the base material. In the third phase, the adequacy of coarse filter materials was investigated by varying the ratio D_{15}/d_{85} between 5 and 20. In addition, the behaviour of soils with lower density was studied, and thinner base zones, thus obtaining larger gradients at the interface between materials. It was found that, in most cases, a precipitation of air bubbles and colloidal rust accumulation in the uppermost part of specimens was responsible for significant decreases in permeability. In the vertical test, the hydraulic gradient at the interface between the base and filter was below 0.5 (overall hydraulic gradient equal to 5). In the horizontal tests, the local gradient at the interface was as high as 14. All the tests indicated that the different soils tested were stable.

Lafleur (1984) performed tests on base soils that were well-graded gravelly silt-sands, considered represented of tills used extensively for dam construction in the James Bay project, Quebec. The test setup is shown in Figure 2.9. A constant water head was applied using calibrated springs to support water tanks, thereby maintaining a constant differential water level. The soil specimen had a diameter of 15 cm with a base soil length of 15 cm and 20 cm for the filter. An effective cell pressure of 100 kPa and a back-pressure of 800 kPa was applied to the soil specimen. The hydraulic gradient applied was up to 8, and the test duration was between 50 and 880 h. All of the base soil movement was found to take place in the first 50 h. Analysis of laboratory data indicated that the limiting criterion of Bertram was applicable when the size d_{85}' is based on the fraction smaller than sieve N° 4, thereby confirming the general observations of Karpoff (1955) (and the design criterion for soil retention first proposed by Terzaghi, 1939)

Sherard et al. (1984) carried out an investigation to gain an improved understanding of the fundamental properties and behaviour of filters. The permeameter used is shown in Figure 2.10. The base soil was a uniform fine, medium or coarse sand. The filter material was a uniform coarse sand, uniform gravel, or well-graded sandy gravel. The test specimens were 10 cm in diameter, with a length between 5 and 10 cm (base soil) and 12.5 to 17 cm (filter soil). The test was run with downward flow, and without surcharge, for a duration of 5 to 10 min. Afterwards, the soil specimen was placed on a shake-table for 60 s if little or no base soil had migrated through the filter layer. Results suggested the existing filter criterion, $D_{15}/d_{85} \leq 5$, is conservative but should be remain as the basis for judging filter acceptability. Criteria based on the ratios D_{50}/d_{50} and D_{15}/d_{15} were not believed to have a experimental or theoretical basis, and it was suggested that they should be abandoned. It was also argued that the particle size distribution curve of the filter is not required to be similar in shape to the base soil.

Kenney and Lau (1985) studied the effect of disturbing forces such as seepage and vibration on internal stability. Constant-head tests (see Figure 2.11) were performed on specimens of approximate diameter 245 mm or 580 mm. The smaller specimen had a length of 450 mm (base soil), and the larger one a length of 860 mm (base soil). Comparative details of the test are shown in Table 2.3. A mild vibration was applied to the specimen throughout the test, and was found to have a profound influence on the response of some of the soils. Results of the seepage tests were used to propose a method for evaluating the potential for grading instability based on the shape of the grain size curve (see Figure 2.12). In this method, the value of F corresponds to the "mass fraction smaller than" a particle diameter D. The value of H corresponds to the mass fraction between particle sizes D and 4D. The rationale for selecting a ratio of 4D is the size of the predominant constrictions in the void network of a filter is approximately equal to one-quarter the size of the smallest particle making up the filter. The postulated boundary between stable and unstable grading curves was initially defined as H/F = 1.3, over the portion of gradation up to F = 20% for widely graded soils and F = 30 % for narrowly graded soils. Later the boundary was redefined as a slope of H/F = 1.0 (Kenney and Lau, 1986) upon discussion of the data by Milligan (1986).

Lafleur et al. (1989), used his previous equipment and test methodology (as described by Lafleur (1984) and Lafleur et al. (1986)), to further study the response of base soils. The base soils were very well graded silty gravel-sands, with a trace to some clay, and gap-graded silty gravels. The filter materials were uniform gravels, and well-graded gravel-sands. There were two series of tests. The first series, called compatibility tests (see

Figure 2.9), involved filters of increasing coarseness. In the second series, called screen tests (see Figure 2.13), the quantity of soil particles lost during the self-filtration process was measured using glass beads as base materials. In the first series, after consolidation at a cell pressure of 800 kPa and effective cell pressure of 100 kPa, hydraulic gradients were applied in steps up to a value of 8 during an average test duration of 77 h. These constant head tests were performed on specimens that were 150 mm in diameter, with a length of 150 mm (base soil) and 200 mm (filter soil). In the second series of tests, a variety of glass beads were mixed to obtain bases with different gradation curves and a conventional square mesh sieve was used as a filter. Downward flow was maintained using a constant head tank, with hydraulic gradients ranging from 2.5 to 6.5. An air filter was used to ensure no air bubbles entered the system. Vibration was applied to impede the formation of arches at the filter interface. The test duration was 2.5 h, after which sieve analyses were performed on different layers. It was found that the self-filtration process in the base-filter interface is very important to control the stability of the base soil. It was also identified that the broadness coefficient had a big influence in the self-filtration process. On the other hand, vibration promoted the downward movement of particles and discouraged the formation of soil arches. The opening size of filters must be compared with an indicative size of the base soil to ensure minimal particle migrations.

Skempton and Brogan (1994) performed piping tests on sandy gravels. The objective was to compare the theoretical value of the critical gradient to cause piping under upward flow on stable and unstable materials. The tests apparatus comprised a permeameter 13.9 cm in diameter, with internal piezometers located at different positions. The base specimen was approximately 15.5 cm in length, see Figure 2.14, with a screen and filter material below it. The test procedure imposed an incremental increase of the upward flow of water until piping occurs. Piping yielded a big increase in flow. An abrupt transition was noted from unstable to stable response. The boundary defined by Kenney and Lau (1985, 1986) was found reasonable. Further, the limit proposed by Kézdi(1979) was also found to agree with their findings.

Honjo et al. (1996) examined aspects of self-filtration behaviour in widely and gap graded cohesionless base soils, using permeameters of diameter 15 and 32 cm. The specimen length was 10 cm, and it was supported on a metal screen filter (see Figure 2.15). A light surcharge of 0.9 kPa was applied to the specimen. Downward flow of distilled water was imposed, either with head control (at a hydraulic gradient between 2.5 to 14) or using a pump (for hydraulic gradients up to 52). There was a stage with continuous tapping by a rubber hammer while maintaining the downward flow of water. All tests were run for a period of two hours. The mass of soil passing the screen was measured to define the potential for internal stability. Several test series were performed, including some on gap-graded soils. For these soils a gap ratio of 4 (defined as the large diameter / smaller diameter in the gap range) was considered as an upper limit for stability.

Tomlinson and Vaid (2000) studied the influence of confining pressure on soil migration. Different values of the ratio D_{15}/d_{85} where evaluated at various levels of confining stress, hydraulic gradient, rate of gradient increase, and filter thickness. Downward flow of tap water was imposed. Surcharge pressure was varied between 50 to 400 kPa. The applied hydraulic gradient was in the range of 0 to 25. No vibration was applied during the test. The base soil specimen had a diameter of 10 cm and a length of 4 cm. Analysis of the data revealed no piping for D15/d85 < 8 and spontaneous piping at D15/d85 > 12. In addition, the critical gradient at which piping is triggered was found to decrease with an increase in D15/d85. At a given D15/d85, the critical gradient decreases with increase in confining pressure.

2.5 An example of seepage behaviour: Bennett Dam

The performance of WAC Bennett Dam has been of great interest since its construction in 1968. During the 1980's unexpected high pore water pressure were noticed. This increase on water pressure, during the first years of operation, was followed by a continued steady reduction (see Figures 2.16 and 2.17). In 1996, two sinkholes suddenly emerged centered on survey benchmark tubes. A hypothesis to explain the manner in which the high pore pressure dissipated and details of the dam behaviour were presented by Stewart and Garner (2000). They concluded that:

- "the numerous mechanisms observed at this dam indicate that earthfill dams are not always inert structures. Some may be continually adjusting to ever-changing conditions. Reservoir cycling, daily and seasonal temperature variations, long-term pore pressure build-up and dissipation, consolidation and stress adjustments, and the mechanics of filtering all have impacts that require continuous attention over the life of the dam";

- "the safety of the dam has been assessed with due consideration of the various mechanisms which may have caused the sinkholes, and the safety remains independent of these mechanisms"; and,
- "Further research is required to assess the effects of air exsolution, gradients and fines migration within broadly graded cores and filters".

Additional study has been made to study the performance of filters and cores. Results of laboratory tests indicate that potentially internal unstable materials and gas exsolution can combine to trigger piping (Garner and Sobkowicz, 2002). More specifically they found:

- "the process of suffusion can produce extremely low permeability, high gradient zones within internally unstable materials such as found in widely graded cores, filters and foundations of earthfill dams";
- "the process of suffosion can occur in gap-graded materials that have a coarse fraction with widely dispersed particles";
- "the process of suffosion can be triggered by the introduction of gassy water"
- "the concept of self-healing in widely graded cores and filters may not apply to gap-graded soils vulnerable to suffosion"; and ,
- "preliminary results indicate that the Kenney-Lau internal stability criteria of H/F = 1 could possibly be extended to beyond F = 20%.

3. TEST DEVICE

3.1 Introduction

The gradient ratio test device was originally developed to evaluate the filtration compatibility of a soil and geotextile under unidirectional flow. It is configured to impose a constant water head, and hence hydraulic gradient, across a reconstituted soil specimen that is 100 mm long and 100 mm in diameter (ASTM D4595). In this study, after modifications, the device was used to study the internal stability of soils under seepage.

3.2 Modified gradient ratio device

A GR device, that was designed and used extensively for assessment of soil-geotextile compatibility in both unidirectional and reversing flow (Shi, 1994; Hameiri, 2000; Hawley 2001), was modified for the purposes of this study. The main objective of these modifications was to allow for the application of constant vibration energy to the permeameter, by automatic tapping of the test specimen. This was achieved through mounting a small automatic air hammer on the device. Additionally, the base plate was reconfigured to accept a wire mesh screen rather than support a geotextile. Figure 3.1 shows the arrangement of the test assembly and Figure 3.2 provides more details of the permeameter and flow control system.

3.2.1 Soil specimen and filter screen

The rigid-wall permeameter (plexiglass pipe) has a diameter of 100 mm. Following the recommendations given by the ASTM for use of permeameters, the maximum particle size of the reconstituted soil specimen was limited to 12 mm. This limitation ensures that boundary effects on the behaviour of the soil specimen are minimal.

The soil specimen rests on a wire-mesh screen that is supported by a rigid base plate. The base plate is perforated by a systematic array of circular holes, larger that the square openings of the wire mesh screen. This base plate provides a rigid lower boundary to the system. The purpose of the wire-mesh screen is to support the overlying soil during preparation and testing, while allowing for an unimpeded movement of fine particles from the specimen under the influence of seepage and/or vibration. Accordingly, the opening size of the wire mesh screen was selected to achieve a relatively constant ratio to the particle size of the soil being tested. This relation was obtained by matching the screen opening size to the particle size corresponding to 15 % passing of the coarse fraction of the soil specimen. Kézdi (1979) proposed that it is possible to divide the distribution curve in two parts. The coarse skeleton could be considered as the filter of the remaining fine particles. Using this criteria, the 15 % by weight of the skeleton can be represented by F_{15}^{S} = .85 F_n + 0.15, where F_n is the percentage belonging to the point where the distribution curve was divided. This value is very easy to obtain in gap-graded soils where the size distribution curve is horizontal in the gap range. It is possible to observe, in the Figure 3.3, the relative position or size of the screen for different soil specimens.

A collector trough is located below the opening where seepage water flows to the outlet water tank. Particles passing through the wire mesh screen are collected clamps on a flexible tube below the collector trough. Hence it was feasible to measure the amount of soil that migrated at different stages of applied hydraulic gradient and dynamic excitation.

3.2.2 Water supply and control system

A peristaltic pump is used to supply the inlet tank with de-aired and distilled water from a reservoir tank (see Figure 3.1 and 3.4). The overflow in the inlet tank is returned to the reservoir and hence the water head is maintained constant. The position of the inlet tank can be changed to provide different water head to reach hydraulic gradients in the range of 0.1 to 17. The pump is a model 7529-20, manufactured by Masterflex.

Preliminary work to commission the apparatus established that, in order to avoid clogging in the top of the soil specimen, it is necessary to use distilled/de-aired water. The clogging phenomenon is attributed to fine suspended materials, and was confirmed by observation of the permeameter and a microscopic analysis of the top layer. The distilled/de-aired water was periodically replenished during testing, by adding fresh de-aired water to the reservoir tank after about 3 hours. Confirmation of the success was achieved by measuring the dissolved oxygen content of the water and visual observations of the permeameter and hydraulic gradients at the top of the soil specimen, over time. The measurements were never found to exceed 2.5 mg/L of dissolved oxygen. This technique ensured the water was clean and without a high content of dissolved oxygen.

3.2.3 Vibration system

The vibration system comprises an air-operated double acting hammer. The hammer was mounted on the base plate of the device. The supply pressure to the hammer was maintained constant at 400 kPa by a Fairchild regulator. Using a solenoid valve, the frequency of vibration is kept constant at 1 Hz. In this way the energy of excitation remained constant in all tests. In one test the applied energy was modified in order to observe the influence of energy on the downward rate of soil migration.

3.3 Instrumentation

3.3.1 Water head distribution

The variation in water pressure along the soil specimen was measured using two independent systems. The first system comprised six manometer tubes that are connected to different ports on the permeameter. A schematic layout of the port locations is shown in Figure 3.5. Ports 1 and 7 are used to measure the applied or system hydraulic gradient. The same port locations are also used to record the difference in water head, using a differential pressure transducer. The setup consists of three ± 7 kPa (i_{13}, i_{56}, i_{67}) and one ± 17 kPa (i_{35}) differential wet/wet pressure transducers. It was necessary to use a wider range transducer to be able to measure the differential water pressure between the ports 3 and 5 under larger hydraulic gradients. The differential pressure transducers were model C230 units manufactured by Setra. After calibration in the laboratory the accuracy was found to be ± 1 mm of water. The distance between the ports is given in Table 3.1.

3.3.2 Flow rate

The permeability of the reconstituted soil, and its variation with time during a test, was deduced from measurements of volumetric flow of the water in the hose to the outlet tank (see Figure 3.2) over a variable time interval depending on the flow rate, knowing the value of hydraulic gradient applied to the specimen. The value of hydraulic conductivity determined by this method is believed accurate to ± 3e-3 cm/sec.

3.3.3 Vertical load

Axial load was applied to the specimen through the top plate. It was measured using a compression load cell manufactured at UBC. This load cell has a capacity of 50 kg. The resolution of load yields a resolution of vertical stress of ± 0.5 kPa.

3.3.4 Vertical displacement

A Linear Variable Differential Transformer (LVDT) was used to measure displacement of the top plate in contact with the specimen. This LVDT recorded displacement to a resolution of $\pm$ 0.1 mm.

3.4 Data acquisition system

An electronic data logger records all output voltages from the transducers (namely, the differential pressure transducers, load cell, and LVDT). In addition, this system was used to control the frequency of the vibration in the double acting air hammer. The system comprises a power supply, a signal-conditioning unit that amplifies the output signal from the transducers, and a Metrabyte DAS-16 board connected to a desktop computer. The DAS-16 board is a multifunction board with a 12-bit resolution and digital input and output. Voltage in the transducers is amplified in the signal-conditioning box manufactured at UBC. Data from the 6 channels is collected at a rate of 1 Hz, and written to an output file. The software used is Labtech Notebook, by the Laboratory Technologies Corporation.

4.0 Experimental Investigation

4.1 Material properties

Test specimens were reconstituted using one of two materials: glass beads manufactured by Rotair Industries, and soil obtained from a borrow pit for the Bennett Dam. Properties of these materials are described below.

4.1.1 Glass beads

Glass beads were used in commissioning the apparatus, and in a few additional tests in order to compare the behaviour of different shaped particles. The glass beads used are of 9 different size ranges that are mixed to achieve the target gradation in each test. The specific gravity of these particles is close to 2.5. The particles are perfectly spherical and clear (see Figure 4.1). Experience showed that, because of their transparent nature, they prove effective in detecting any anomalies during the permeameter test.

4.1.2 Soil

Soil from a borrow pit for the Bennett Dam was excavated and shipped to UBC. Upon receipt, the soil was sieved, washed and divided into different sizes. Inspection shows the particles to have a sub-angular shape (see Figure 4.2 and 4.3). Different size ranges of the particles were mixed together to obtain the desired gradation curve for testing, as was done with the glass beads. The measured specific gravity of the Bennett Dam material is 2.69.

4.2 Specimen reconstitution

The objective of the reconstitution technique is to replicate a homogeneous test specimen in a loose state. Experience shows this to be especially difficult when preparing specimen of gap graded or broadly graded soils, because such potentially internally unstable soils are prone to segregation during placement in construction, and likewise are challenging to reconstitute in the laboratory.

4.2.1 Slurry preparation

A uniform or homogeneous specimen is required for any systematic study of fundamental soil properties. In other words, it is necessary to replicate test specimens using a routine method of preparation. The technique must produce soil specimens that are homogeneous, of similar density and fully saturated. The method used in this study is the modified slurry deposition technique (Kuerbis, 1989). The soil is prepared as a slurry after which a method of discrete deposition was used to reconstitute the specimen in the permeameter. In this method a mass of 1700 g of soil was boiled in a flask for 30 minutes and then placed under vacuum for 12 hours. Prior to placement in the permeameter, the container with the soil is manually shaken to effect a gentle mixing until there is a homogeneous slurry.

4.2.2 Placement technique

Following the slurry preparation, a technique of discrete deposition was used to place the soil in the rigid-wall permeameter. The soil was deposited under a thin layer of surface water, no deeper than 10 to 15 mm, to ensure the specimen is saturated and did not experience segregation. The placement technique yields a loose specimen. Homogeneity of the specimen was assessed by visual observation, and validated by the reading in the differential pressure transducer at the beginning of a test. Experience showed a small quantity of soil to pass from the bottom of the specimen, through the wire mesh screen, during reconstitution.

4.3 Test procedure

A multi-stage procedure was followed in each test. The multi-stage procedure, described in the following paragraphs, allowed for an assessment of specimen response to three test variables. Test variables were hydraulic gradient across the specimen, the application of vibration, and the size of the wire mesh screen supporting the soil (see Table 4.1).

4.3.1 Multi-stage test procedure

Flow of water is imposed from the top to the bottom of the specimen. The hydraulic gradients were controlled by the difference in elevation of the inlet and outlet constant head device (see Figure 3.1). Hydraulic gradients of approximately 0.1, 1.0, 10, and 20 are applied under downward flow, each for a period of 90 minutes (see Table 4.2). Subsequently, an automatic vibration was applied to the soil for 60 minutes, while maintaining the hydraulic gradient at approximately 20. Thereafter the hydraulic gradient was decreased to 10, in order to compare the response before and after vibration. All soil passing through the bottom screen, including that during specimen reconstitution, was collected using clamps to separate the amount of soil obtained in each stage. The water, which is recirculated during a test, is changed after every two stages, or as needed, to ensure saturation of the specimen (as observed from the dissolved oxygen content)

4.3.2 Post-test observations and measurements

Upon completion of a test, various characteristics were investigated. One of the more important ones was the measurement of mass of soil passing. The dry mass of soil collected in each stage is compared with the initial weight of the soil specimen. In addition, a sieve analysis was made at different positions (bottom, middle, and top) in the specimen. These grain size distributions were then compared with the target size distribution curve. In addition, a series of pictures were taken while the soil specimen was being systematically excavated in the permeameter. These pictures were used to compare and visualize the generalized nature of the fines particles movement.

4.4 Program of investigation

Results of 16 tests are reported (see Table 4.1), on two selected gradations of Kenney and Lau (1985) and three selected gradations of Honjo et al. (1996). The gradations curves are shown in Figure 4.5. Soil D is a mix of sand and gravel particles, $C_U = 21.5$, found to be unstable by Kenney and Lau (1985). In contrast soil K is a sand, $C_U = 4$, that was found to be internally stable. Soil G3-C is a gap graded sand, $C_U = 8.5$, that lost about 20% of fines in Honjo et al. (1996). Soil G4-C is a gap graded sand, $C_U = 14.5$, that was found to be internally unstable. Soil G1-D is a sand, $C_U = 7$, with a gap ratio of 2.8 that was internally stable. A series of preliminary tests (C#1, C#2, C#3 and SP#1) were used to commission the apparatus. The objective of these tests was mainly to determinate the influence of different factors such as specimen preparation, water quality, duration, energy and frequency of the automatic vibration. Upon completing the commissioning of the apparatus, the main program of investigation was followed as shown in Table 4.1.

The intent of tests GL#1, Be#1, Be#2, Be#3, Be#4, Be#5, and Be#6 was to replicate, for the selected gradations, results obtained for those gap-graded soil by Honjo et al (1996). Spherical glass beads and sub-angular soils were examined to determine differences in behaviour between the two materials. Dry tests (D#1 and D#2) were made to evaluate the migration of fines under vibration alone. To explore the influence of boundary conditions, different sizes of wire mesh opening were used to examine the impact or amount of soil passing through it (Be#2, Be#3). In some of the tests a longer period of seepage and vibration was applied after the normal test duration (Be#1, Be#3, Be#4).

Repeatability of these tests was also verified with two identical specimens (Be#5, Be#6). Tests Be#7, Be#8, and Be#9 were used to replicate the results obtained by Kenney and Lau on soils "D" and "K".

In each test a series of stages was followed, as is shown in Table 4.2. The hydraulic gradient was increased gradually to observe the behavior of the specimen. Then vibration was applied to the base of the soil specimen and maintained for 60 minutes. After this stage the hydraulic gradient was decreased to a value of 10, and a new seepage stage performed.

5.0 Results

Following a summary of findings from preliminary tests to commission the apparatus, measurement of water head distribution, mass of soil passing and deduced values of permeability are reported for the main test program.

5.1 Preliminary tests to commission the apparatus

In the first tests (C#1, C#2, and C#3), using de-aired water, a blinding layer was found to develop on the top surface of the soil specimen. This low permeability layer was found to remain during a test. A sample of the water was analyzed under the microscope (see Figure 5.1), and found to contain suspended particles. The top blinding layer, and resultant loss of water head, is attributed to the accumulation of these particles on the top of the specimen. It causes the hydraulic gradient across the soil-wire mesh (filter) boundary to be lower than if this layer were not present. Consequently the use of distilled de-aired water was recommended in all subsequent tests.

It is important that the reconstitution technique yield a homogeneous, saturated specimen. Therefore a standardized routine was used for specimen preparation (see section 4.2). One test gap-graded specimen, SP#1, was used to examine the success of this method. The top, middle and bottom layers were analyzed immediately after specimen preparation. Gradation curves for these layers are shown in Figure 5.2. Inspection suggests the specimen is essentially homogeneous, with a uniformity coefficient

($C_u=D_{60}/D_{10}$) equal to 7.1 in the top, 10.2 in the middle and 9.5 in the lower portion of the specimen. The grain size distribution of the lower portion is a little below the other gradation curves, which is attributed to a small (0.9 %) loss of soil during placement against the mesh filter. Generally, the reconstitution technique yielded a value of dry density in the range 1.93 to 2.07 g/cm^3 for the soil specimens, with an average of 1.97 g/cm^3 for all specimens and a standard deviation less than 3.5% (see Table 5.1). Consequently, the specimen preparation is considered as satisfactory.

5.2 Repeatability of the specimen response

Tests Be #5 and Be #6 involved identical conditions, and were performed to evaluate the repeatability of the test results. The quantity of soil collected in both test was very similar (see Figure 5.26). The sieve analysis performed at the end of the test indicates that in each layer the fines content is similar (see Figures 5.25 and 5.29). There is enough similitude between these two tests to consider a satisfactory repeatability of the test procedure.

5.3 Selected gradations of Honjo et al. (1996)

Test GL#1 is a duplication of the gradation curve of the soil G3-C used by Honjo et al. (1996). The total mass passing during specimen preparation is approximately 2% of the total mass (see Table 5.1 and Figure 5.3). In the following stages, with seepage only, there is a negligible loss of soil (see Table 5.2) for gradients up to $i = 18$. Vibration yields a loss of 6.6 % in the first 60 minutes and 4.6% in the following 60 minutes. The hydraulic conductivity of the specimen was measured at the end of each stage. Values obtained are initially in the range 1.0 to 1.5e-2 cm/s (see Figure 5.4). They increase more than two times following the first stage of vibration and remain constant thereafter. The variation in water head distribution at different stages of the test, see Figure 5.5, shows a linear distribution (for example at $i = 1.1$) which implies a homogeneous specimen. Vibration results in a change, with head loss and therefore hydraulic gradient decreasing at the top of the specimen. The principle of flow continuity then indicates that the hydraulic conductivity is relatively lower in the middle of the specimen. A sieve analysis was performed at the end of the test at three different locations. The results, see Figure 5.6, showed a relatively higher fines content at the middle and bottom of the specimen. These findings are consistent with the distribution of water head.

Test Be#1 was made on the same gradation as test GL#1, using soil rather than glass beads. A higher vibration energy was applied by increasing the air pressure in the air hammer. A similar amount of soil passing as GL#1 was obtained over a shorter period of time (20 minutes) (see Table 5.1). Soil passing after reconstitution is approximately 1 % and soil passing at the end of the test was 13%. Table 5.3 shows a summary of the amount of soil passing through the filter. Figure 5.7 shows the mass of soil collected versus the hydraulic gradient in each stage. Sieve analysis at different locations of the specimen shows that the larger content of fines was situated at the top and the lower content of fines was situated in the bottom of the specimen (see Figure 5.8). During the vibration stage there was an increase in hydraulic conductivity and therefore of seepage velocity. Head losses in the permeameter decreased the overall hydraulic gradient to a value of 4.5. Figure 5.9 shows the variation in hydraulic conductivity in each test. Water head distribution in each stage is shown in Figure 5.10.

Test Be#2 was made on the same gradation (the G3-C of Honjo et al. 1996), but with a 50% reduction in the opening size of the wire mesh to a opening size of 0.45 mm. Once again the soil passing through the wire mesh during specimen preparation (0.81%) is similar to the previous tests, see Table 5.4, and seepage alone yields a very small amount of soil loss (less than 1% for gradients to 17.5). As previously, vibration produces a significant soil loss of 28% (see Figure 5.11). Figure 5.12 shows a drop in hydraulic gradients for this soil specimen. This drop in head is generated due to the increase in seepage flow as explained before. In addition, Figure 5.13 shows the water head obtained at the end of each stage. This figure shows that the sample was very homogeneous at the beginning. Afterwards, at hydraulic gradients of 10 and 17, it is easier to notice the hydraulic gradients are higher at the top than at the bottom of the specimen. This reflects the soil loss at the base during specimen reconstitution. At the end of the test, after vibration, the hydraulic gradients at different locations are more similar. A sieve analysis, made after the test, shows that about 30% of the fines particles have been lost from all portions of the specimen and that the remaining soil is homogeneous (see Figure 5.14).

Test Be#3 was made on the same gradation as test GL#1.The specimen was reconstituted using soil rather than glass beads. The loss of soil during specimen reconstitution, the initial dry density and initial hydraulic conductivity are very similar between tests Be#3 and GL#1 (see tables 5.1, 5.2 and Figure 5.16). As before, seepage induced a negligible migration of fines particles, but the application of vibration triggered a very significant loss of soil (nearly 34 %, see Table 5.5 and Figure 5.15). After this stage the hydraulic conductivity increased by a factor of approximately 5 to $k_{17} = 8.1*10^{-2}$ cm/sec (see Figure 5.16), which again compares well to the response in test GL#1. Experience shows the increase of hydraulic conductivity, and hence seepage velocity, intensifies head losses in the valves and hoses of the water system and acts to decrease the overall hydraulic gradient across the soil specimen. The water head distribution, see Figure 5.17, again shows the soil to be homogeneous at the beginning of the test. After the test, a sieve analysis confirmed the very significant loss of fines (see Figure 5.18). The top of the specimen is made almost entirely of the coarse portion of the original gradation. The gradation of the soil passing corresponds exactly to the fine portion of the soil below the gap location.

Two tests, D#1 and D#2, were performed without any water seepage. The gradation was again the G3-C of Honjo et al. (1996). This was done to evaluate the influence of vibration alone on migration of soil. Specimen preparation was by discrete deposition. In test D#1 the specimen was placed moist (w = 5%), while test D#2 was prepared dry. In test D#1 the soil passing through during specimen reconstitution was equal to 1.3 %, and after 120 min of vibration, the mass passing was 4.9%. In contrast, test D#2 yielded 2.2% during preparation and 19.8 % after 60 minutes of vibration. This demonstrates a considerable difference in behaviour between a moist and a dry soil. In Figure 5.19 the amount of soil passing is compared. The response in test D#2 compares favorably with test GL#1. It suggests that vibration plays an important role in these tests, and implies the influence of hydraulic gradient or seepage velocity is secondary to that of vibration. The relatively small loss of soil in test D#1 is attributed to capillary suction in the moist specimen.

Test Be #4 was carried out using the Honjo et al. (1996) gradation G1-D, which has a gap ratio equal to 2.9 (see Figure 5.23 and Figure 5.6 to notice the difference in gap ratio). The opening of the wire mesh filter was close to the D_{85} of the gradation, which is a relatively larger ratio than those in other tests. Soil loss during preparation was 0.61 %. After 60 min of vibration the loss was 12.5 %. Interestingly, another 60 minutes of vibration yielded an increase to only 12.54 %, which implies no additional soil migration (see Figure 5.20 and table 5.6). The hydraulic conductivity was constant until a minor increase when the hydraulic gradient was approximately equal to 9 (see Figure 5.21). After vibration was applied there was a considerable increase in the seepage velocity and therefore permeability (confirmed only by visual observation). The hydraulic gradient drops to 10.4 due to an increase of head losses in the water system. The water head distribution, see Figure 5.22, shows the fines migration to commence at the bottom of the specimen. The hydraulic gradient is smaller at the bottom, due to small amount of fines in that zone. After vibration, fines move from the top of the specimen that is more homogeneous at this time. At the end of the test, and after sieve analysis, a higher content of fines was found at the top of the specimen. However, there was no significant difference between the fines content at different locations (see Figure 5.23).

Test Be#5 was made on the G4-C gradation of Honjo et al. (1996), which has a gap ratio of 5.6. The behaviour under this gradation differs from those reported before. A 3% loss of soil occurred during specimen reconstitution (see Table 5.7). Seepage flow at low gradients of 0.1 and 1.0 yielded no soil migration, however a loss of 13% occurred at a hydraulic gradient of 9.1. Raising the hydraulic gradient to 14.9 caused an additional 5.2% loss. After this stage the seepage velocity was very high, causing head losses in the system to increase in such a way that the actual hydraulic gradient across the specimen was considerably lower than the imposed hydraulic gradient from the constant head devices. The variation of water head (see Figure 5.24) shows a lower hydraulic gradient near the base of the soil, for i = 0.1 and i = 1.2, which is consistent with a loss of soil during specimen placement. Vibration was not applied to this specimen, given the magnitude of soil loss due only to seepage. A sieve analysis, see Figure 5.25, shows the specimen to be very homogeneous and to have lost about 20% of its fines content.

Test Be#6 had the same soil gradation as test Be#5. One of the objectives of this test was to verify repeatability of the results. Specimen preparation generated a soil loss of 4.3% (see Table 5.8). As with test Be#5, little further loss took place until i = 10, at which point a 19% movement of soil occurred. Comparison indicates good agreement between these tests, see Figure 5.26. A decrease in the actual hydraulic gradient is also obtained at this stage as is shown in Figure 5.27. At the end the actual hydraulic gradient in the specimen is lower than 1.0. Seepage force was the only trigger of soil migration. Figure 5.28, shows the water head at different locations in the soil specimen. There is a big change in the slope of these curves at the top of the specimen, which implies that soil migration starts at the lower location of the specimen. At the end of the stage with an hydraulic gradient of 10 the specimen was homogeneous but with a low fines content. Figure 5.29, and through comparison Figure 5.25, confirm that more than 50 % of the fine portion of the soil is lost, and imply that there is a good repeatability in the test data.

5.4 Selected gradations of Kenney and Lau (1985)

Test Be#7 was carried out using the Kenney and Lau (1985) gradation K. This soil has larger particles and therefore a higher permeability than the soils previously reported. The initial permeability of the soil specimen was close to $9*10^{-1}$ cm/sec. At the end of the test there was only a small increase in permeability (see Figure 5.30). A 1.3 % loss of soi during specimen reconstitution is followed by a negligible additional loss of soil at hydraulic gradients up to 10 (see Table 5.9 and Figure 5.31) Although insensitive to hydraulic gradient, vibration yields a small additional loss of 2.4%. Gradation curves from sieve analysis confirm the soil passing to be smaller than the wire mesh screen (Figure 5.32). In Figure 5.33, the linear water head distributions suggest a homogeneous specimen. This soil gradation proves to be internally stable, given that only a small movement of soil particles took place.

Test Be #8 was performed on the Kenney and Lau (1985) soil gradation "D". This soil possesses a high hydraulic conductivity, equal to 1.2 cm/sec at the start of the test. Considerable head losses occurred in the permeameter system, causing the imposed hydraulic gradient of 17, to develop on hydraulic gradient close to 0.1. Seepage alone, at i

= 0.1, caused no loss of soil from the specimen. Vibration generates a 4.8 % loss after 1 hr and a further 1 % after an additional 30 minutes. A modest increase in hydraulic conductivity is associated with the vibration (see Figure 5.34). A summary of soil collected each stage of testing is given in Figure 5.35 and Table 5.10. Sieve analysis confirms that all of the fines are smaller than the wire mesh openings (see Figure 5.36). A significantly lower content of fines is apparent at the top of the sample, while the middle and bottom layers seem to be very close to the original gradation. The gradation is considered as unstable, given the mass of soil passing under vibration.

6.0 Analysis and Discussion

Test data obtained from the laboratory experiments were analyzed to better understand the generalized nature of soil migration under seepage. Thereafter, the data from the current study were compared directly with the response obtained, in previous studies, on the same gradations (Kenney and Lau, 1985 and Honjo et al., 1996). Analysis of the combined data set then addresses the validity of empirical rules (Kezdi, 1979 and Kenney and Lau, 1985) used to assess the potential for internally instability in a soil.

6.1 Generalized nature of fines movement

6.1.1 Spatial distribution of fines movement

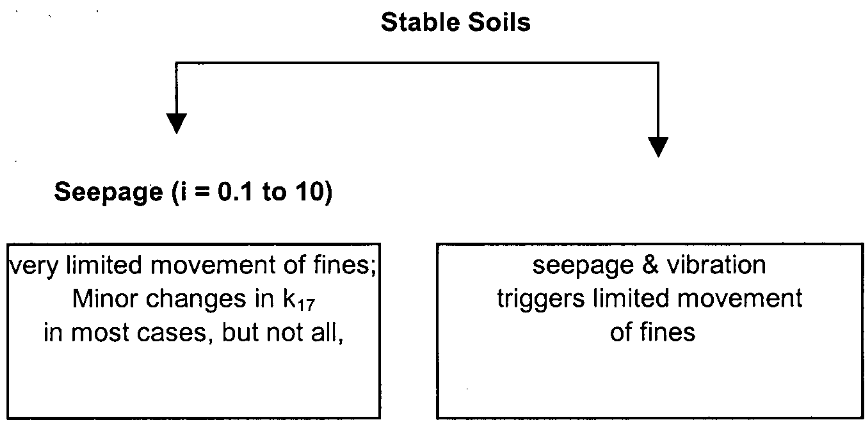
Measurements of differential water head, and visual observations in each stage of a test, together with sieve analyses at the end of a test, are used to establish the mechanism of any particle migration within the test specimen. Variations in water head distribution are used to characterize the development of the fines movement during a test. For example, Figures 5.5 and 5.10 reveal migration starts at the base of the specimen and progresses to the top of the specimen. This implies it may be necessary to "remove" the fine particles at lower or downstream locations in the specimen in order to "release" the fine particles held behind them. In general, the same behaviour was observed in other tests. Figures 6.1 and 6.2 show the change in hydraulic conductivity experienced by a soil specimen. The hydraulic conductivity between ports 5 and 6 and ports 6 and 7 is found to be slightly greater at the beginning of a test ($i_{17} \approx 0.1$). This is attributed to the small loss of fines, as previously noted, through the wire mesh screen during specimen reconstitution. Following an increase in overall hydraulic gradient, little change is observed in the upper to middle part of the specimen (k_{13} and k_{35}). In contrast the lower part of the specimen (k_{56} and k_{57}) exhibits a significant increase in hydraulic conductivity. These observations suggest that fine particles start to migrate from the top of the specimen in the latter stages of a test (high hydraulic gradient and/or vibration) and after a considerable amount of soil has already migrated from the base of the specimen.

Sieve analysis results for test Be#1, see Figure 5.8, reveal a high content of fines (33 %) at the top of the specimen after testing. The fines content is significant lower (20 %) at the bottom. This again confirms that fines migration started at the base of the specimen, and implies the test duration was not long enough or the trigger sufficient to move the remaining fine particles at the top of the specimen. Figure 5.14 shows sieve analysis results for test Be#2; in this case the specimen is very homogeneous at the end of test due to the large amount of soil loss, approximately twice as much as in test Be#1 (see Table 5.1), which left the specimen free of fines.

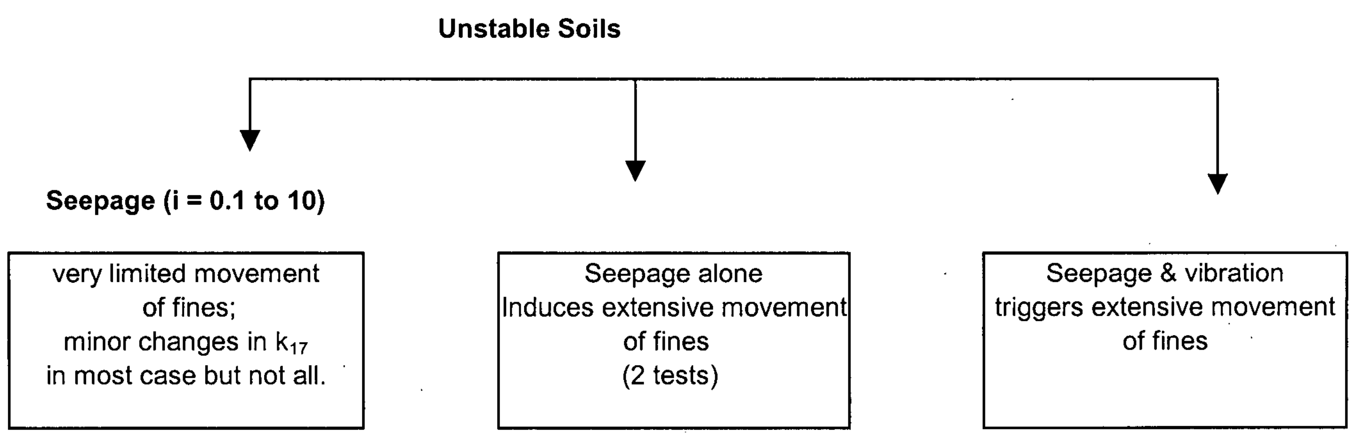
Visual observations during test Be#1 confirm the homogeneity of the specimen at the beginning of the test (Figure 6.3). After seepage flow at a maximum hydraulic gradient of 16.9, there was no significant fines migration (Figure 6.4). After vibration, fines movement was apparent at specific locations (see Figures 6.5 and 6.6). Examination of Figure 6.7, which shows areas of relatively low fines content within the specimen, suggests a pattern of movement that is not restricted to the contact between soil and rigid wall of the permeameter. A schematic representation of this generalized movement of fines, see Figure 6.8, depicts the expected mode of soil migration.

6.1.2 Temporal variation of fines movement

Seepage, and seepage together with vibration did not induced significant fines migration in stable soils:



In general, fines migration did not occur under seepage alone. Rather it was observed to initiate immediately upon applying vibration to the specimen in the unstable soils:



The movement was found to stop approximately 1 minute after vibration ceased (with unidirectional flow still imposed). However, in the case of tests Be# 5 and Be#6, where very extensive soil migration took place under the influence of seepage alone, fines movement began as soon as the hydraulic gradient reached a threshold value, and was continuous until most of the fines had been removed.

6.2 Factors influencing the generalized response

6.2.1 Influence of the filter screen size

Tests Be #2 and Be #3 were identical with the exception of the opening size of the wire mesh screen. An opening size of 0.86 mm was used in test Be #3, while that for test Be #2 was 0.45 mm (see Figure 6.9). The mass of soil passing through is slightly different in the two tests, being 34.8 and 28.0 % respectively (see Table 6.1). The amount of soil passing through is less with the smaller mesh screen. It appears that, although the ratio between the coarse particles and the opening size of the wire mesh is similar to that shown in Figure 6.10, the smaller wire mesh (with an opening size larger than the maximum particle size of the fine portion of the soil) does not inhibit the migration of soil (see Figure 3.4).

A relatively larger opening size of 1.15 mm was used in test Be#4. The opening size is equivalent to the D_{85} of the gradation curve. Although the mass of soil collected was high, representing a 12% loss (see Table 6.1), the soil behaved as a stable material. A grain size analysis of soil passing though the wire mesh screen in two other tests, see Figures 6.11 and 6.12, shows the maximum particle size to be strongly influenced by the opening size of the mesh. It appears the filter screen size exerts a clear influence on mass of soil passing through. Accordingly, care should be taken regarding the sizes of particles that are potentially able to move and the opening size of the screen.

6.2.2 Relative influence of vibration

Tests Be #1 and Be #3 were identical, with the exception that the energy of vibration was greater in test Be #1 (with the duration reduced to 20 minutes). Inspection of Table 6.1 shows the rate of soil migration to be similar, given a 13% loss in 20 mins and 34.8% loss in 60 min respectively. Hence vibration does not seem to influence the rate of soil migration, assuming that the energy is sufficient liberate the fine mobile particles. Further study must be undertaken to confirm that seepage velocity controls the rate of fines movement.

Using the same vibration energy, test D#2 demonstrated that a completely dry soil under vibration (and without seepage flow) experiences a similar quantity of soil passing as a completely saturated specimen under seepage and vibration (test Be#1), see Figure 5.19.

Hence it appears that, in comparison to mesh opening size, the vibration energy imposed in this study was a secondary or minor influence.

6.3 A comparison with original studies

6.3.1 The study of Kenney and Lau (1985)

The primary basis for a comparison with the findings of Kenney and Lau (1985) is the loss of soil reported in a test. Table 6.1 shows the total mass of fines migration reported in that study to be none for soil K and 19 % for soil D. The current study yielded values of 3.7 and 9.3 % respectively. The absolute values are different, yielding a good agreement for soil K, but not for soil D. Nonetheless, both studies imply soil gradation "K" is stable and soil "D" is unstable.

6.3.2 The study of Honjo et al. (1996)

Again, the preliminary basis for comparison between studies is the loss of soil reported in a test. Figure 6.13 and Table 6.1 summarize the relevant data. Honjo et al. (1996) report a 2.5 % loss for soil G1-D, compared to 12.5 % in this study. For soil G3-C, Honjo et al. (1996) report a 20.0 % loss, while the range in this study is found to be 13 to 34.8%. For soil G4-C the values are 33.0% (Honjo et al.) and 21.4% and 23.1% (from a repeated test) respectively. The differences are attributed to variations in test procedure and test variables (see Table 2.3). For example, it is postulated that tests Be #5 and Be #6 exhibited a smaller amount of soil migrating because no vibration was applied (while Honjo et al. applied manual tapping) and the duration of the test was shorter.

Notwithstanding the variation in test procedure and variables, an upper bound value for the gap ratio that will avoid instability of the soil is confirmed to be in the range 2.8 to 4.0. Note, this criterion is only recommended when the gradation curve of the soil has a horizontal or almost horizontal gap.

6.4 On the validity of criteria for internal instability

6.4.1 Kenney and Lau criterion

Gradations evaluated using the Kenney and Lau (1985, 1986) criteria to define internally stable soil are reported in Figure 6.14. Soil G4-C is characterized as unstable, since lies below the H=F curve at values of F< 20%. Soil G3-C lies on the boundary between stable and unstable, and soil G1-D is characterized as stable. Results from the UBC study indicate soils G4-C and G3-C are unstable, and soil G1-D is stable. Based on these comparisons, it appears the method for evaluating the potential for grading instability is reasonably applicable.

The instability of soil G3-C observed in both the current study and that of Honjo et al. (1996) is not clearly predicted by the method of Kenney and Lau (1985). This raises the issue of whether the method could benefit from any revisions or modifications. It relies on two boundaries, namely the H=F boundary and for widely-graded soils, the F= 20% boundary. The apparent stability of soil G1-D and instability of soil G3-C, when contrasted with the location of these boundary lines (see Figure 6.14), suggests careful consideration and further study required to establish any revision to this method.

6.4.2 Kézdi criterion

Kézdi (1979) proposed a method to split the gradation curve into a coarse and fine fraction. A value of D_{15}' and d_{85}' is then determined to establish if the two individual curves meet Terzaghi's criterion for filters (equation 2.1). If they do not met this criterion, the original gradation is considered to be potentially unstable. The procedure is repeated at different arbitrary division points to determine the maximum value of D_{15}'/d_{85}', where D_{15}' is the 15 % passing of the coarse fraction and d_{85}' is the 85 % passing of the fine fraction. A split of gradation G3-C, using the gap location of 40 % as the arbitrary division point, is given in Figure 6.15. The value of D_{15}'/d_{85}' is equal to 5.3, which does not satisfy Terzaghi's criterion of $D_{15}/d_{85} < 4$. As a result, this gradation would be classified as unstable using this method. The value of D_{15}'/d_{85}' is 3.8 for gradation G1-D, implying it is stable. Gradation G4-C yields a value of D_{15}'/d_{85}' equal to 7.5 and is therefore considered unstable. Figure 6.16 provides a summary of D_{15}'/d_{85}' values for each of the gradations tested, the data are plotted against total loss of soil (from Table 6.1). It provides a synthesis of data from Honjo et al. (1996), Kenney et al. (1985), and the current study. Inspection shows D_{15}'/d_{85}' greater than 4 to be a suitable threshold to fines migration. This infers the method of Kézdi (1979) can be used with reasonable confidence to predict the internal instability of soils.

6.5 On evaluating the likely severity of a potential instability

Following directly from the approach of Kezdi (1979), a methodology is proposed to evaluate the maximum possible quantity of fines migration, and hence the severity of a potential instability. The methodology is as follows.

- the original particle size distribution curve is split in an arbitrary point;
- a curve is drawn parallel to that of the coarse fraction, where the distance between the curves is given by D/4.0 (D being any diameter in the coarse fraction curve);
- a vertical line is drawn at the minimum particle size (D_0) on the new curve;
- the zone between the vertical line and the parallel curve is defined as stable (see Figure 6.17)
- fine particles located outside this stable zone are deemed susceptible to migration;
- the procedure is again repeated, using another arbitrary point to split the original curve; and,
- after a number of iterations, the greatest quantity of potentially unstable soil defines a limit to the maximum possible loss.

An application of the methodology to soil G3-C is shown in Figure 6.17. The arbitrary point used to split the gap-graded curve was the D_{40}. Using this method, 87% of the fines fraction is defined as potentially unstable, which corresponds to 35 % of the total mass. A number of iterations established this arbitrary point as the worst case. Figure 6.18 and Table 6.1 compares the maximum quantity of fines migration calculated by this method and the quantity observed in this study. The predicted values are in good agreement with the results of the extreme observations in testing. Some data points do not are in perfect agreement, which is to be expected. For gradation G4-C (tests Be#5 and Be#6) the soil that migrated was lower than the amount predicted. However, vibration was not applied during these tests and duration was shorter (as noticed earlier). It is anticipated that the amount of soil collected would be higher if vibration were applied tests Be#5 and Be#6. Due to the larger size of the wire mesh used in test Be #4, particles corresponding to the coarse fraction were found to pass through the wire mesh screen, causing an increase in the measured quantity of soil migration.

7.0 Conclusions and Recommendations

Assessment of the likely impact of seepage flow through a potentially internal stability soil requires (i) a threshold to the onset of stability be defined and (ii) the extent of instability be quantified.

Empirical criteria have been proposed (Kezdi, 1979; Kenney and Lau, 1985, 1986) to define the potential for a soil gradation to be internally unstable. Those criteria are based on interpretations of laboratory permeameter studies that mainly consider geometrical constraints, and do not explicitly account for other parameter such as hydrodynamic constraints. New laboratory data, from permeameter testing with a Gradient Ratio test device, provide further confidence to the use of those two empirical approaches. The Kezdi (1979) method, based on analysis of split gradations, proved very successful. The Kenney and Lau (1985, 1986) method, based on shape of the gradation curve, appears reasonable. It may, however be unduly constrained by a limit on the value of percentage finer (F, %) over which the H:F boundary of 1:1 is evaluated.

A synthesis of the data from this and selected previous studies indicates the extent of the instability is wide ranging in terms of total loss of soil. While there is a basis for reasonable confidence in identifying soils that are potentially unstable, the role of hydrodynamic influences and the implications of any instability (in terms of total soil loss) cannot be described with confidence.

Design and operation of the permeameter are critical. Conditions such as the flow regime, energy imposed by tapping the specimen and geometry of the mesh filter screen are important when evaluating the mass of soil migrating. This presents a challenge when attempting to characterize the response at the field scale from laboratory element tests.

In element testing, the specimen reconstitution technique must ensure a homogeneous and repeatable specimen. The slurry mixing technique, with discrete placement, used in this study proved to be an effective method for broadly and gap graded soils.

A multi-stage method was used to examine, separately, the influence of seepage and automatic tapping of the specimen. Different values of hydraulic gradient were imposed, in search of a threshold value of hydraulic gradient. However, in most of the tests it proved necessary to use vibration (automatic tapping) to trigger soil migration. Comparisons between laboratory studies would benefit from standardizing the method, and therefore energy, imparted to the specimen.

In further studies of the internal stability of a soil, it is recommended the permeameter have the following characteristics:

- measurement of axial load at the top and bottom of the soil specimen, to characterize the influence of sidewall friction;
- measurement of pore water on opposite sides, to characterize spatial variations in soil migration;.

- control of the water supply system to maintain the differential water head across the specimen at a constant value during a test, and thereby prevent drops in hydraulic gradients occurring because of an increase in seepage flow (because soil migration); and,
- monitoring of the soil specimen to assure saturation or to evaluate the role of exsolved air on the soil behaviour.

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Table 2.1 Design criteria for granular filters (after US Bureau of Reclamation, 1987)

Rule Number	Filter Design Rule	Requirement
1	$D_{15F} < 4 \times d_{85B}$ (1) $D_{15F} < 0.7\ mm+(40-A)(4xd_{85B}-0.7mm)/25$ (2)	**Stability** (1) sand and gravels:less than 15% fines (2) silty and clayey sands and gravels: 15 to 39 finer
2	For gap-graded and unstable, broadly graded base soils, the filter should be designed to protect the fine matrix of the base soil	
3	The permeabilty of the filter should be at least 25 times that of the base material. Generally: $D_{15F} > 5xd_{15B}$	**Permeability** D_{15F} not less than 0.1 mm
4	The percent fines finer than No 200 sieve must not exceed 5% by weight after compaction	
5	The ratio D_{90F}/D_{10F} should decrease rapidly with increasing D_{10F}	**Segregation** Generally, a filter should be uniformly graded to prevent segregation during processing, hauling, and placing.
6	Filter should have relatively uniform grain-size distribution curves, without "gap grading"	

Notes: D_{10F} and D_{90F} limits for preventing segregation

Minimum D_{10F} (mm)	Maximum D_{90F} (mm)
<0.5	20
0.5 - 1.0	25
1.0 - 2.0	30
2.0 - 5.0	40
5.0 -10	50
10 - 50	60

Table 2.2 Design criteria for granular filters (after Geotechnical Hong Kong)

Rule Number	Filter Design Rule	Requirement
1	$D_{15}F_c < 5 \times D_{85}S_f$	**Stability** (I.e the pores in the filter must be small enough to prevent infiltration of the material being drained)
2	Should not be gap-graded (I.e. having two or more distinct sections of the grading curve separated by sub-horizontal portions)	
3	$D_{15}F_f > 5 \times D_{15}S_c$	**Permeability** (I.e. the filter mustbe much more permeable than the material being drained)
4	Not more than 5% to pass 63μm sieve and that fraction to be cohesionless	
5	Uniformity Coefficient $4 < D_{60}F/D_{10}F < 20$	**Segregation** (I.e. the filter must not become segregated or contaminated prior to, during, and after installation)
6	Maximum size of particles should not be greater than 50 mm	

Notes: (1) D_{15F} is the size (in mm) that allows 15% by weight of the filter material to pass through. Similarly, D_{85S} is the size of sieve (in mm) that allows 85% by weight of the base soil to pass through. The subscript c denotes the coarse side of the envelope, and subscript f denotes the fine side.

(2) For a widely graded base soil, with original $D_{90}S > 2$ mm and $D_{10}S < 0.06$ mm, the above criteria should be applied to the "revised" base soil grading curve consisting of the partilces smaller than 5 mm only.

(3) The thickness of a filter should not be less than 300 mm for a hand-placed layer, or 450 mm for a machine-placed layer.

(4) Rule 5 should be used to check individual filter grading curves rather than to desing the limits of the grading envelope.

(5) The determination of the particle size distributions of the base soil and the filter should be carried out without using dispersants.

(6) D_mF_c, D_mF_f : the size of sieve (in mm) that allows m% by weight of the filter material to pass through with subscripts c and f denoting the coarse and fine side of the grading envelope respectively

(7) D_mS_c, D_mS_f : the size of sieve (in mm) that allows m% by weight of the base soil material to pass through with subscripts c and f denoting the coarse and fine side of the grading envelope respectively

Table 2.3 A summary of selected filtration studies

Year	Author	Specimen Size	Surcharge	Water Quality	Hydraulic gradient	Flow direction	Vibration	Duration
1940	Bertram	base h = 6 cm d = 5 cm unif. d = 10 cm broad.	no surcharge	distilled and de-aired	6-8; 18-20; 10-12 (i =50 preliminary test)	downward or upward	no	2-4 hrs; 16 1/2 hrs
1941	U.S. Army Corps of Eng.	h = 16.5 cm d = 7.6, 21.3 cm	no surcharge	tap water	1 to 2	downward some upward	tapping	15 - 30 min 2.5 - 5 hrs
1955	Karpoff, K.P.	h = 20 cm d = 20 cm	no surcharge	not mentioned	1 to 24	downward	no	not mentioned
1979-1984	Sherard, A. et al.	h = 6.5 in. base d = 4 in.	no surcharge	fine soil: distilled coarse soil: tap water	4 kg/cm^2 pressure through the slots	downward or horizontal	with and without vib.	few minutes
1982	Paré, J.	h = 95 cm d = 61 cm		de-aired and tap water depending on flow	0.5 to5	downward or horizontal	no	aver: 48 hrs max. 250 hrs
1984	Lafleur, J	h = 15 cm d = 15 cm	back pressure	not mentioned	8	downward	no	50 - 880 hrs
1985	Kenney and Lau	h =20-50 cm d =24.5-58 cm	10 kPa	recirculated water	R' > 10	downward	mild vib.	30 - 100 hrs
1994	Skempton and Brogan	h =15.5 cm d = 13.9 cm	no surcharge	not mentioned	0 - 1	upward	no	1.5 hrs
1996	Honjo et al.	h =10 cm d = 15 and 30cm	0.9 kPa	tap water, no de-aired	36-52 pump pres. 2.5 -19 constant head	downward	tapping	2 hrs
2000	Tomlinson & Vaid	h =4 cm d = 10 cm	50 - 400 kPa	tap water	25	downward	no	3 1/2 hrs
Current	UBC	h = 10 cm d = 10 cm	25 kPa	distilled and de-aired	0.1-15	downward	automatic vibration	11 hrs

Table 3.1 Gauge length between permeameter ports in mm.

	Port 1	Port 3	Port 5	Port 6	Port 7	Screen
Port 1	0	***3.5***	8.5	10.2	2	11
Port 3	***3.5***	0	***5***	6.7	9.5	7.5
Port 5	8.5	***5***	0	***1.7***	4.5	2.5
Port 6	10.2	6.7	***1.7***	0	2.8	0.8
Port 7	13	9.5	4.5	2.8	0	2
Screen	11	7.5	2.5	0.8	2	0

Note: Port 2 in Figure 3.5 was not used in this study.

Table 4.1. Program of investigation

MATERIAL	GLASS BEADS	GLASS BEADS	GLASS BEADS	"DUMMY" SOIL	GLASS BEADS	BENNETT SOIL	BENNETT SOIL	BENNETT SOIL
Test #	C #1	C #2	C #3	SP #1	GL #1 (G3-C)	Be #1 G3-C	Be #2 G3-C	Be #3 G3-C
Gradient	min = 0.1 max = 14	min = 0.1 max = 16.2	min = 0.1 max = 15		min = 0.2 max = 17.9	min = 0.1 max = 16.9	min = 0.2 max = 17.7	min = 0.2 max = 18.5
Type of water	De-aired water	De-aired water	De-aired water & bleach	Distilled de-aired water	Distilled de-aired water	Distilled de-aired water	Distilled de aired water	Distilled de aired water
Vibration	manual vibration	manual vibration	automatic vibration		automatic vibration	automatic vibration	automatic vibration	automatic vibration
	10 min. Max.	2 hours	5 hours		2 hours	20 min.	60 min	60 min
Top Load	25 kPa	25 kPa	25 kPa		25 kPa	25 kPa	25 kPa	25 kPa
Sample Prep.	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation
	pluviation deposition	pluviation deposition	discrete deposition (5cm)	discrete deposition	discrete deposition (2cm)	discrete deposition	discrete deposition	discrete deposition
Density	vibrated before test	vibrated before test	vibrated before test					
		$\rho = 1.91$ g/cm^3	$\rho = 1.99$ g/cm^3	$\rho = 1.86$ g/cm^3	$\rho = 1.95$ g/cm^3	$\rho = 2.0$ g/cm^3	$\rho = 1.93$ g/cm^3	$\rho = 1.93$ g/cm^3
Wire mesh	0.86 mm	0.86 mm	0.86 mm	0.86 mm	0.86 mm	0.86 mm	0.45 mm	0.86

Table 4.1 (cont'd)

MATERIAL	GLASS BEADS	GLASS BEADS	BENNET SOIL	BENNET SOIL	BENNET SOIL	BENNET SOIL	BENNET SOIL	BENNET SOIL *
Test #	D #1	D #2	Be #4 G1-D	Be #5 G4-C	Be #6 G4-C	Be #7 K	Be #8 D	Be #9 K
Gradient			min = 0.1	min = 0.2	min = 0.2	min = 0.1		
			max = 14.9	max = 10	max = 10	max = 10		
Type of water			Distilled de aired water	Distilled de aired water	Distilled de aired water	Distilled de aired water	Distilled de aired water	Distilled de aired water
Vibration	automatic vibration	automatic vibration	automatic vibration	No vibration	No vibration	1 hour vibration	1 hour vibration	
	2 hours	60 min.	2 hours				+ 1/2 hour	
Top Load	25 kPa	25 kPa	25 kPa	25 kPa	25 kPa	25 kPa	25 kPa	25 kPa
Sample Prep.	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation
	discrete deposition	discrete deposition	discrete deposition	discrete deposition	discrete deposition	discrete deposition	discrete deposition	discrete deposition
Density								
	$\rho = 1.95$ g/cm^3	$\rho = 2.10$ g/cm^3	$\rho = 1.95$ g/cm^3	$\rho = 2.03$ g/cm^3	$\rho = 2.02$ g/cm^3	$\rho = 1.89$ g/cm^3	$\rho = 2.07$ g/cm^3	
Wire mesh	0.86 mm	0.86 mm	1.15 mm	1.15 mm	1.15 mm	2.76	6.5 (D_{15} coarse= 3.74-4.6)	

Table 4.2. Stages during tests

	STAGE 1 i = 0.1	STAGE 2 i = 1	STAGE 3 i = 10	STAGE 4 i = 20	STAGE 5 i = 20	STAGE 6 i = 10
					1 hr Vibration	
Normal	25	25	25	25	25	25
Stress						
Duration	1.5 Hours	1.5 Hours	1.5 Hours	1.5 Hours	1.5 Hours	1.5 Hours
Limiting	or	or	or	or		or
Condition	no change in head	no change in head	no change in head	no change in head		no change in head
	and no visible soil	and no visible soil	and no visible soil	and no visible soil		and no visible soil
	passing through	passing through	passing through	passing through		passing through
Data	Beginning 10 Hz	Beginning 10 Hz	Beginning 10 Hz	Beginning 10 Hz	1 HZ	Beginning 10 Hz
Collection	End 1 Hz	End 1 Hz	End 1 Hz	End 1 Hz	End 10 Hz	End 1 Hz

Table 5.1 Summary of tests performed.

Material	Glass beads	Glass beads	Glass beads	Glass beads	Glass beads	Bennett Soil	Bennett Soil	Bennett Soil
Test #	C #1	C #2	C #3	SP #1	GL#1(G3-C)	Be#1 (G3-C)	Be#2 (G3-C)	Be#3 (G3-C)
Gradient	min = 0.1 max = 14	min = 0.1 max = 16.2	min = 0.1 max = 15		min = 0.2 max = 17.9	min = 0.1 max = 16.9	min = 0.2 max = 17.7	min = 0.2 max = 18.5
Type of water	De-aired water	De-aired water	De-aired water bleach	Distilled de-aired water	Distilled de-aired water	Distilled de-aired water	Distilled de aired water	Distilled de aired water
Vibration	manual vibration (10 min. Max.)	manual vibration (120 min.)	automatic vibration (300 min.)		automatic vibration (120 min.)	automatic vibration (20 min.)	automatic vibration (60 min)	automatic vibration (60 min)
Vertical Stress	25 kPa	25 kPa	25 kPa		25 kPa	25 kPa	25 kPa	25 kPa
Specimen Reconst.	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation	slurry preparation
	pluviation deposition	pluviation deposition	discrete deposition (5cm)	discrete deposition	discrete deposition (2cm)	discrete deposition	discrete deposition	discrete deposition
	vibrated before test	vibrated before test	vibrated before test					
Dry density		$\rho = 1.91$ g/cm^3	$\rho = 1.99$ g/cm^3	$\rho = 1.86$ g/cm^3	$\rho = 1.95$ g/cm^3	$\rho = 2.0$ g/cm^3	$\rho = 1.93$ g/cm^3	$\rho = 1.93$ g/cm^3
Mass loss during S.P.	1%	1.20%	1%	0.90%	2%	1%	0.81%	1.13%
Total mass loss	4%	6%	8%		13.3%	13.1%	28%	34.8
Opening Wire mesh	0.86 mm	0.86 mm	0.86 mm	0.86 mm	0.86 mm	0.86 mm	0.45 mm	0.86
Permeability	Small increase during vibration	Permeability $1*10^{-2}$ almost constant	Permeability decrease with time. Decrease with vib.					
Lower content of fines			Bottom	Bottom		Bottom	Bottom	Top
Higher content of fines			Middle and top	Top	Middle	Top	Middle	Bottom (almost the same)

Table 5.1 Summary test performed (Cont.)

Glass beads	Glass Beads	Bennett Soil	Bennett Soil	Bennett Soil	Bennett Soil	Bennett Soil	Bennett Soil
D#1	D #2	Be#4 (G1-D)	Be#5 (G4-C)	Be#6 (G4-C)	Be#7 (K)	Be#8 (D)	Be#9 (K)
		min = 0.1 max = 14.9	min = 0.2 max = 10	min = 0.2 max = 10	min = 0.1 max = 10		
		Distilled de aired water	Distilled de aired water	Distilled de aired water	Distilled de aired water	Distilled de aired water	Distilled de aired water
automatic vibration (120 min.)	automatic vibration (60 min.)	automatic vibration (120 min.)	No vibration	No vibration	automatic vibration (60 min.)	automatic vibration (90 min.)	
25 kPa	25 kPa	25 kPa	25 kPa	25 kPa	25 kPa	25 kPa	25 kPa
slurry preparation discrete deposition	slurry preparation discrete deposition	slurry preparation discrete deposition	slurry preparation discrete deposition	slurry preparation discrete deposition	slurry preparation discrete deposition	slurry preparation discrete deposition	slurry preparation discrete deposition
$\gamma = 1.95$ g/cm^3	$\gamma = 2.10$ g/cm^3	$\gamma = 1.95$ g/cm^3	$\gamma = 2.03$ g/cm^3	$\gamma = 2.02$ g/cm^3	$\gamma = 1.89$ g/cm^3	$\gamma = 2.07$ g/cm^3	
1.30%	2.20%	0.65%	3.02%	4.26%	1.28	3.76%	
4.9%	19.8%	12.5%	21.4%	23.1%	3.7%	9.3%	
0.86 mm	0.86 mm	1.15 mm	1.15 mm	1.15 mm	2.76 mm	6.5 mm	
Bottom		Bottom	Top	Top-Bottom		Top	
Top		Top	Bottom - Middle	Middle		Bottom-Middle	

Table 5.2. Soil passing test GL#1

STAGE	MASS SOIL (g)	ACCUM MASS. (g)	SOIL COLLECTED (%)	ACCUM. (%)	HYD. GRAD. i_{17}
Specimen Preparation	29.75	29.75	1.94	1.94	0
i = 0.1	---	15.77	0.00	1.94	0.2
i = 1.0	0.29	30.04	0.02	1.95	1.1
i = 10	1.73	31.77	0.11	2.07	10.7
i = 18	1.1	32.87	0.07	2.14	17.9
i = 18 (60 min vib.)	101.08	133.95	6.58	8.72	14.0
i = 18 (120 min vib.)	70.21	204.16	4.57	13.29	14.0

Table 5.3 Soil passing test Be#1

STAGE	MASS SOIL (g)	ACCUM MASS. (g)	SOIL COLLECTED (%)	ACCUM. (%)	HYD. GRAD. i_{17}
Specimen Preparation	15.49	15.49	0.97	0.97	0
i = 0.1	---	15.49	0.00	0.97	0.1
i = 1.0	0.15	15.64	0.01	0.97	1.0
i = 10	1.1	16.74	0.07	1.04	9.9
i = 20	2.5	19.24	0.16	1.20	16.9
i = 20 (30 min vib.)	190.57	209.81	11.87	13.07	6.3

Table 5.4 Soil passing test Be#2

STAGE	MASS SOIL (g)	ACCUM MASS. (g)	SOIL COLECTED (%)	ACCUM. (%)	HYD. GRAD i_{17}
Specimen Preparation	12.08	12.08	0.81	0.81	0
i = 0.1	---		--	0.81	0.2
i = 1.0	0.5	12.58	0.03	0.85	1.1
i = 10	1.4	13.98	0.09	0.94	10.5
i = 20	1.9	15.88	0.13	1.07	17.5
i = 20 (1 hour vib.)	400.8	416.64	26.96	28.03	1.5
i = 20 (2 hour vib.)					

Table 5.5 Soil passing test Be#3

STAGE	MASS SOIL (g)	ACCUM MASS. (g)	SOIL COLLECTED (%)	ACCUM. (%)	HYD. GRAD. i_{17}
Specimen Preparation	17.22	17.22	1.13	1.13	0
i = 0.1	---		--	1.13	0.2
i = 1.0	0.53	17.75	0.03	1.17	1.3
i = 10	0.92	18.67	0.06	1.23	11.3
i = 20	1.52	20.19	0.10	1.33	18.5
i = 20 (1 hour vib.)	507.9	528.1	33.47	34.80	10.0

Table 5.6 Soil passing test Be#4

STAGE	MASS SOIL (g)	ACCUM MASS. (g)	SOIL COLLECTED (%)	ACCUM. (%)	HYD. GRAD. i_{17}
Specimen Preparation	9.22	9.22	0.61	0.61	0
i = 0.1	---	9.22	--	0.65	0.1
i = 1.0	0.23	9.45	0.02	0.62	1.2
i = 10	0.32	9.77	0.02	0.64	9.1
i = 20	0.82	10.59	0.05	0.70	14.9
i = 20 (1 hour vib.)	179.21	189.8	11.80	12.50	10.4
i = 20 (2 hour vib.)	0.66	190.5	0.04	12.54	9.5

Table 5.7 Soil passing test Be#5

STAGE	MASS SOIL (g)	ACCUM MASS. (g)	SOIL COLLECTED (%)	ACCUM. (%)	HYD. GRAD. i_{17}
Specimen Preparation	47.63	47.63	3.02	3.02	0
i = 0.1	---		--	3.02	0.1
i = 1.0	2.75	50.38	0.17	3.20	1.2
i = 10	204.87	255.25	13.00	16.20	9.1
i = 20	81.88	337.13	5.20	21.39	14.9

Table 5.8 Soil passing test Be#6

STAGE	MASS SOIL (g)	ACCUM MASS. (g)	SOIL COLLECTED (%)	ACCUM. (%)	HYD. GRAD. i_{17}
Specimen Preparation	67.7	67.7	4.26	4.26	0
i = 0.1	---		--	4.26	0.2
i = 1.0	1.5	69.2	0.09	4.36	1.0
i = 10	297.91	367.11	18.77	23.13	10.0
i = 20	0	367.11	0.00	23.13	

Table 5.9 Soil passing test Be#7

STAGE	MASS SOIL (g)	ACCUM MASS. (g)	SOIL COLLECTED (%)	ACCUM. (%)	HYD. GRAD. i_{17}
Specimen Preparation	19.06	19.06	1.28	1.28	0
i = 0.1	---		--	1.28	
i = 1.0	0.1	19.16	0.01	1.29	1.0
i = 10	0.1	19.26	0.01	1.30	10.0
i = 10 (1 hour vib.)	35.46	54.72	2.39	3.69	10.0

Table 5.10 Soil passing test Be#8

STAGE	MASS SOIL (g)	ACCUM MASS. (g)	SOIL COLLECTED (%)	ACCUM. (%)	HYD. GRAD. i_{17}
Specimen Preparation	58.58	58.58	3.76	3.76	0
i = 0.1	0.25	58.83	0.02	3.78	0.1
i = 0.1 (1 hour vib)	74.06	132.89	4.76	8.54	0.1
i = 0.1 (1 1/2 hour vib)	12.21	145.1	0.78	9.32	0.1

Table 6.1 Loss of soil through particle migration

Soil code	Total Loss (%)	Comments
Kenney and Lau(1985)		
K	0	manual tapping; D_{15} = 20 mm
D	19.0	manual tapping; D_{15} = 20 mm
Honjo et al. (1996)		
G1-D	2.5	manual tapping; 0.6 mm screen
G3-C	20.0	manual tapping; 0.83 mm screen
G4-C	33.0	manual tapping; 1.2 mm screen
UBC (test code)		
K (Be #7)	3.7	automatic tapping (60 min); 2.76 mm screen
D (Be #8)	9.3	automatic tapping (60 min); 6.5 mm screen
G1-D (Be #4)	12.5	automatic tapping (60 min); 1.15 mm screen
G3-C (Be #1)	13.0	automatic tapping (20 min); 0.86 mm screen
G3-C (Be #2)	28.0	automatic tapping (60 min); 0.45 mm screen
G3-C (Be #3)	34.8	automatic tapping (60 min); 0.86 mm screen
G4-C (Be #5)	21.4	no vibration; 1.15 mm screen
G4-C (Be #6)	23.1	no vibration; 1.15 mm screen

Table 6.2. Prediction of soil passing versus soil passing during tests.

SOIL GRADATION	SOIL PASSING THROUGH	PREDICTION	COMMENTS
G1-D	13.42	8	
G3-C	13	35	glass beads
G3-C	13	35	20 min vib.
G3-C	19.81	35	dry
G3-C	28	35	mesh 0.45 mm
G3-C	34.8	35	
G4-C	21.39	40	no vibration
G4-C	23.13	40	no vibration
K	3.69	3	
D	9.32	12.5	

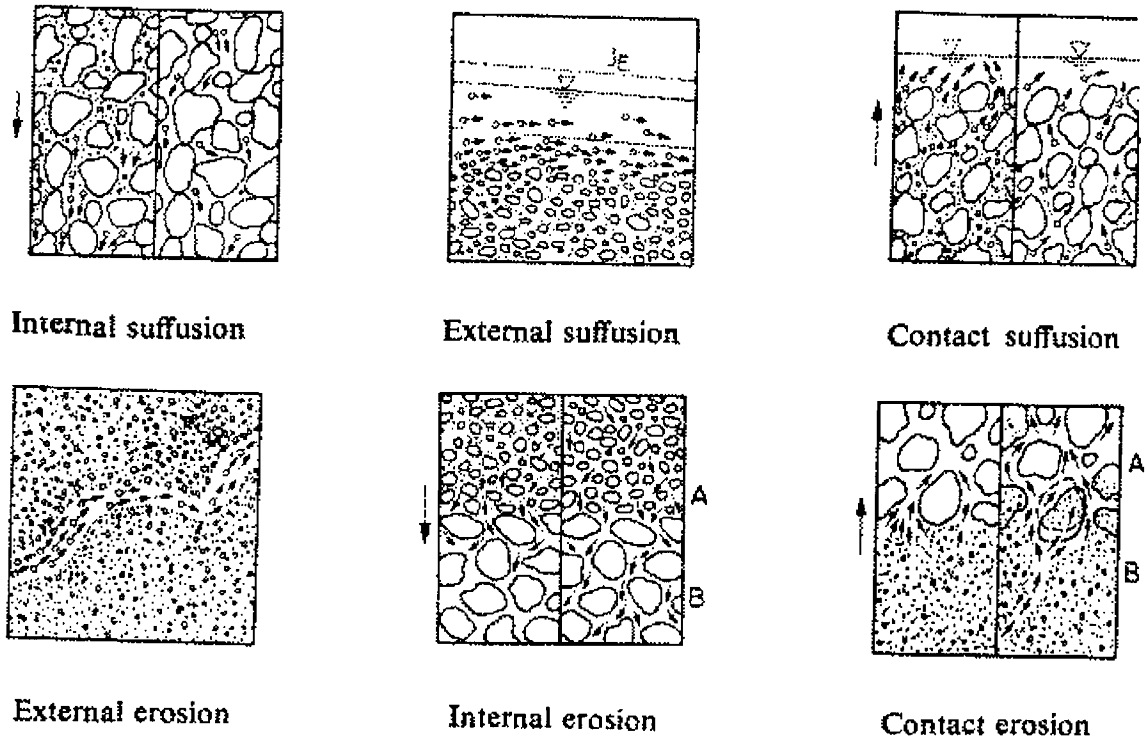

Figure 1.1 Classification of soil migration due to seepage water (after Kezdi, 1979)

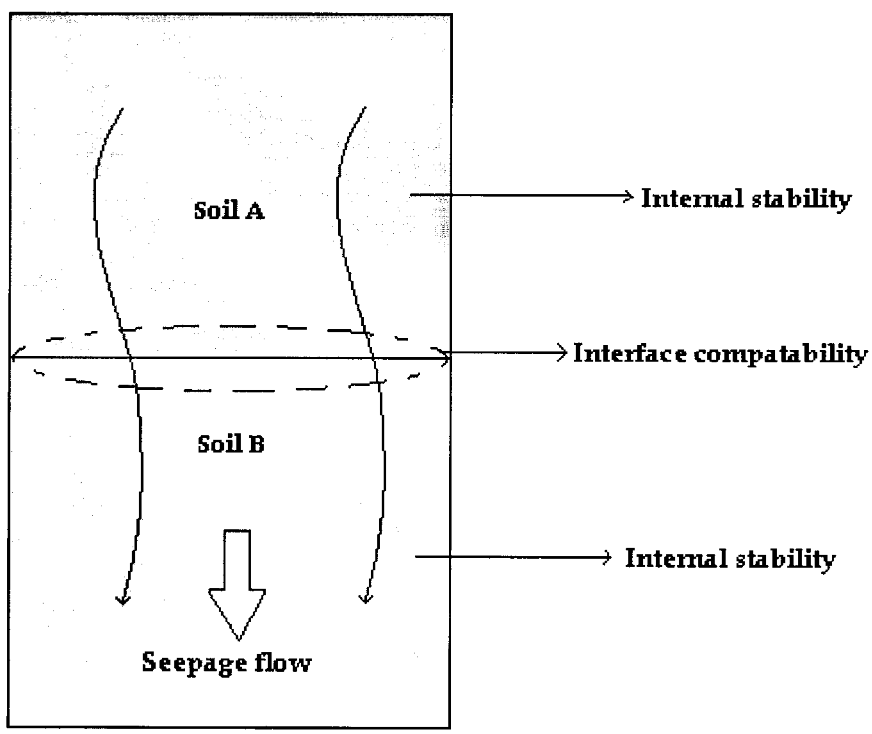

Figure 1.2. Elements for stable seepage in soil structures

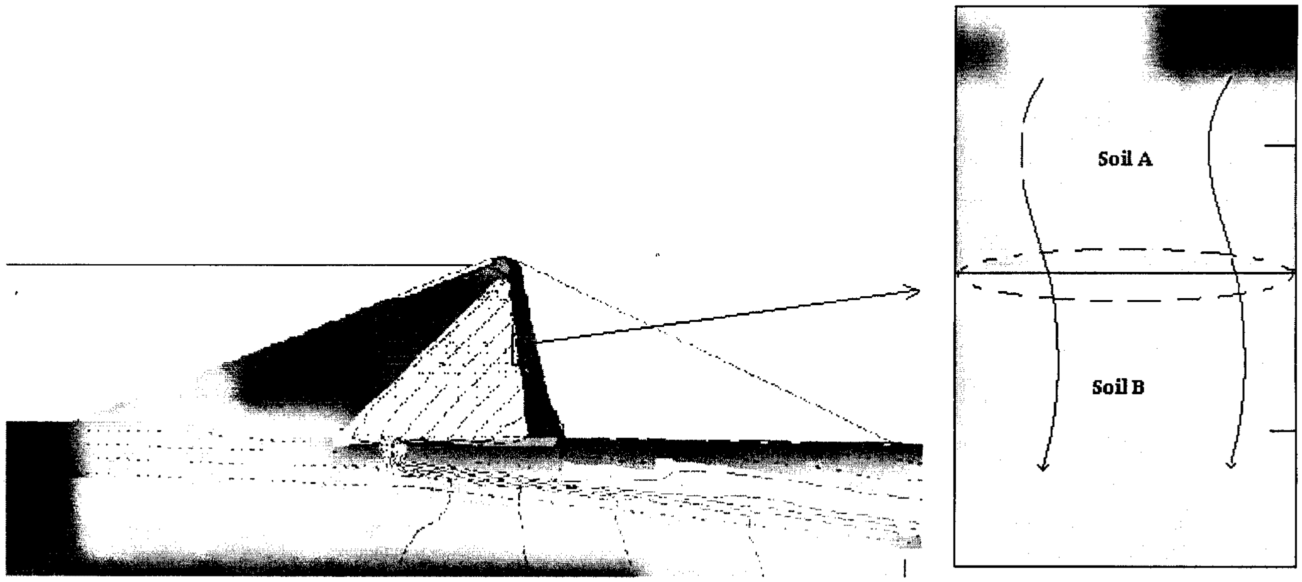

Figure 1.3. Field conditions simulation on laboratory tests.

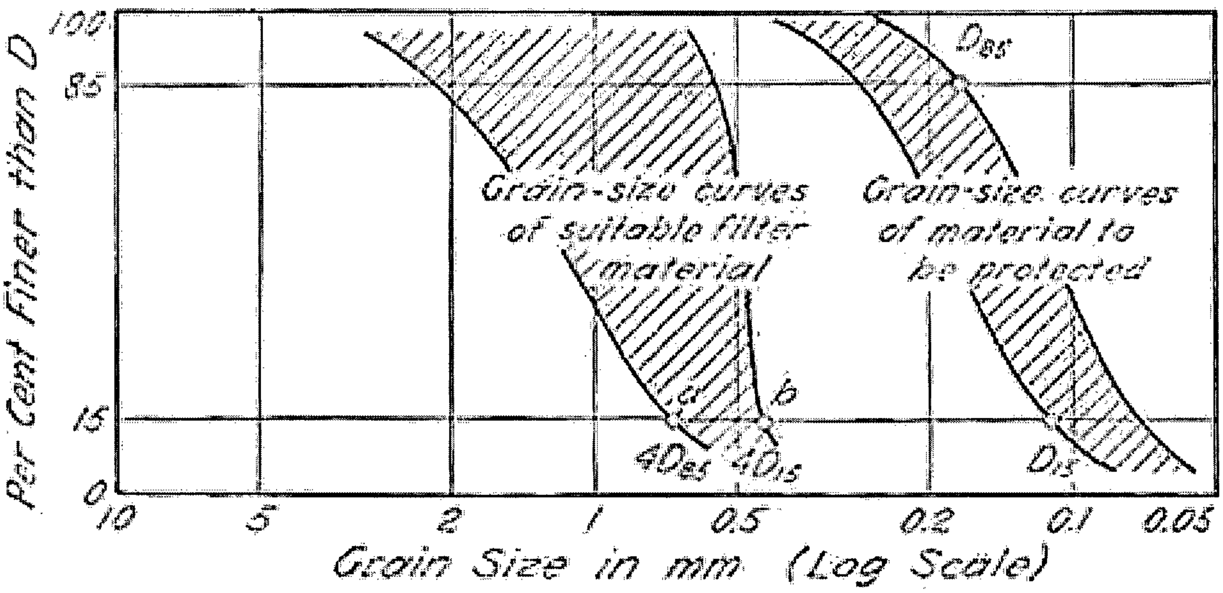

Figure 2.1. Filter criteria (after Terzaghi, 1929)

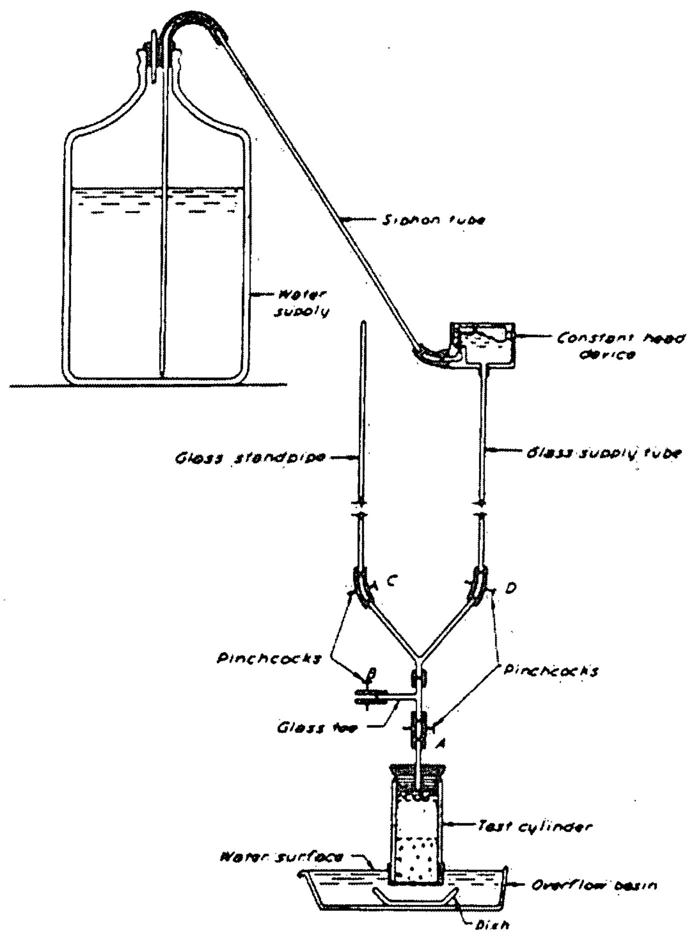

Figure 2.2 Constant head apparatus (after Bertram, 1940)

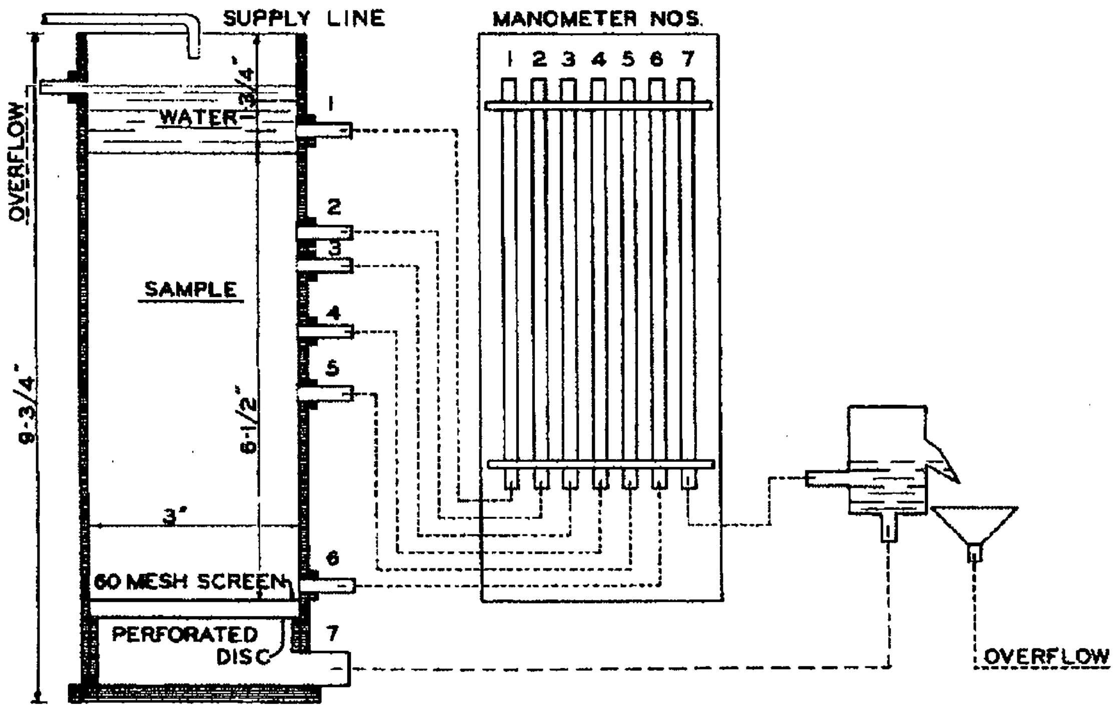

Figure 2.3. Small permeameter (U.S Army corps of Engineers, 1941)

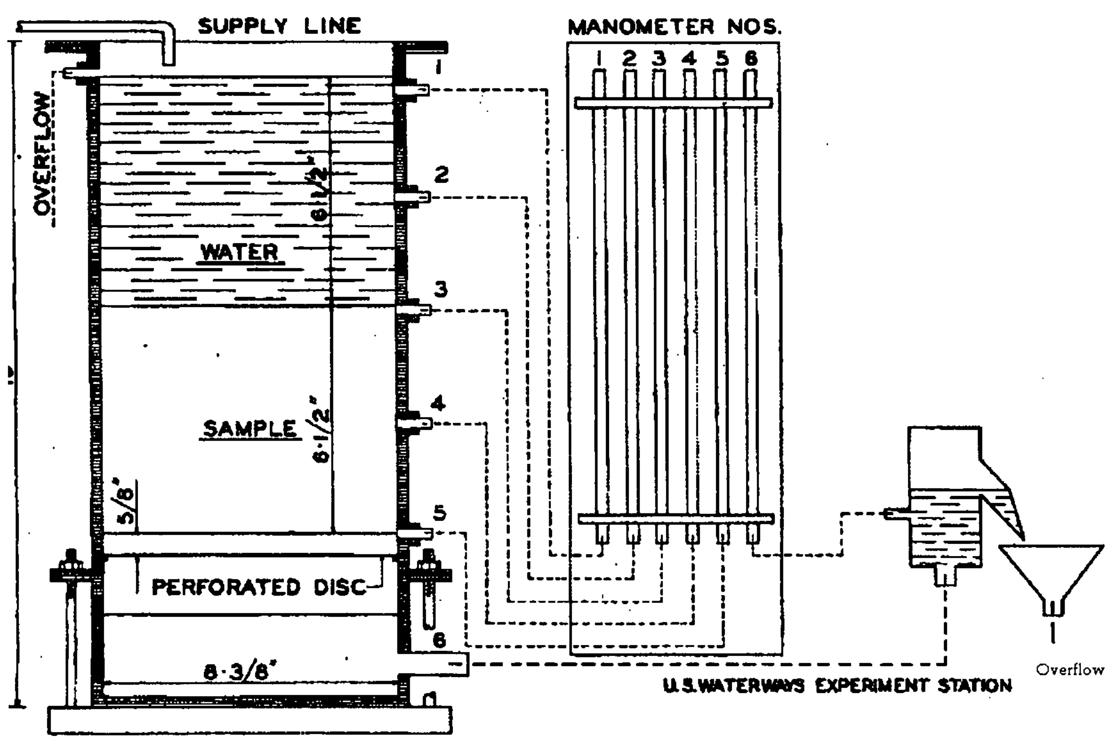

Figure 2.4. Large permeameter (U.S Army corps of Engineers, 1941)

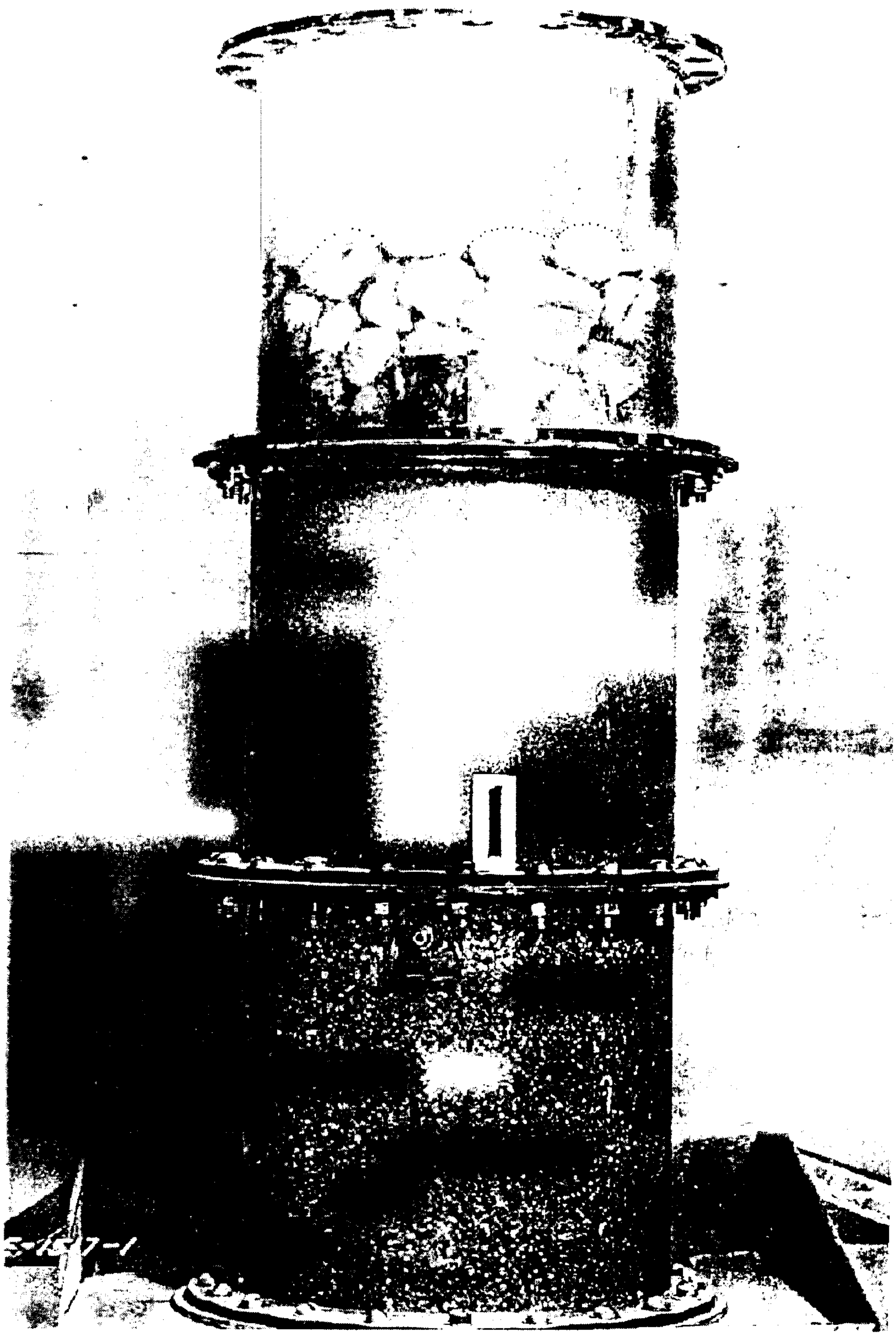

Figure 2.5. Permeameter used by United States Bureau of Reclamation (1947-1955)

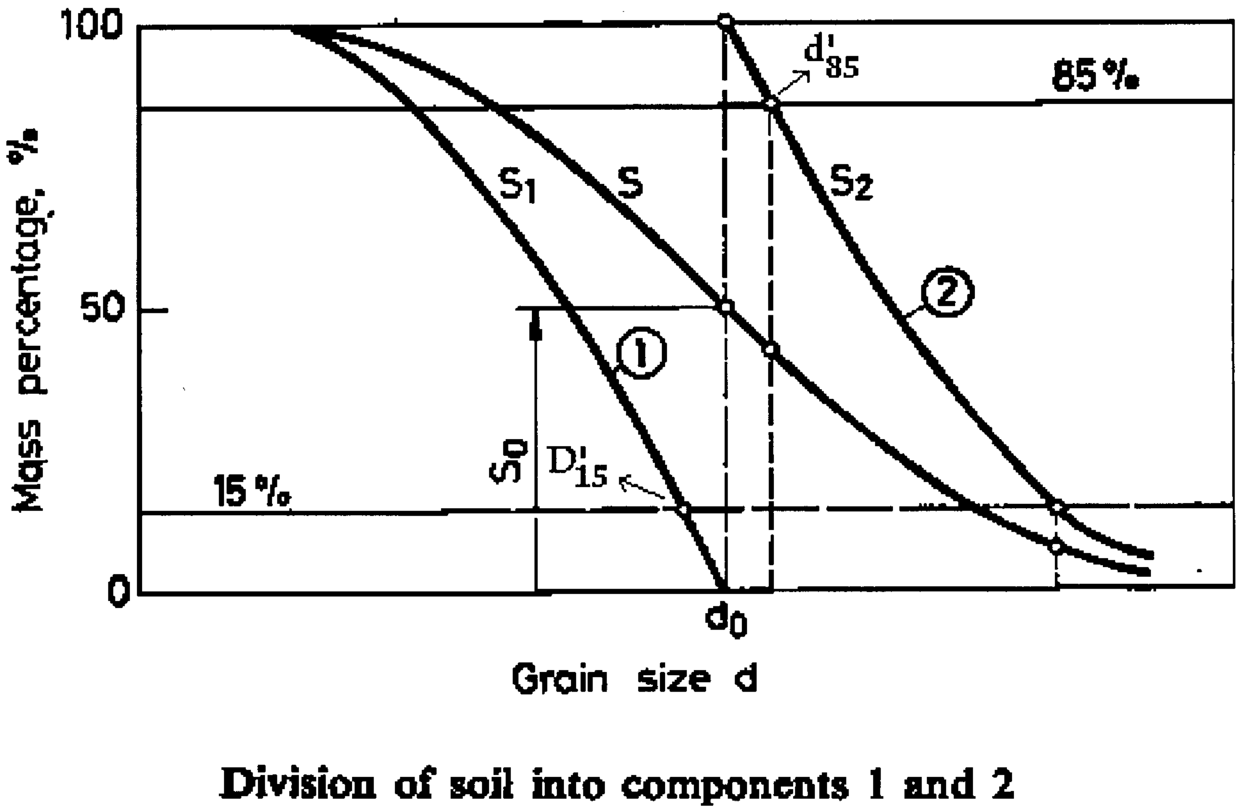

Figure 2.6. A method to assess internal stability (after Kézdi, 1970)

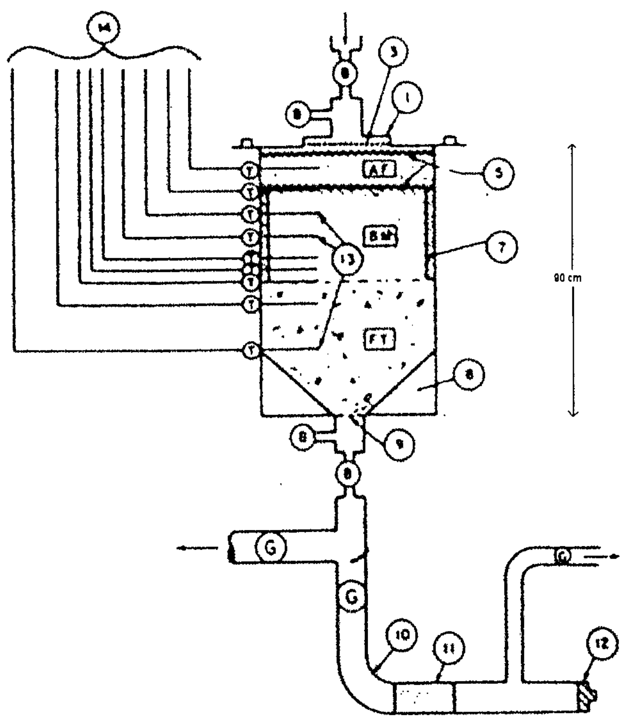

Figure 2.7. Downward permeameter. Pare (1982)

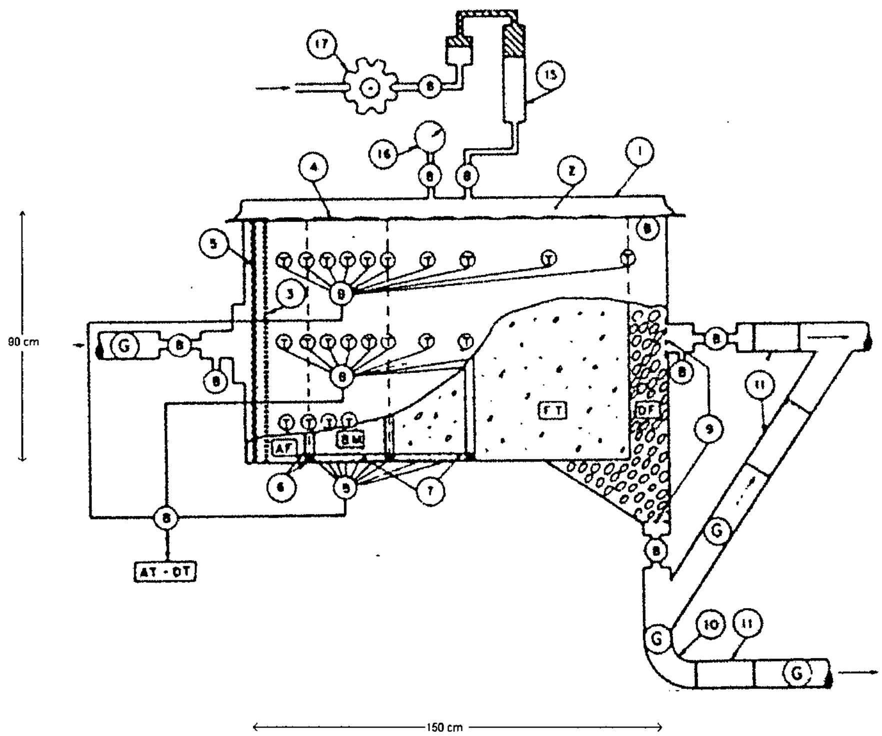

Figure 2.8. Horizontal Permeameter. Pare (1982)

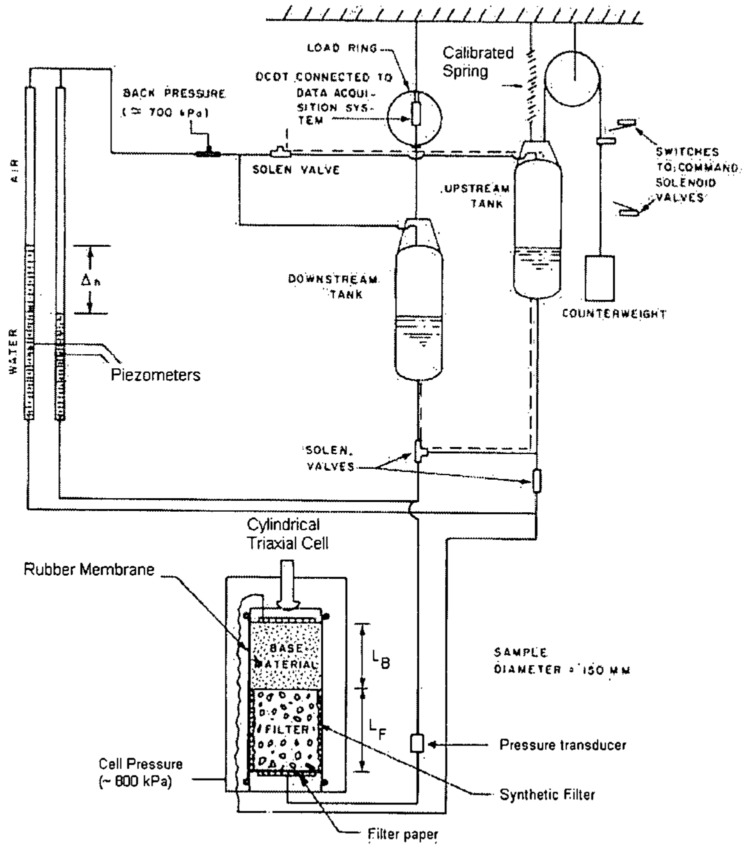

Figure 2.9. Permeameter test assembly (Lafleur, 1984)

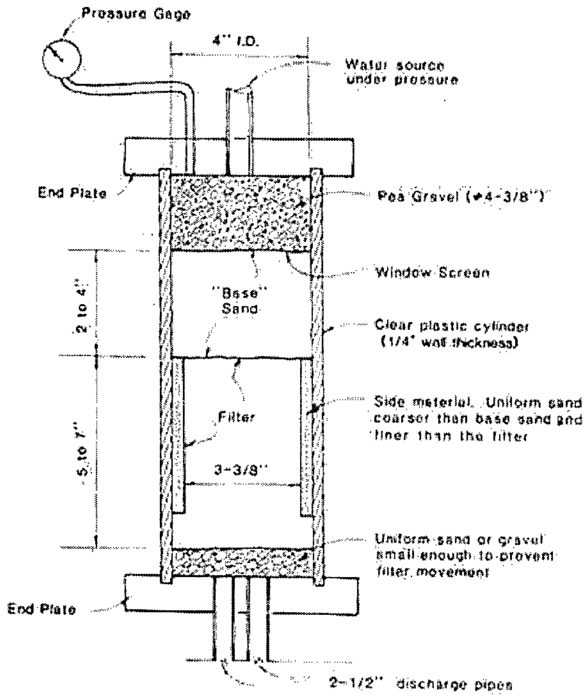

Figure 2.10. Permeameter device (after Sherard et al., 1984)

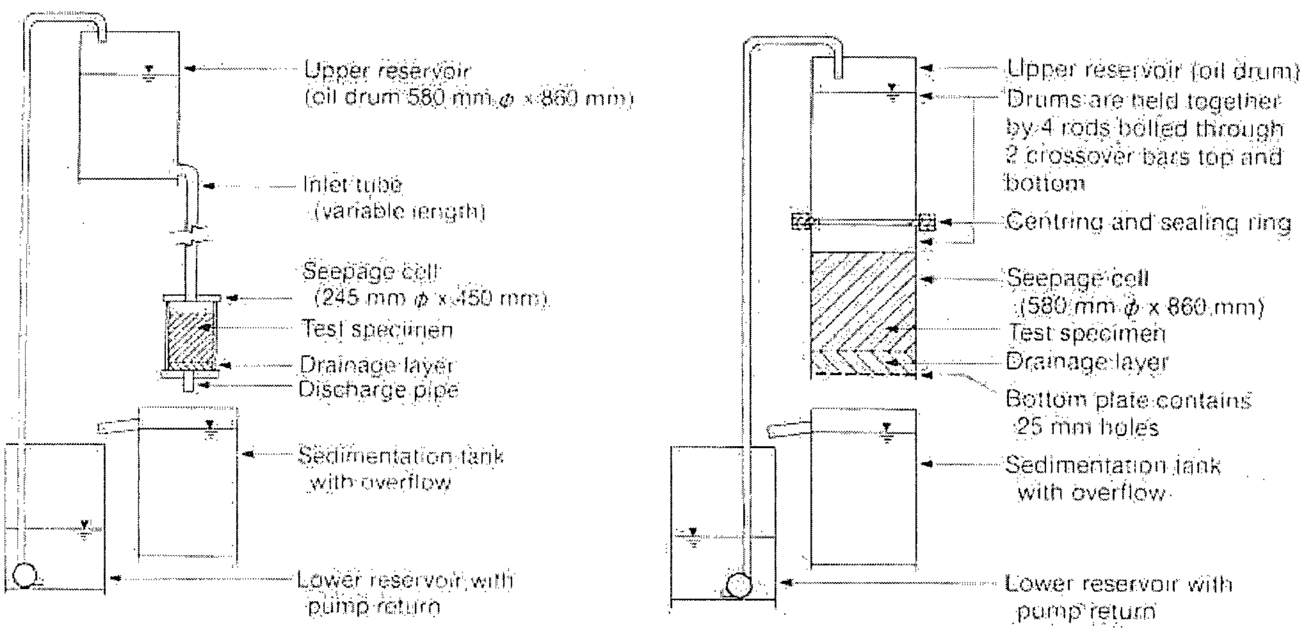

Figure 2.11. Test arrangement (after Kenney and Lau, 1985)

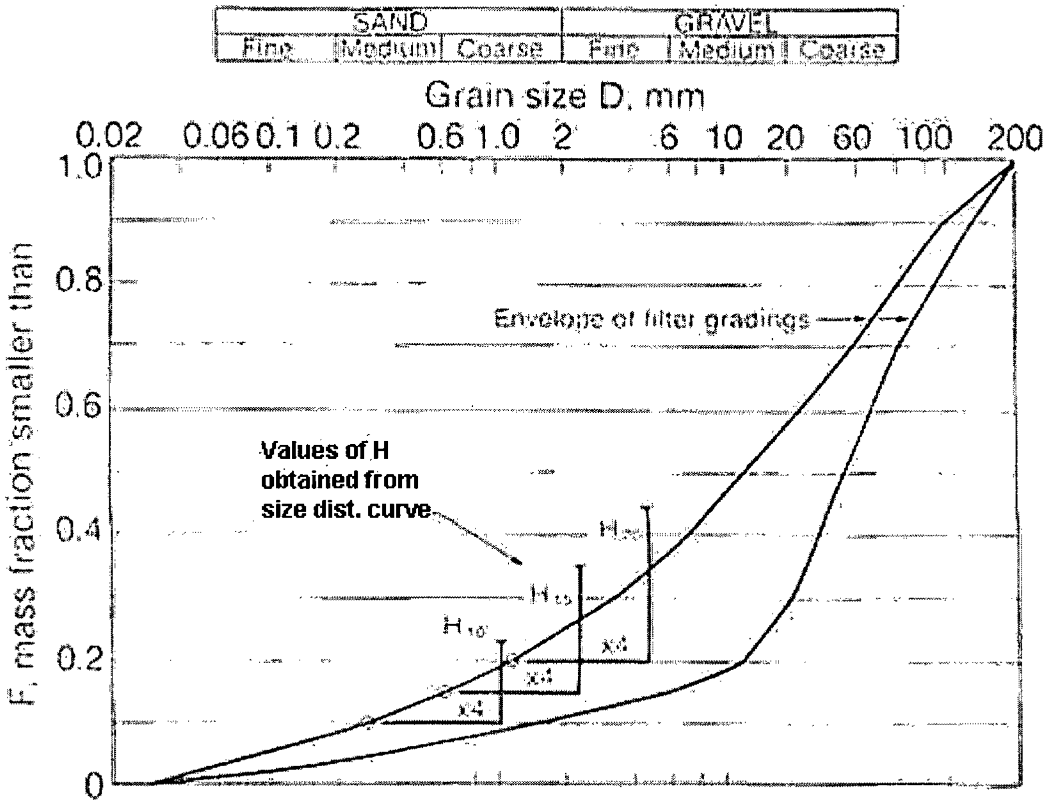

Figure 2.12. Stability criterion (after Kenney and Lau, 1985)

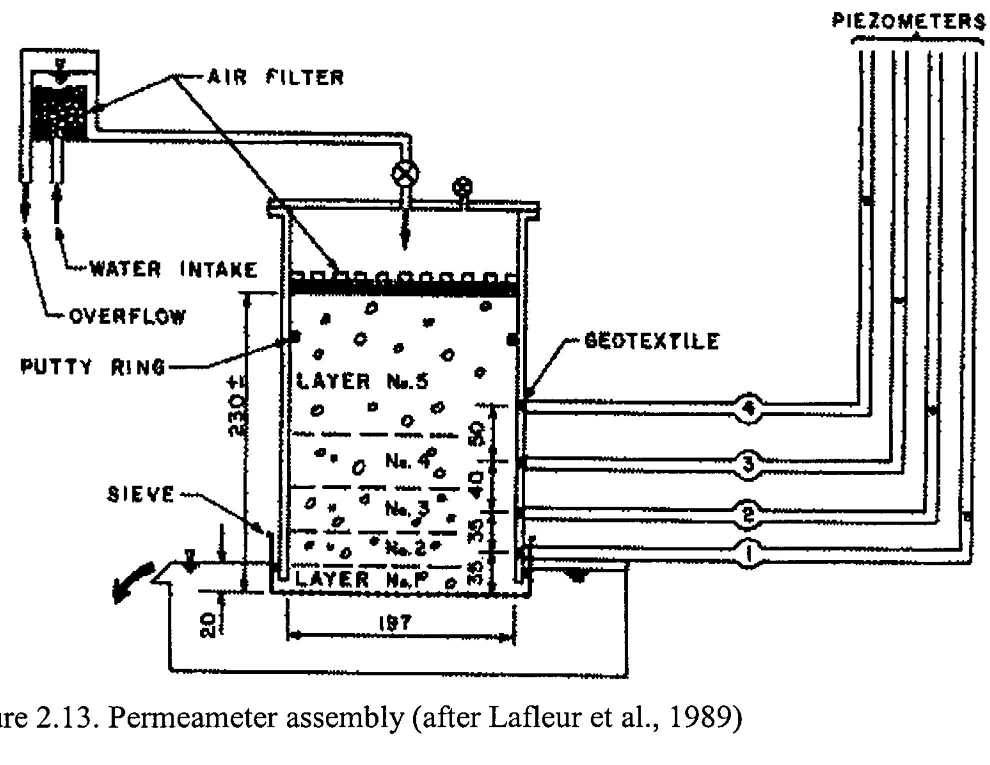

Figure 2.13. Permeameter assembly (after Lafleur et al., 1989)

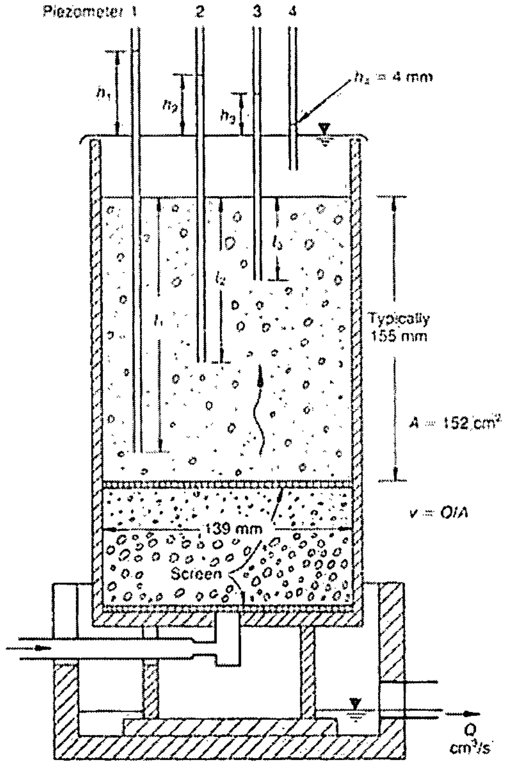

Figure 2.14. Test specimen (after Skempton and Brogan, 1994)

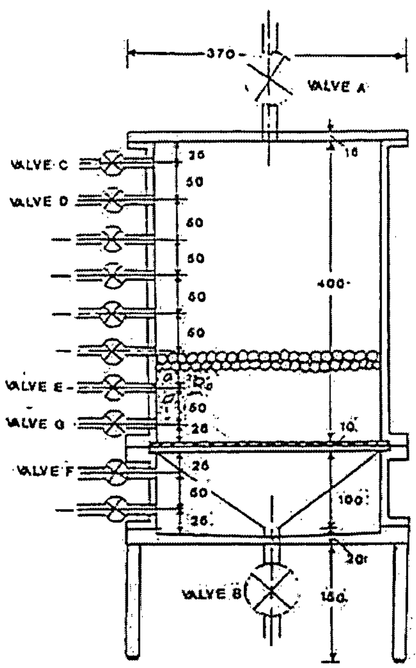

Figure 2.15. Permeameter setup (after Honjo et al., 1996)

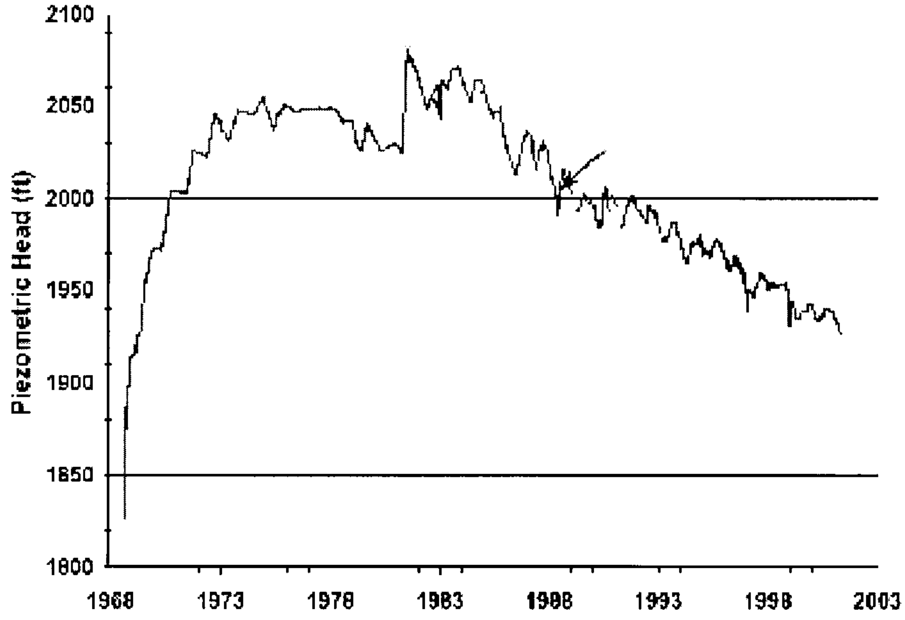

Figure 2.16. Pore pressure dissipation measured on a weir (after Stewart and Garner, 2000)

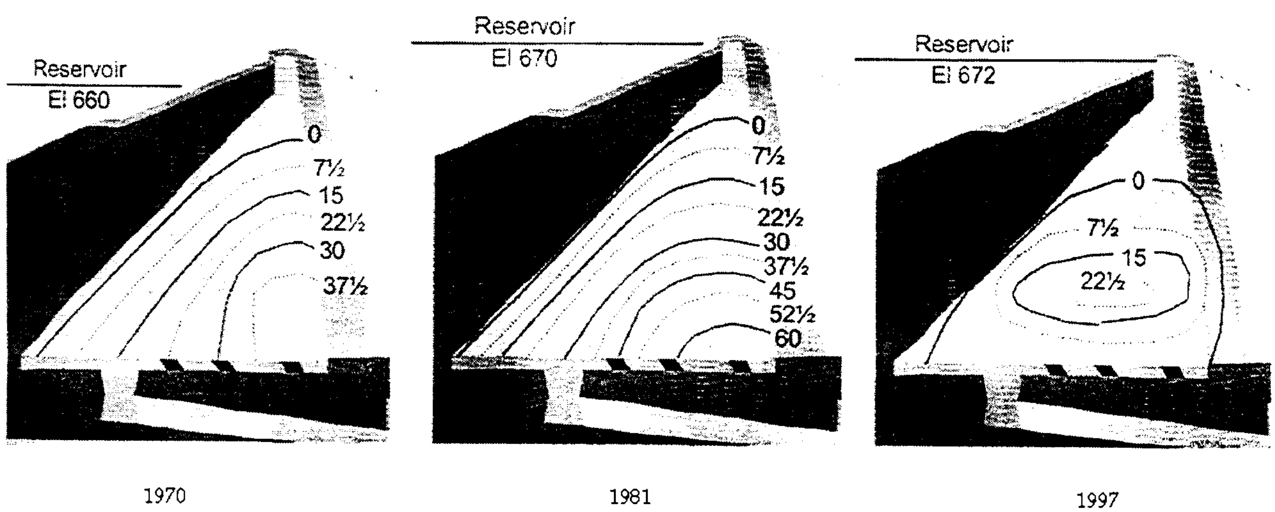

Figure 2.17. Excess pore pressure in a section of the dam (after Stewart and Garner, 2000)

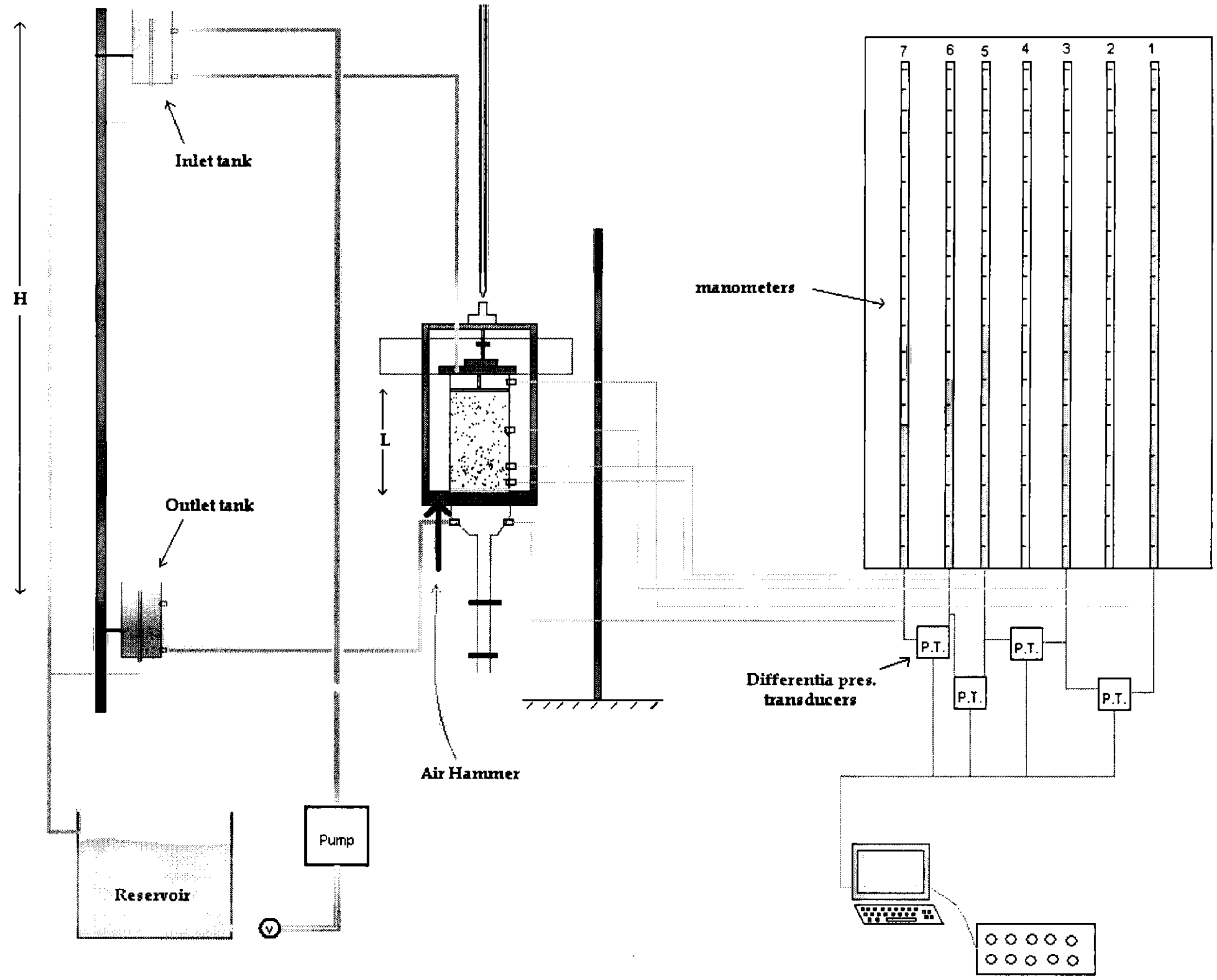

Figure 3.1. Permeameter test assembly.

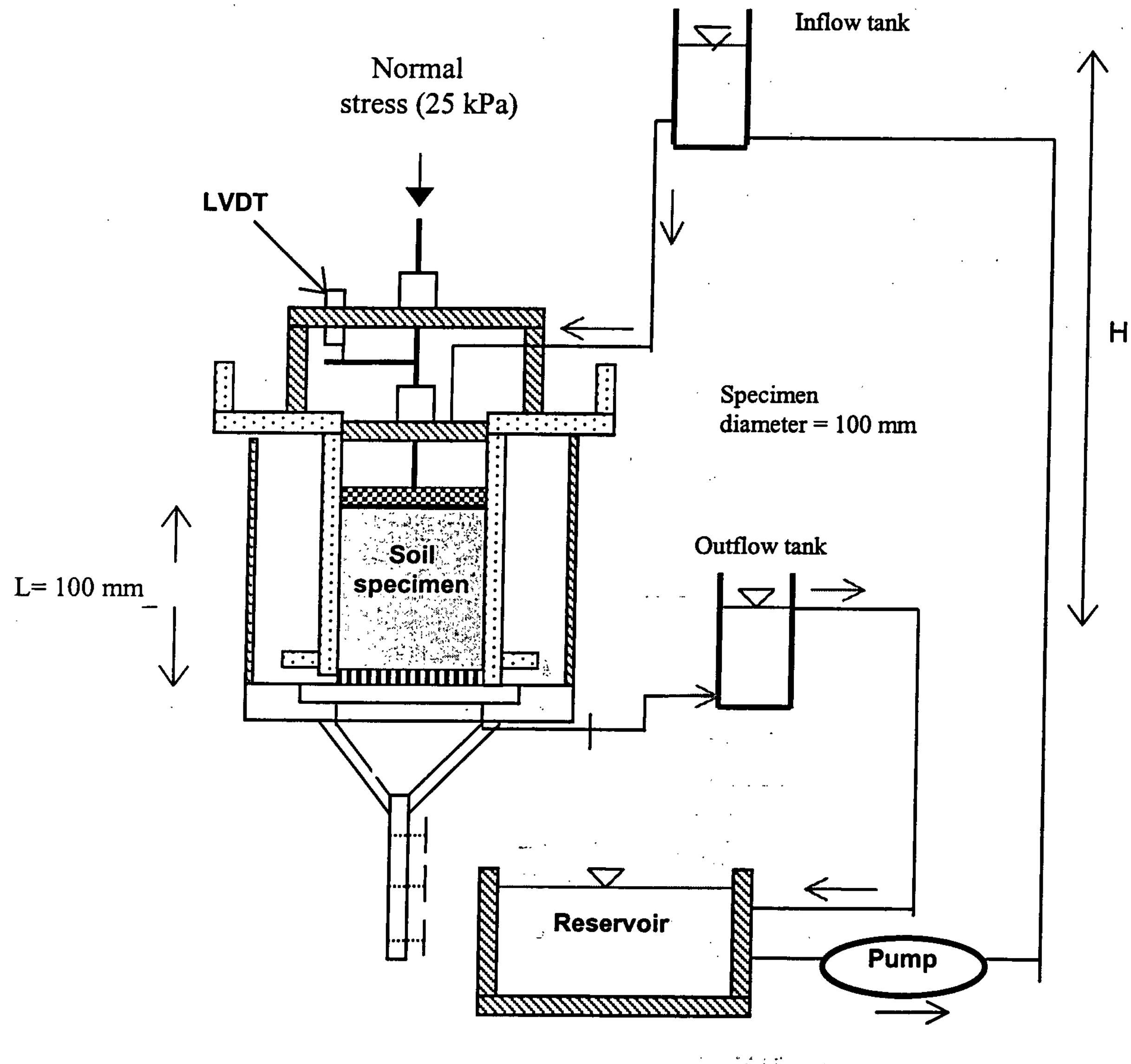

Figure 3.2. Details of the permeameter and flow control system

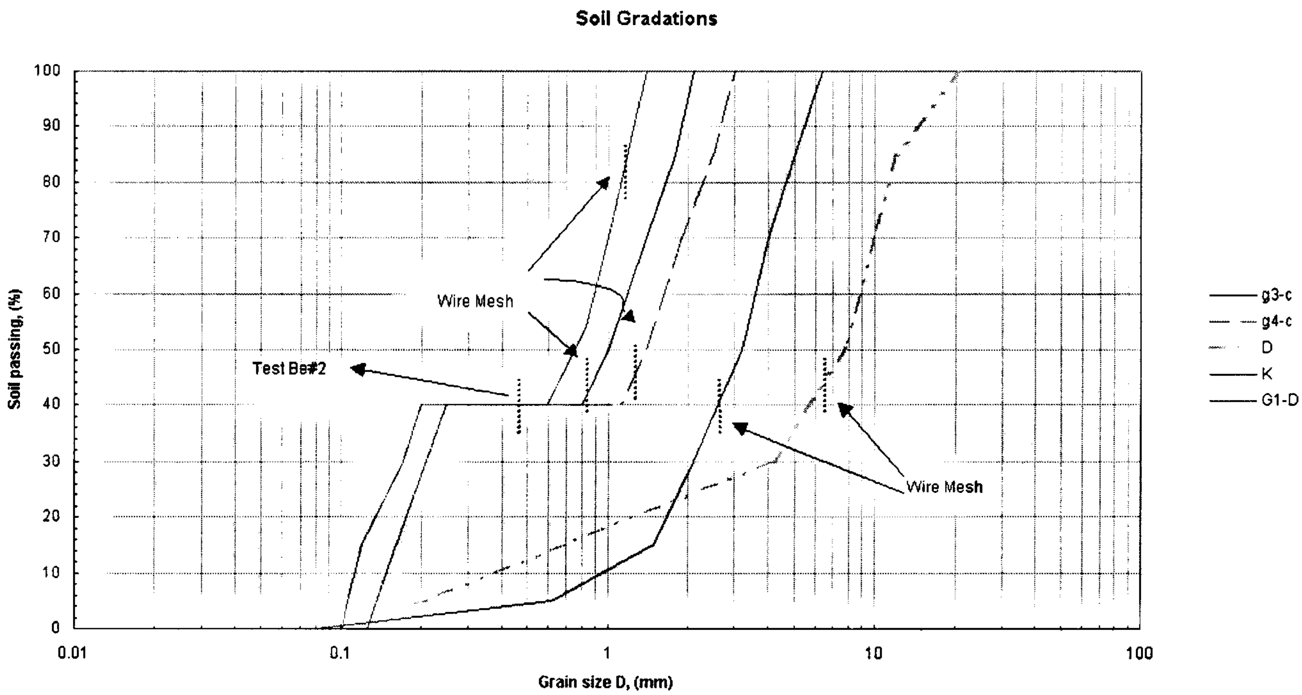

Figure 3.3. Screen sizes used during the research.

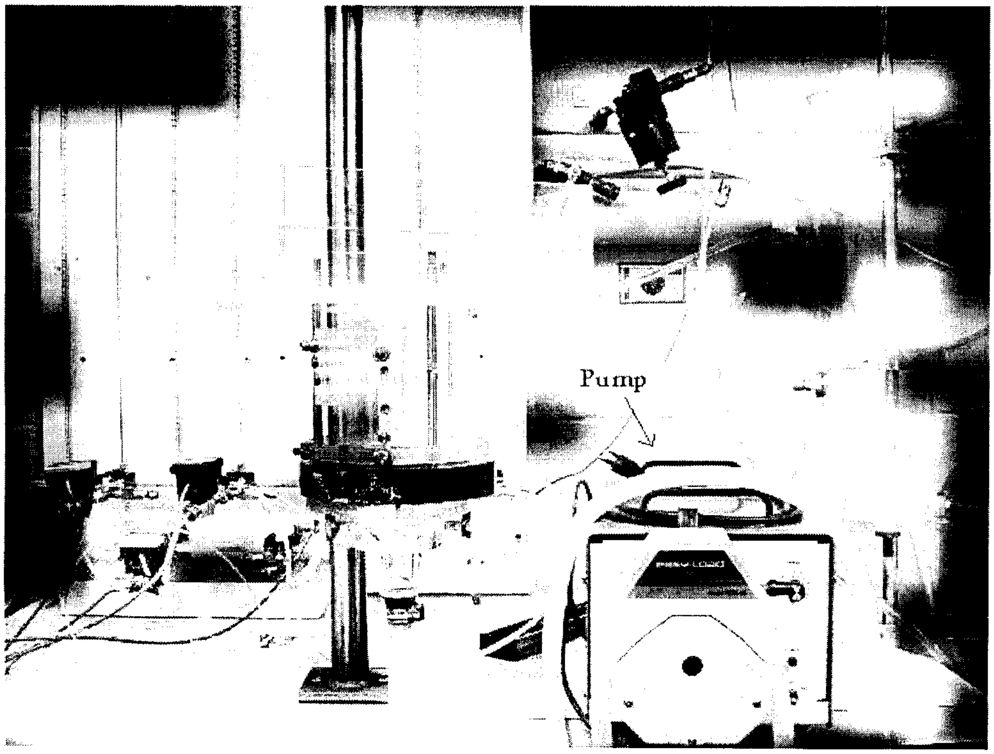

Figure 3.4. Water supply system

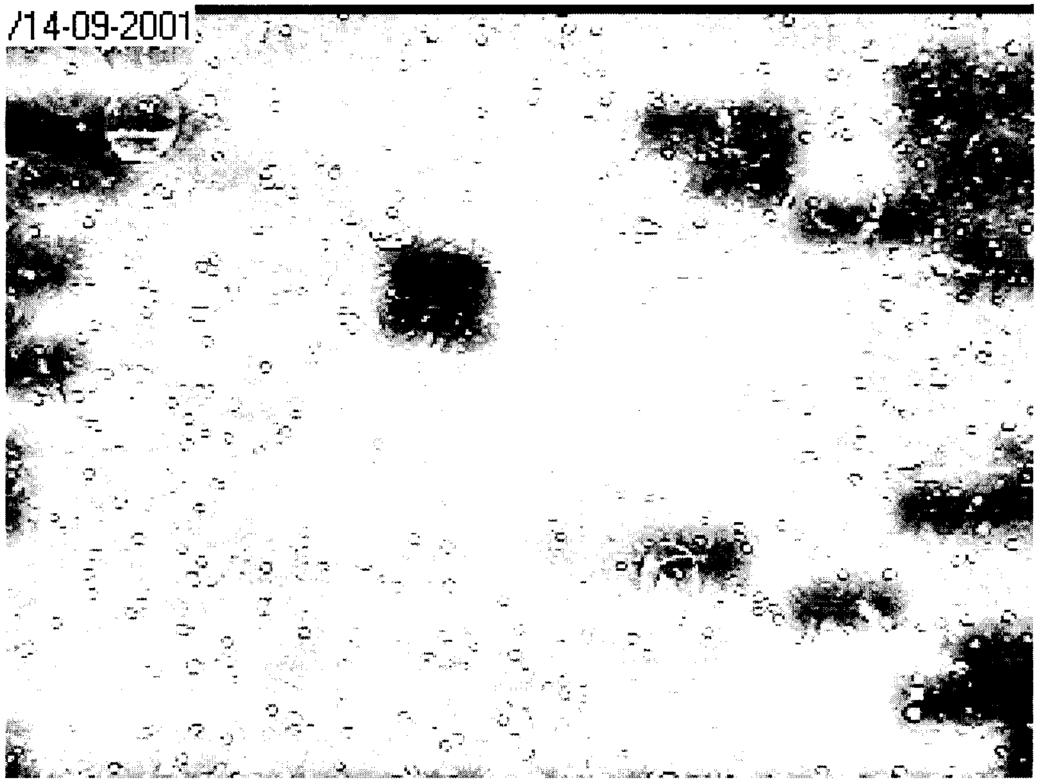

Figure 4.1. Glass beads particles.

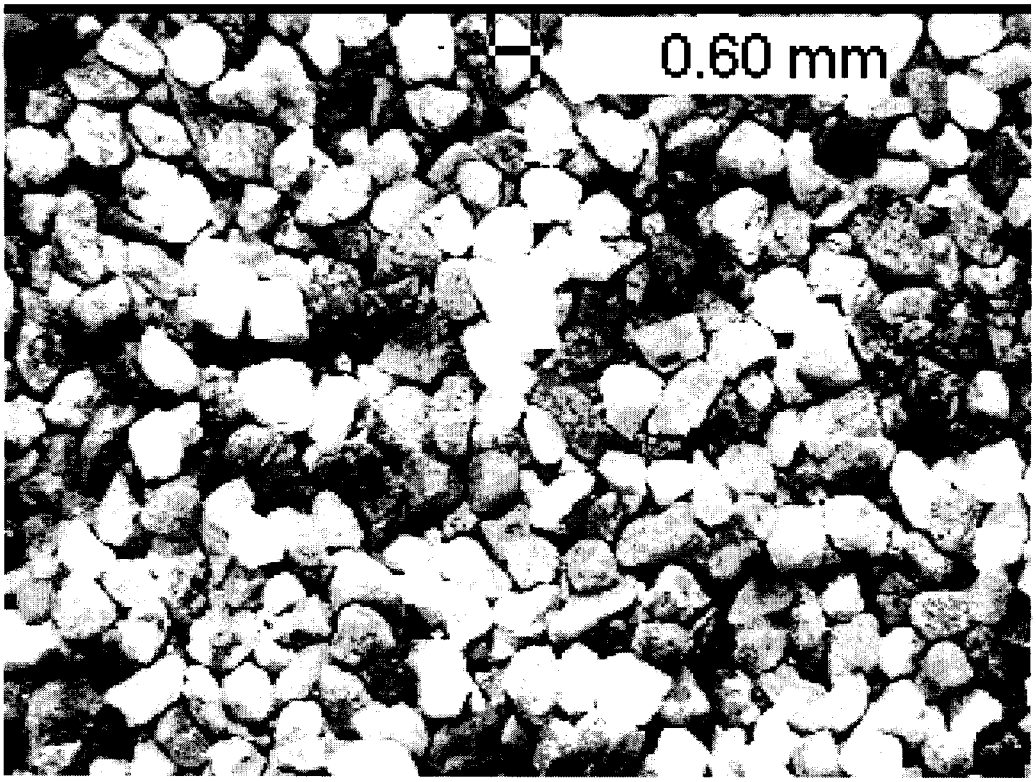

Figure 4.2. Soil particles between 0.58 to 0.83 mm

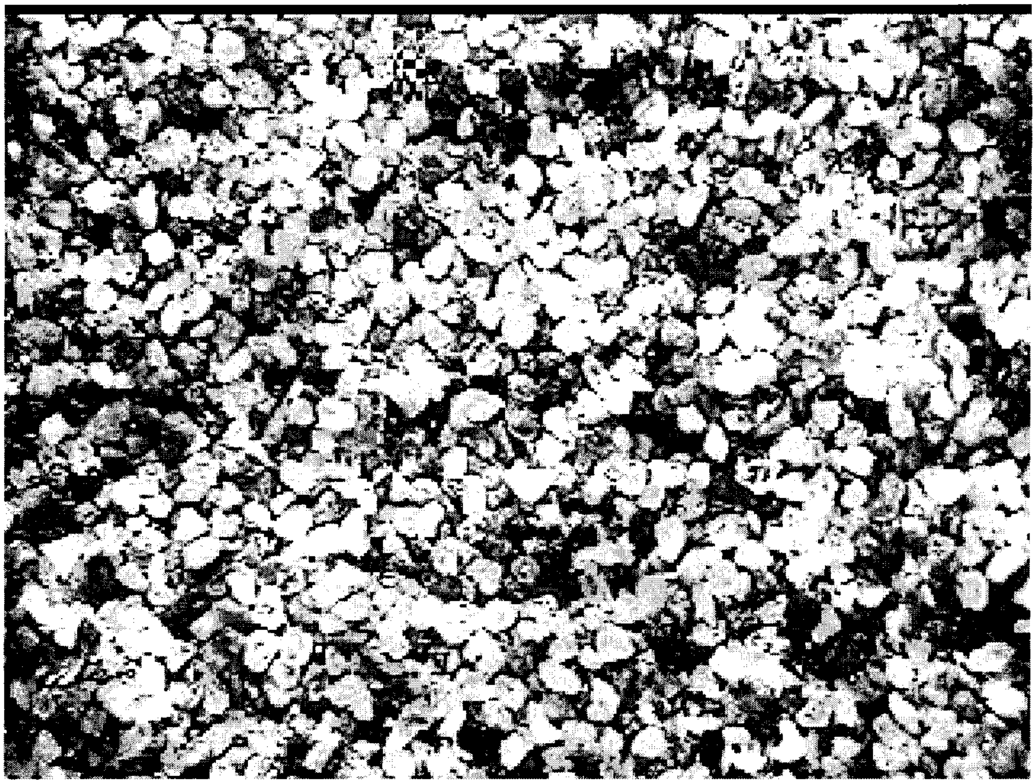

Figure 4.3. Soil particles between 0.14 to 0.24 mm

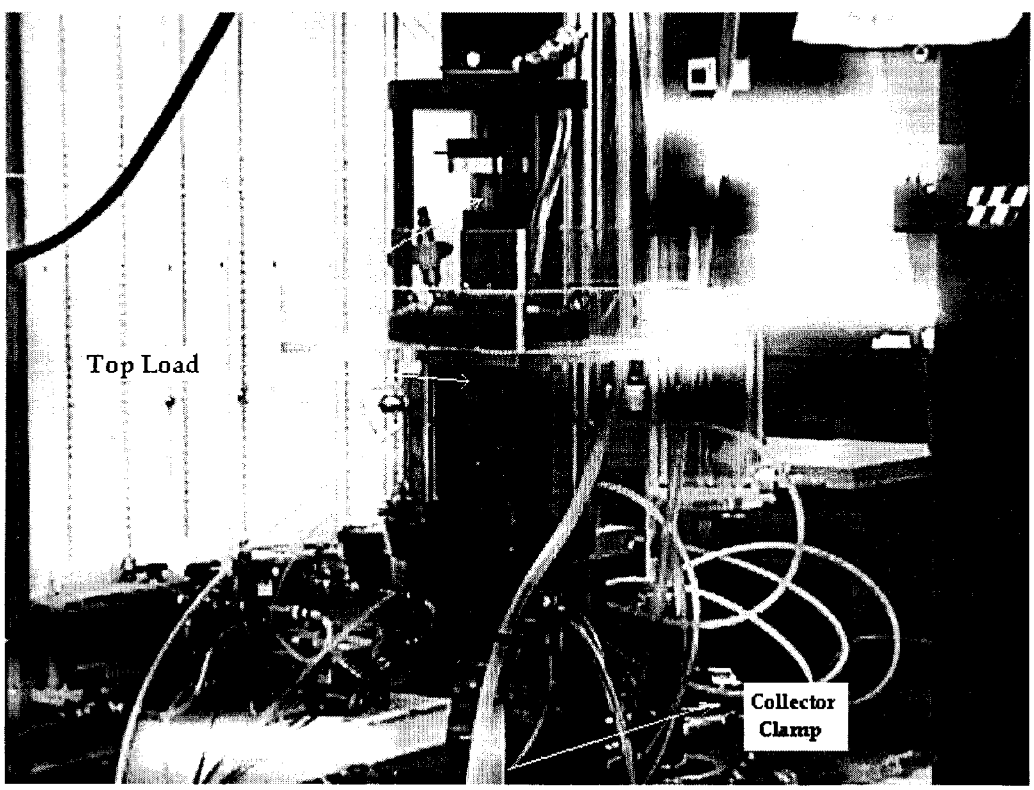

Figure 4.4. Test set-up

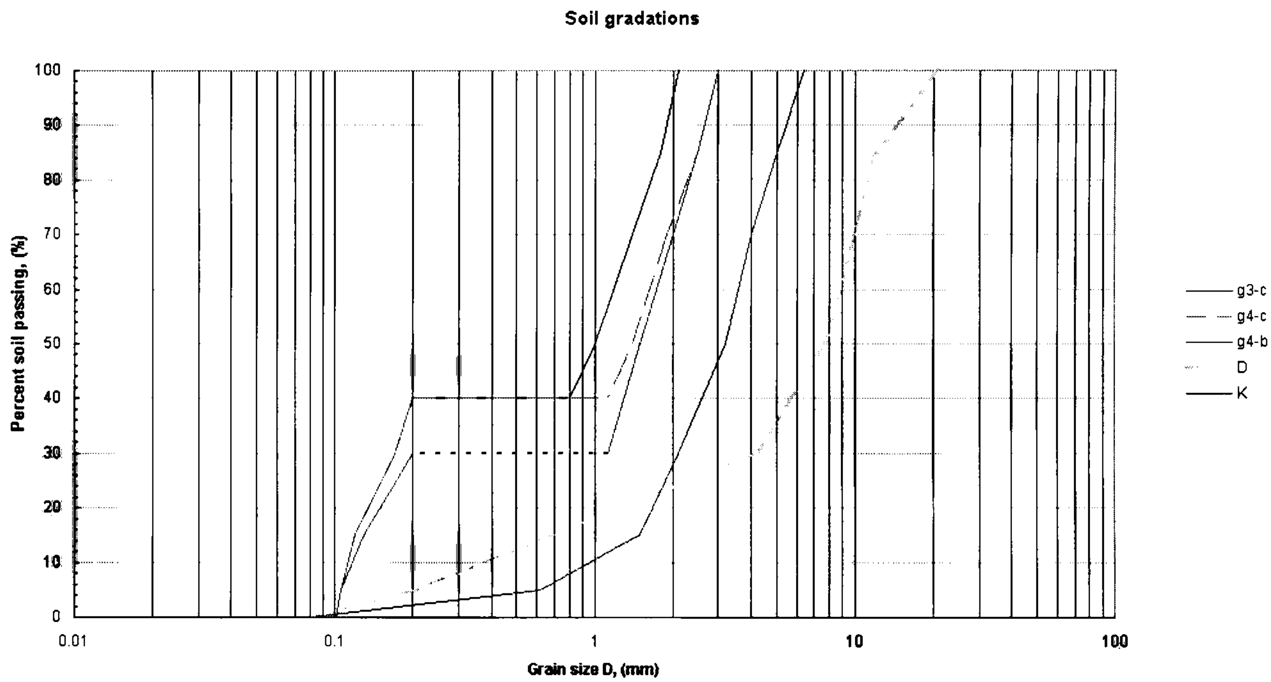

Figure 4.5. Soil gradations.

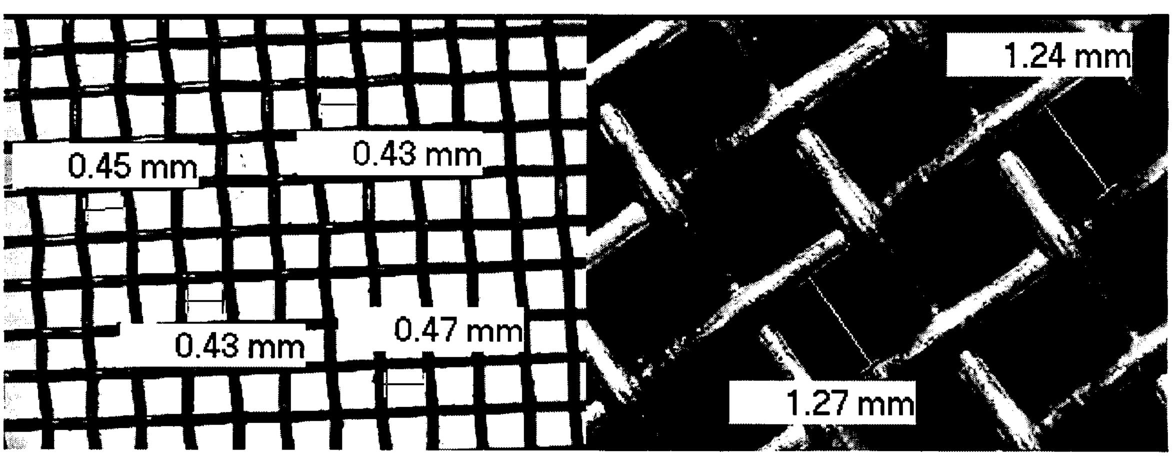

Figure 4.6. Wire mesh screens

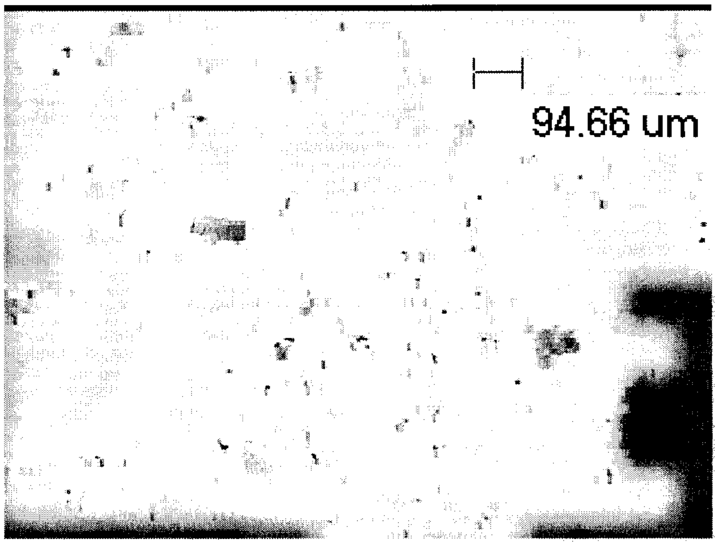

Figure 5.1. Top blinding before the use of distilled water (test C#2).

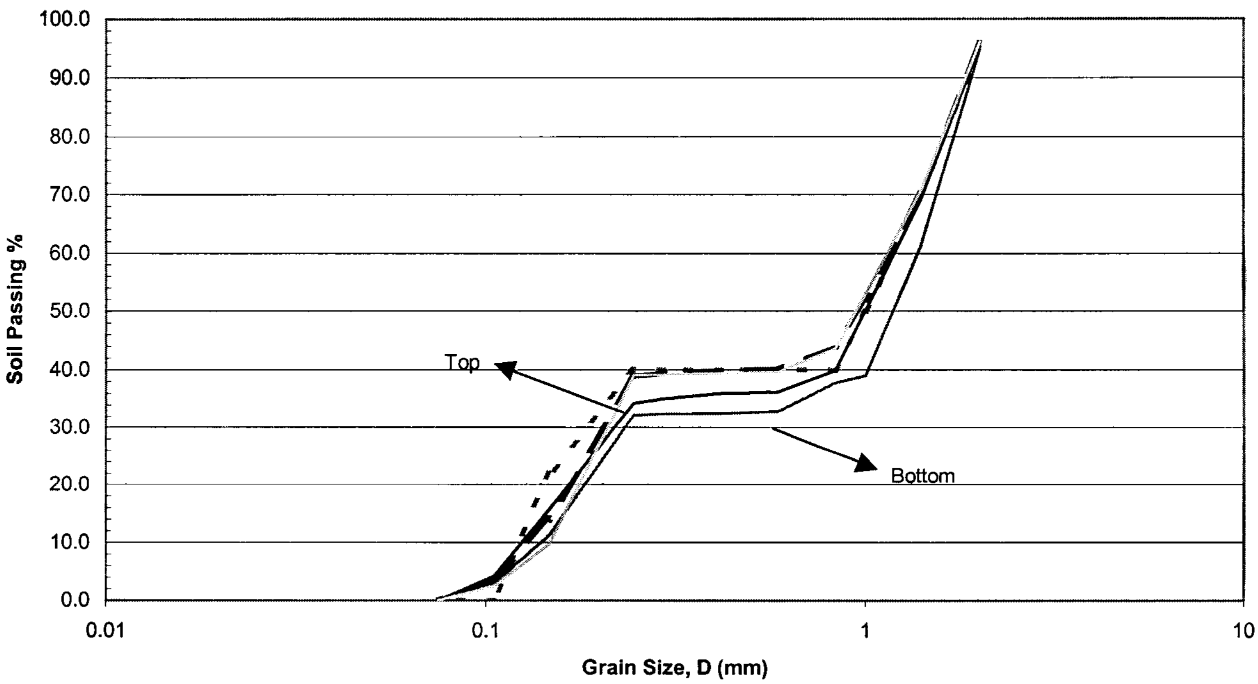

Figure 5.2. Homogeneity confirmation of specimen preparation (test SP#1).

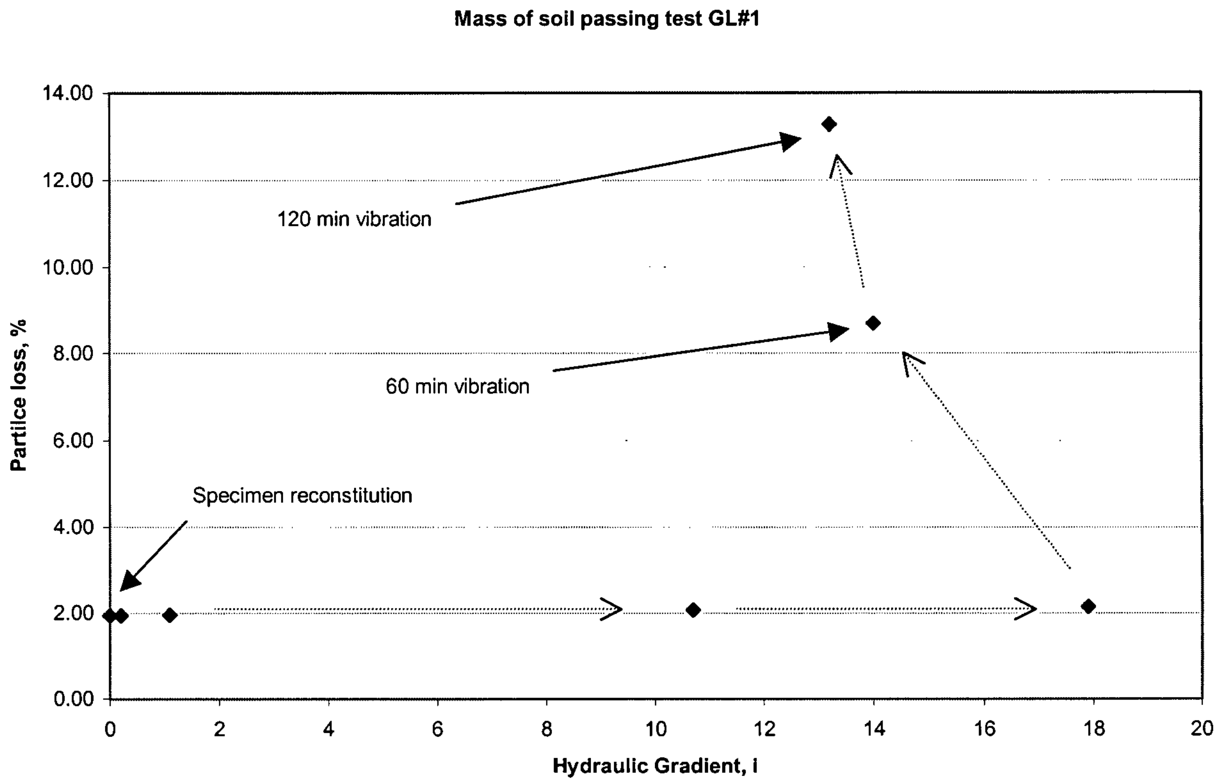

Figure 5.3. Soil passing test GL#1

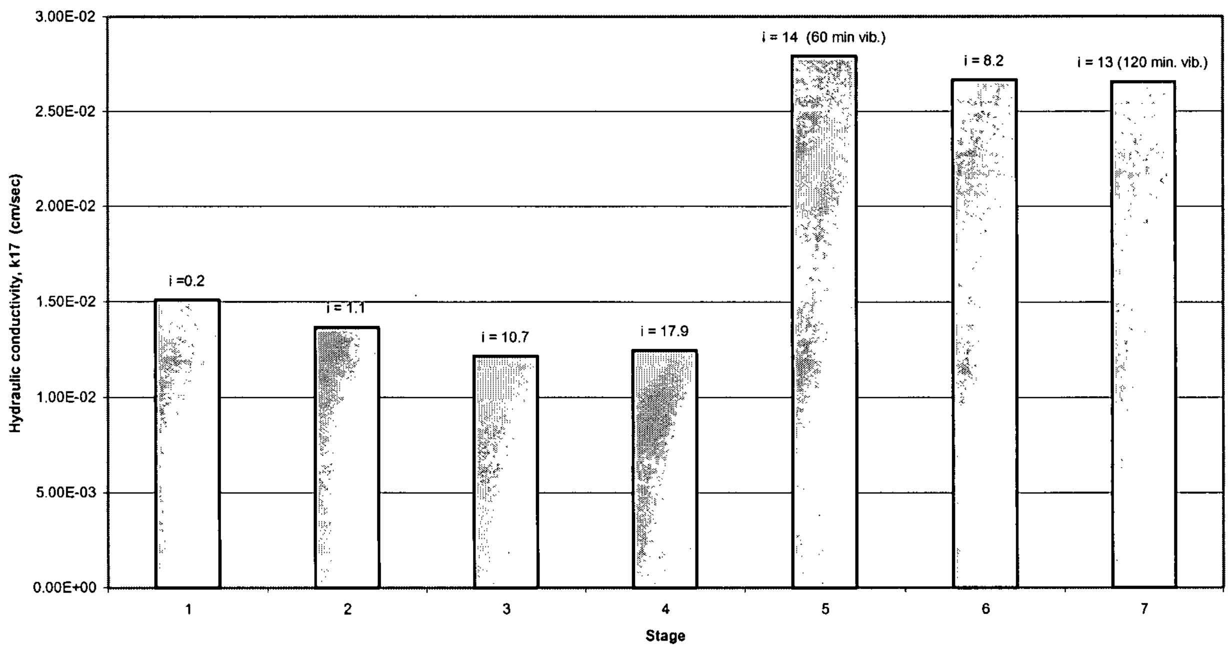

Figure 5.4 Hydraulic conductivity test GL#1

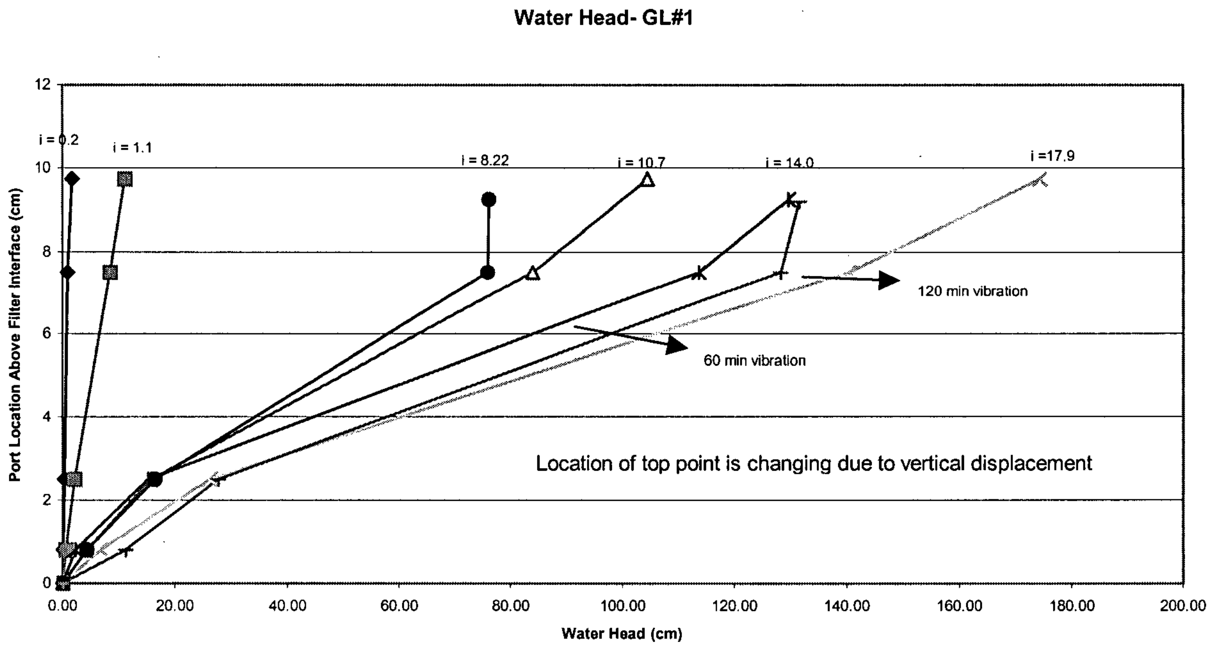

Figure 5.5 Water head test GL#1

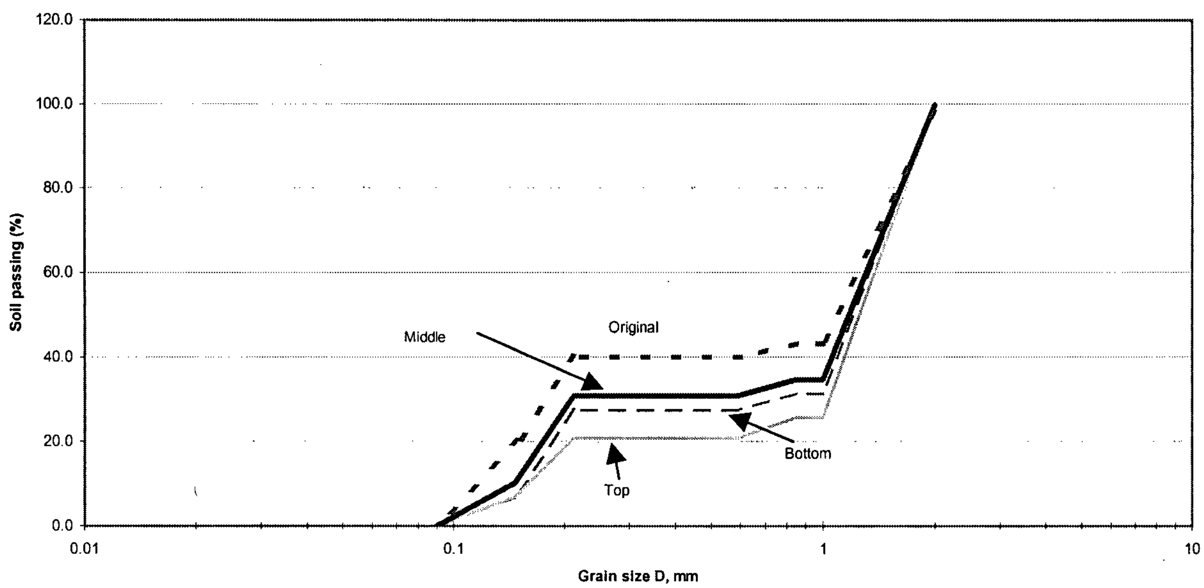

Figure 5.6 Sieve analyses GL#1

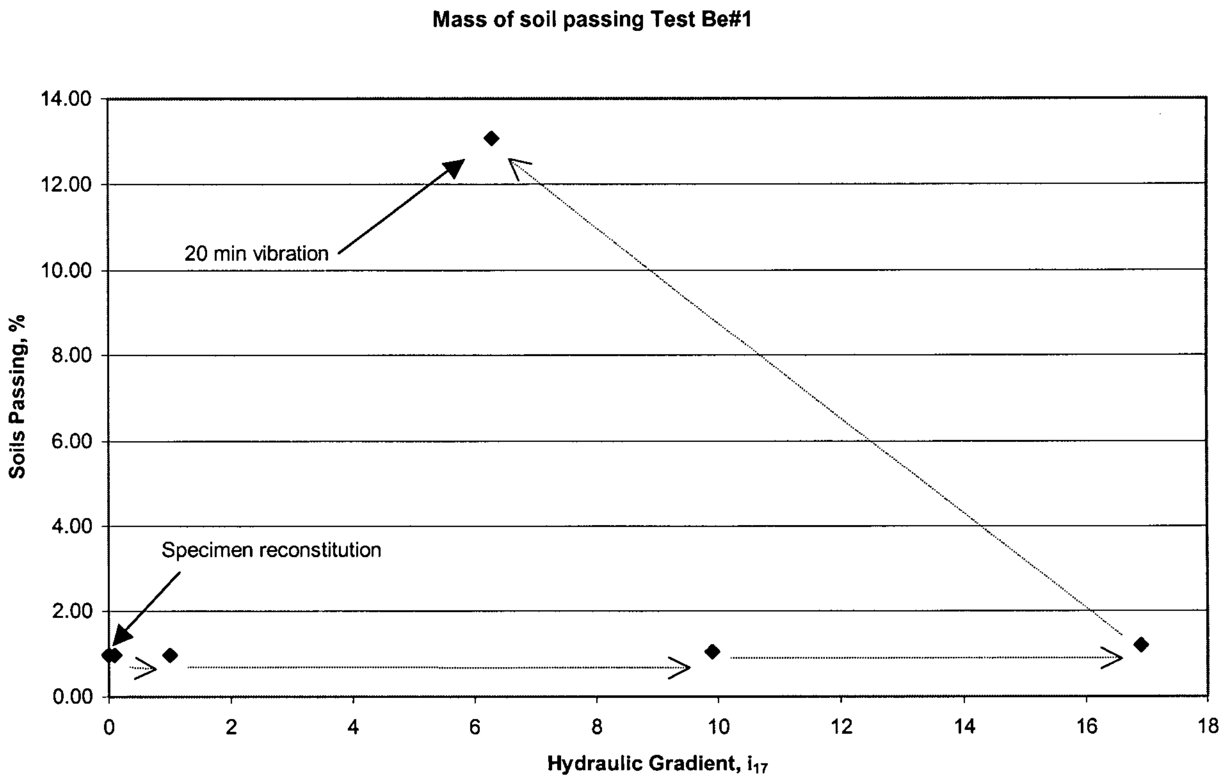

Figure 5.7. Soil passing test Be#1

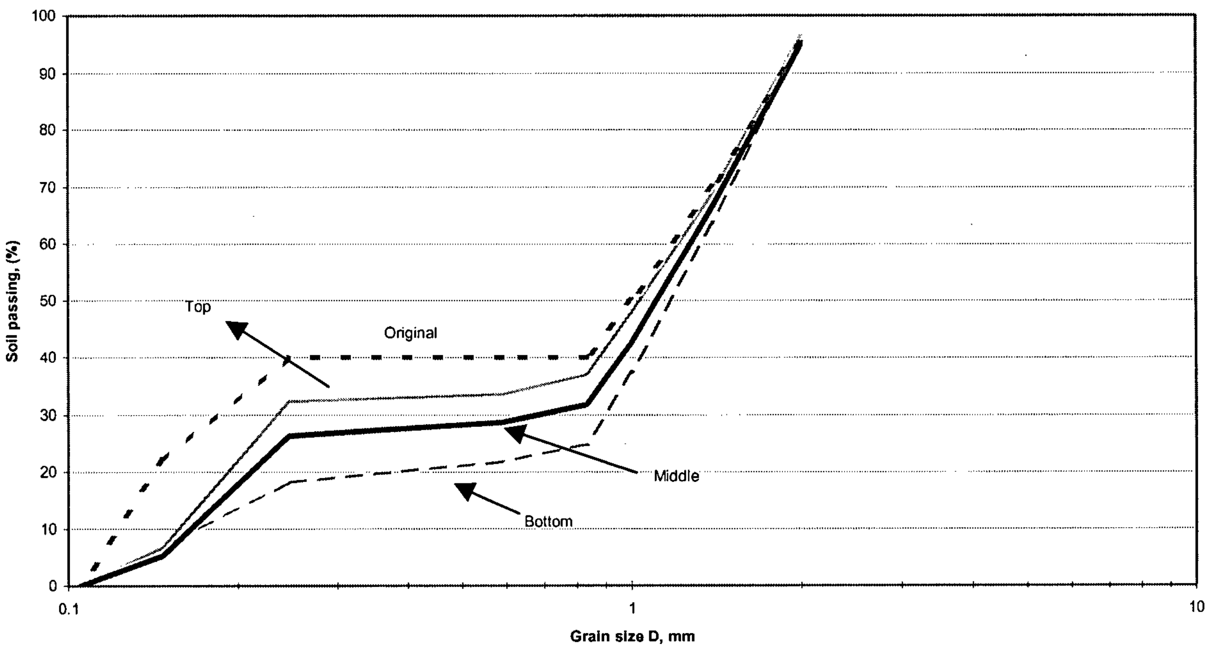

Figure 5.8. Sieve analysis Be#1

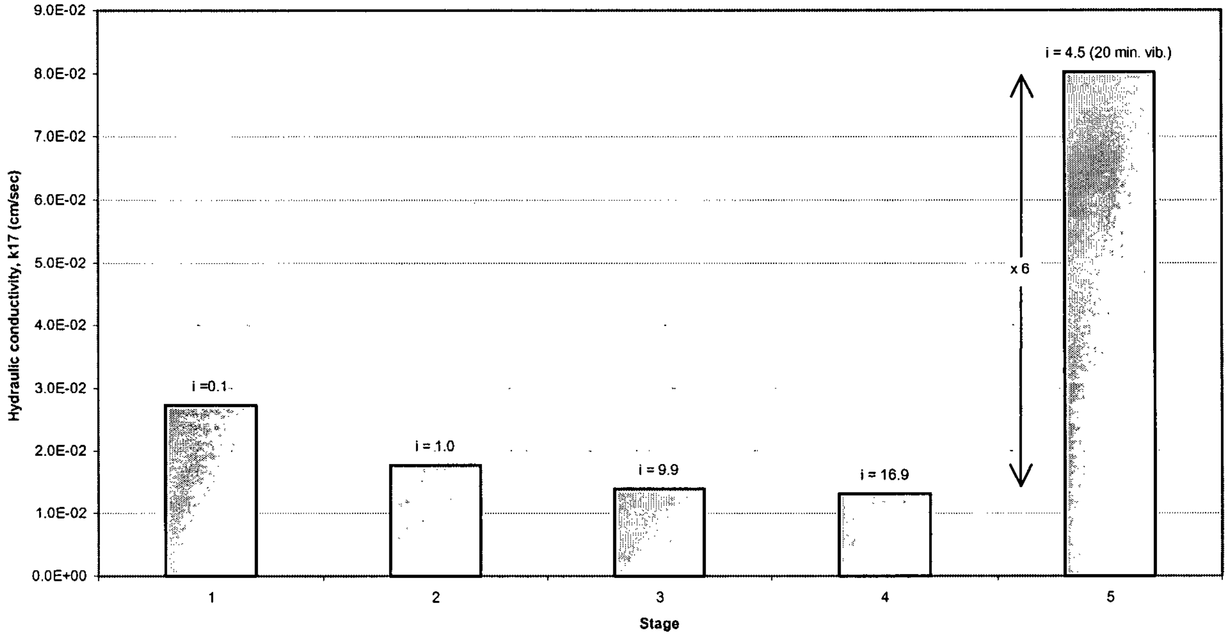

Figure 5.9. Hydraulic conductivity test Be#1

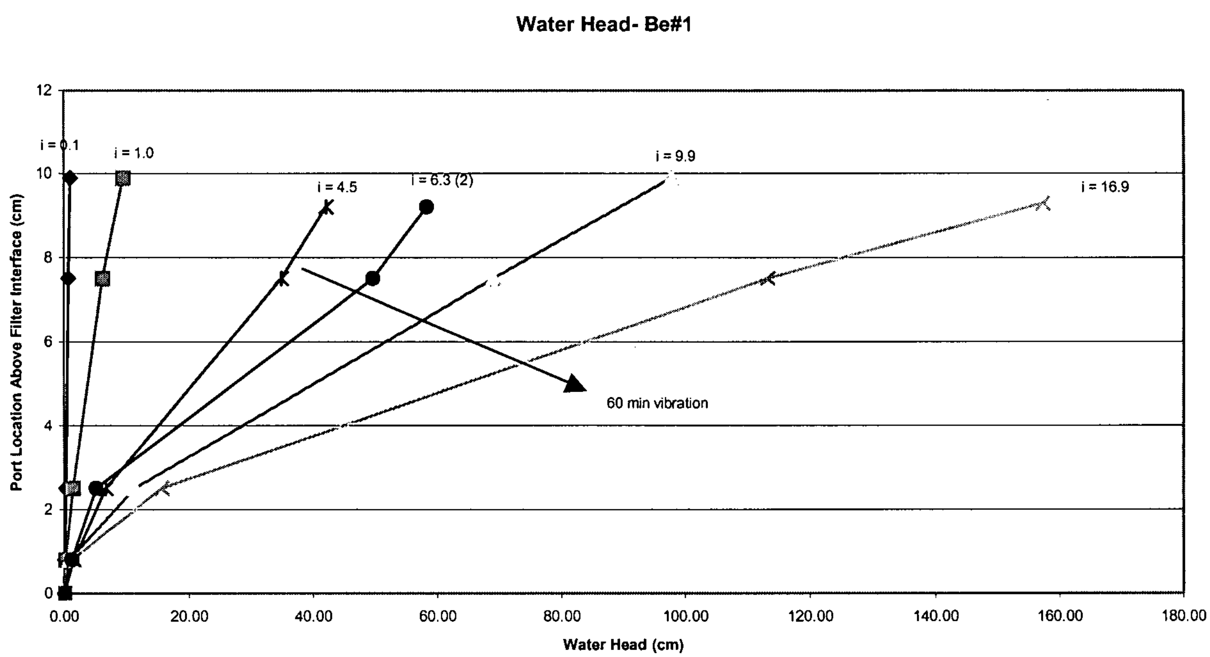

Figure 5.10. Water head test Be #1

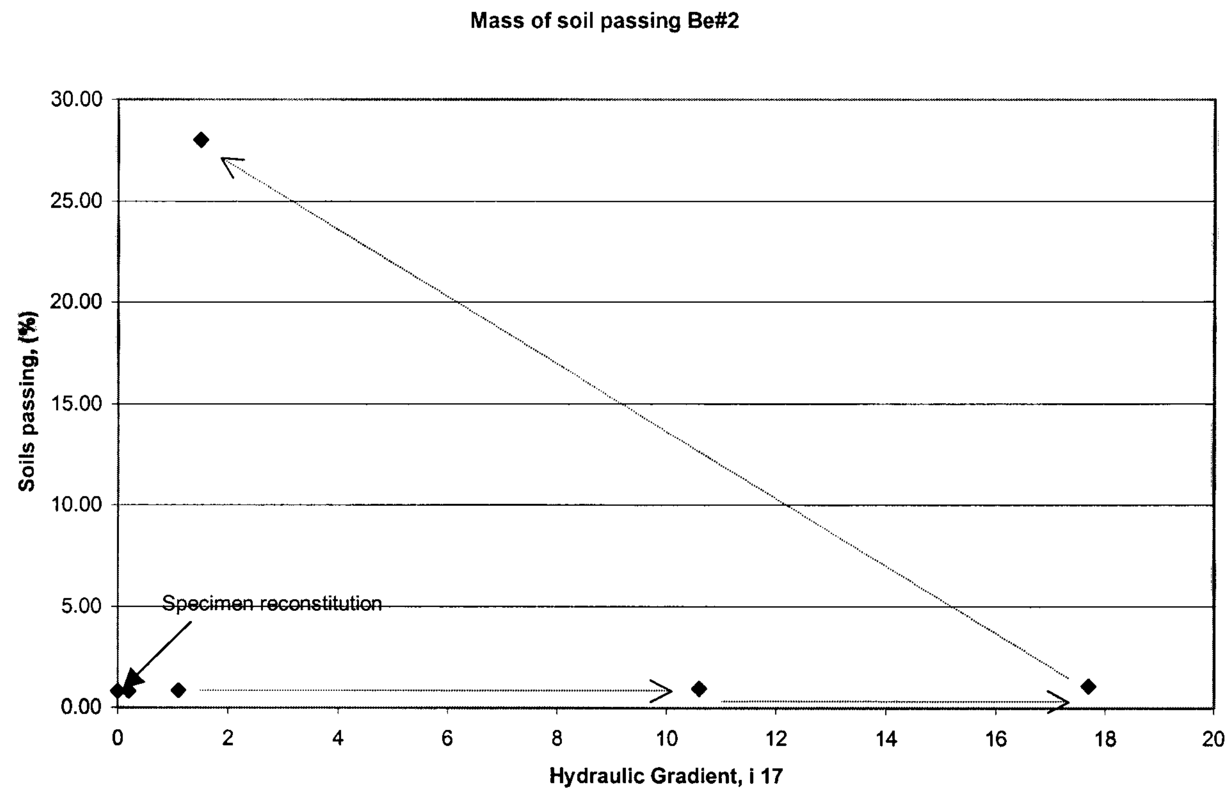

Figure 5.11. Soil passing test Be #2

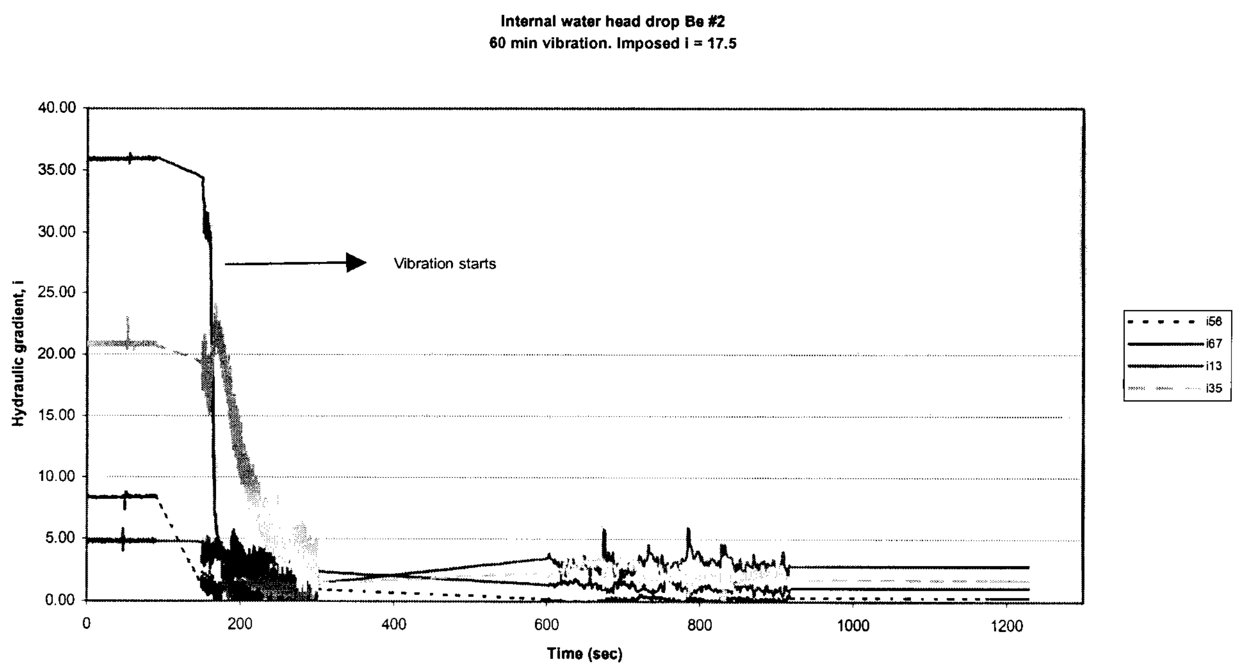

Figure 5.12. Internal variation of hydraulic gradient with elapsed time, during vibration.

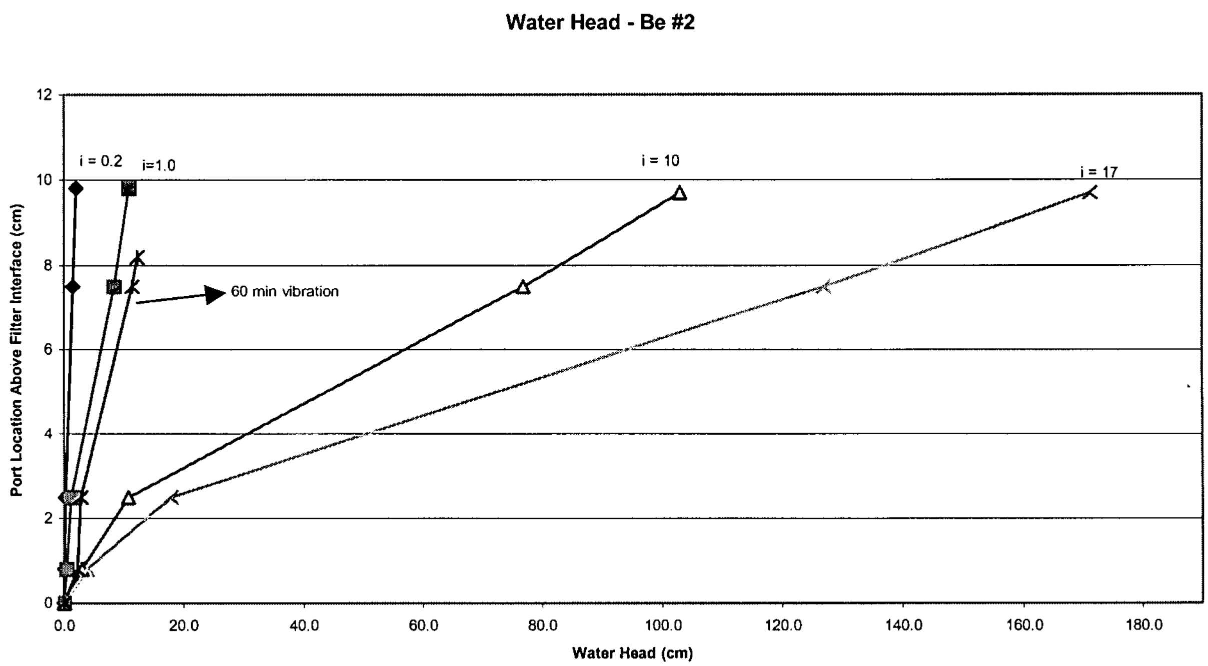

Figure 5.13. Water head test Be #2

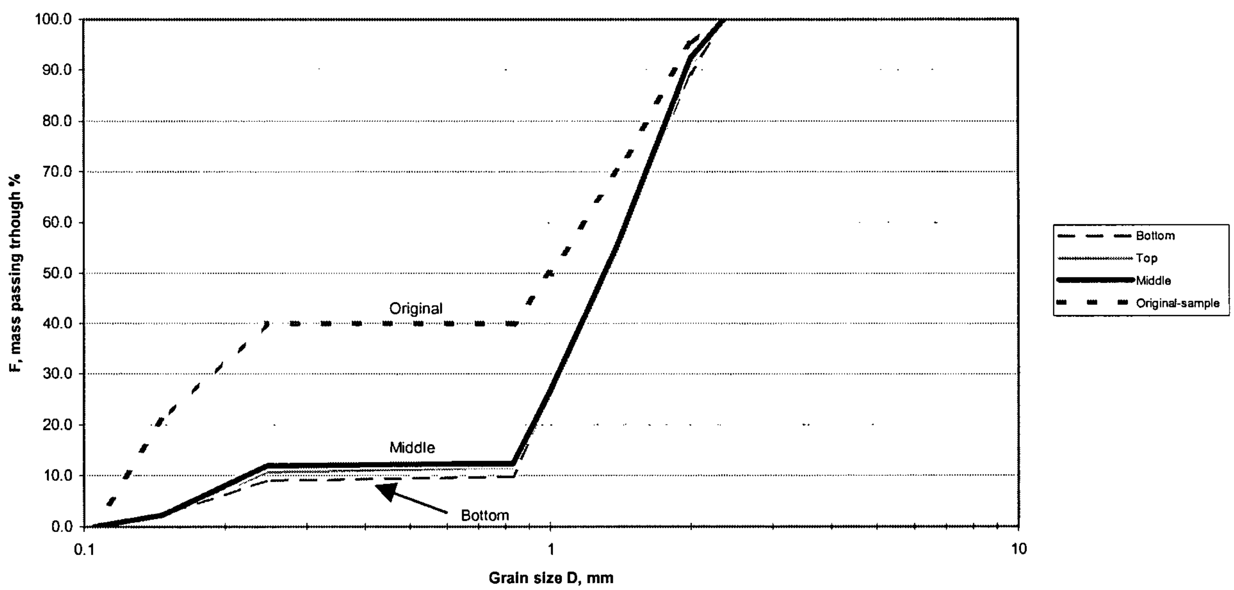

Figure 5.14 Sieve analysis test Be #2

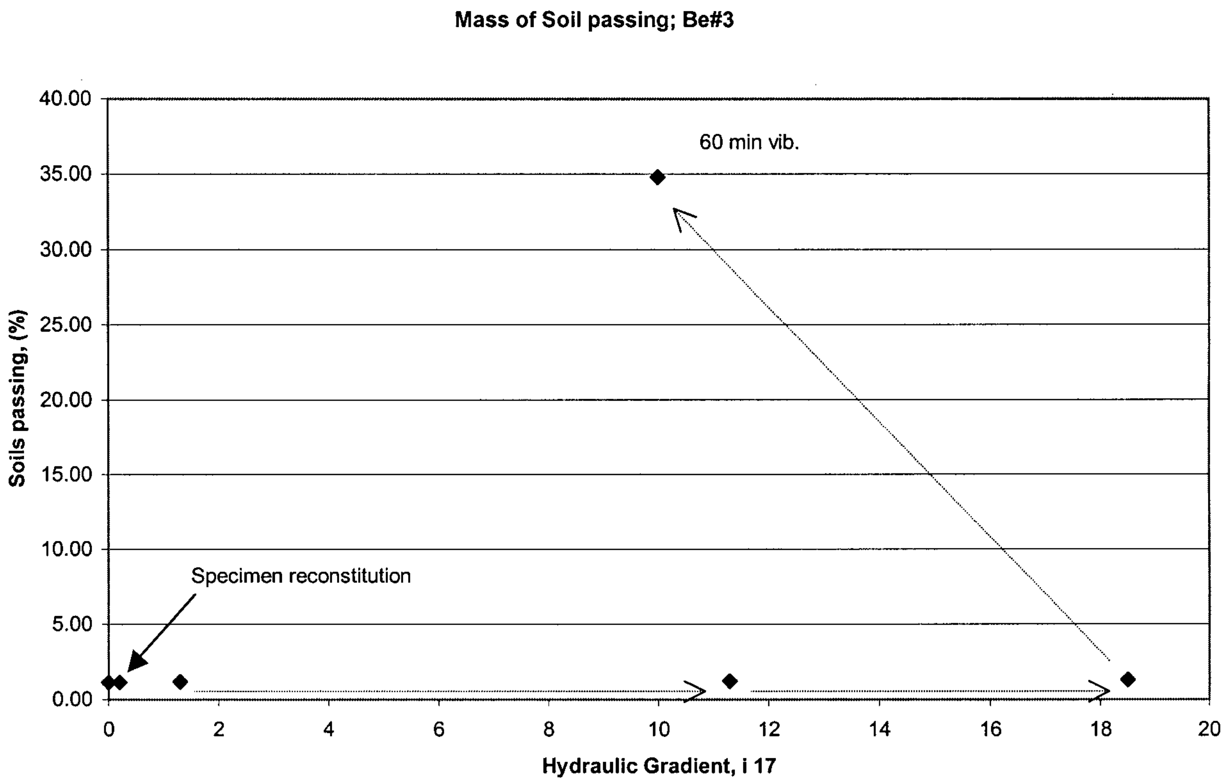

Figure 5.15. Soil passing test Be #3

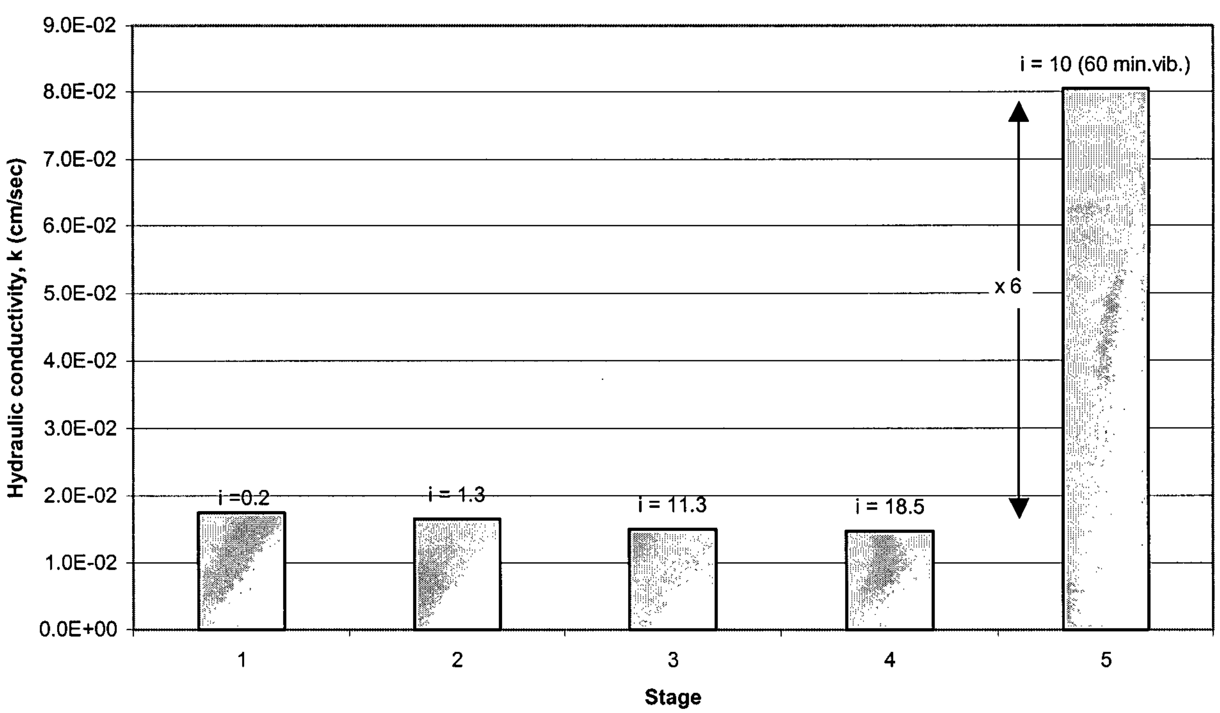

Figure 5.16. Hydraulic conductivity Test Be #3

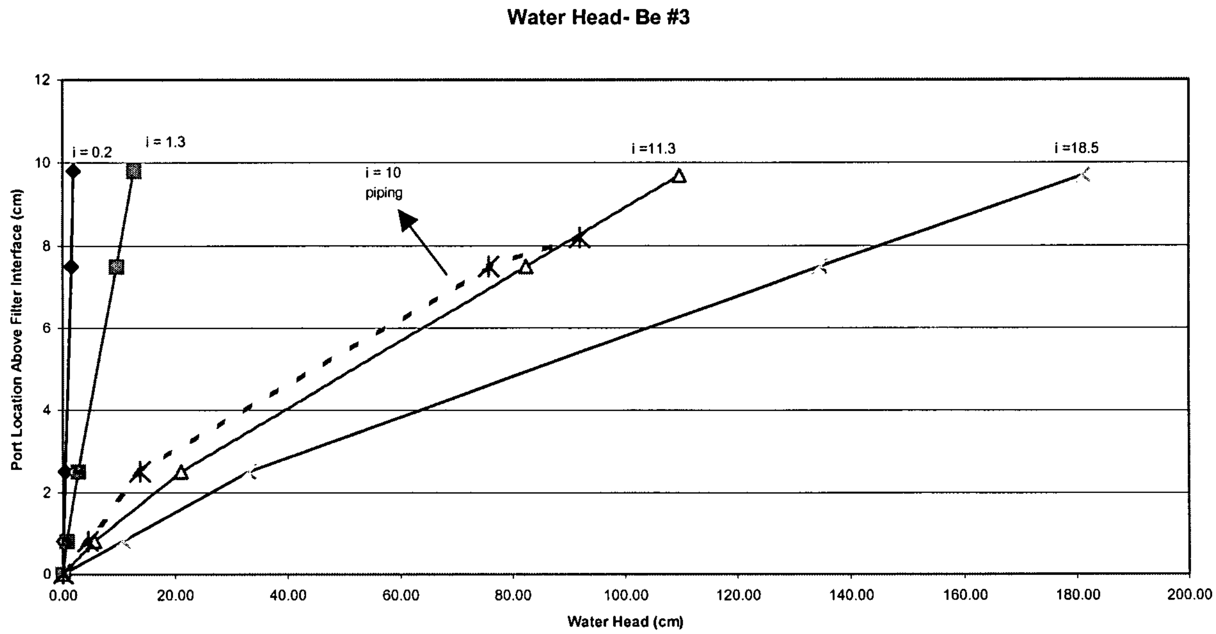

Figure 5.17. Water head test Be #3

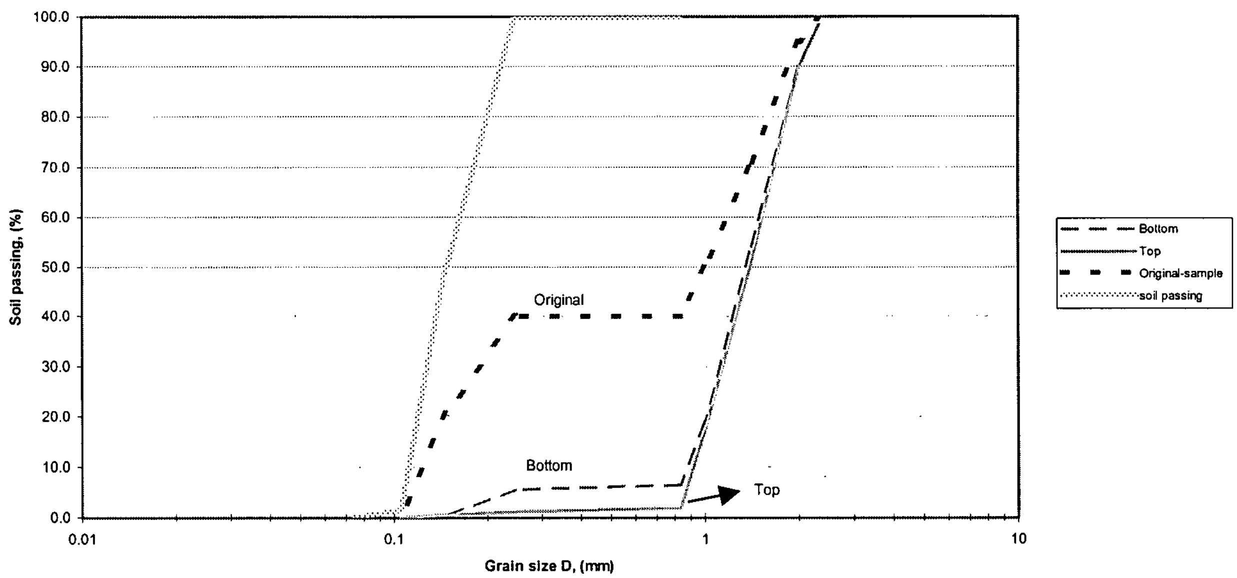

Figure 5.18. Sieve analysis test Be #3

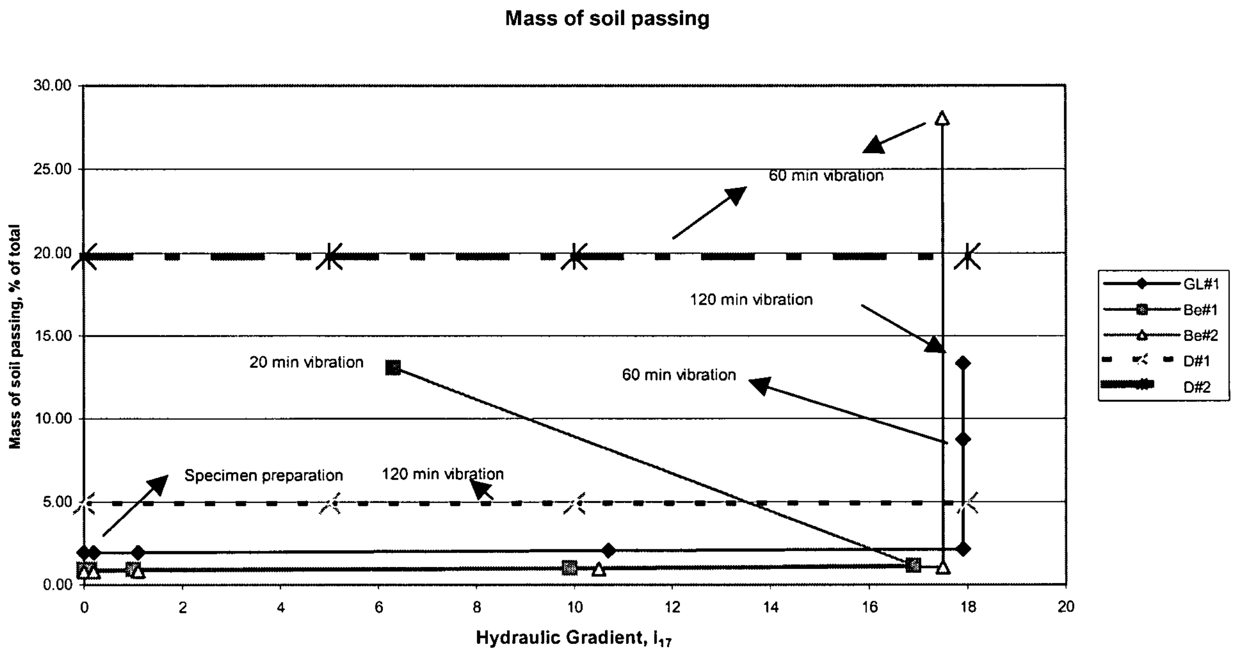

Figure 5.19. Comparison soil passing dry test.

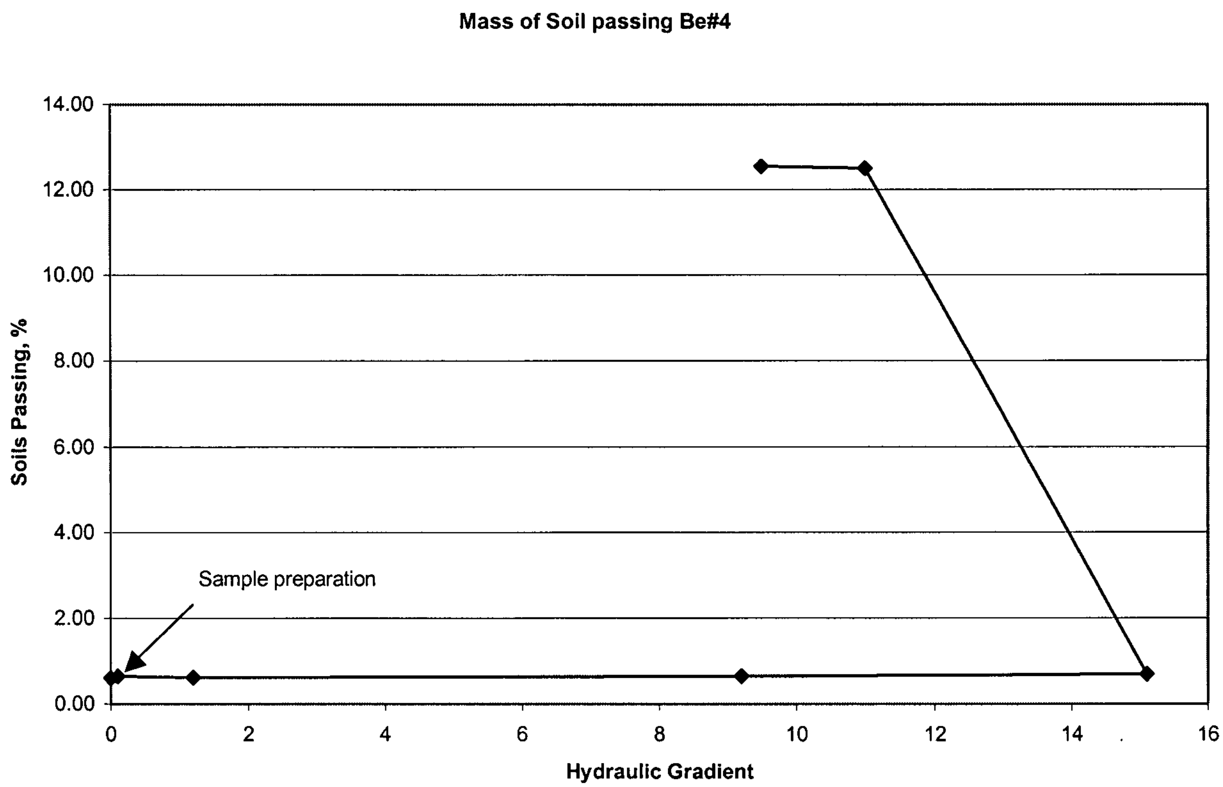

Figure 5.20. Soil passing test Be #4

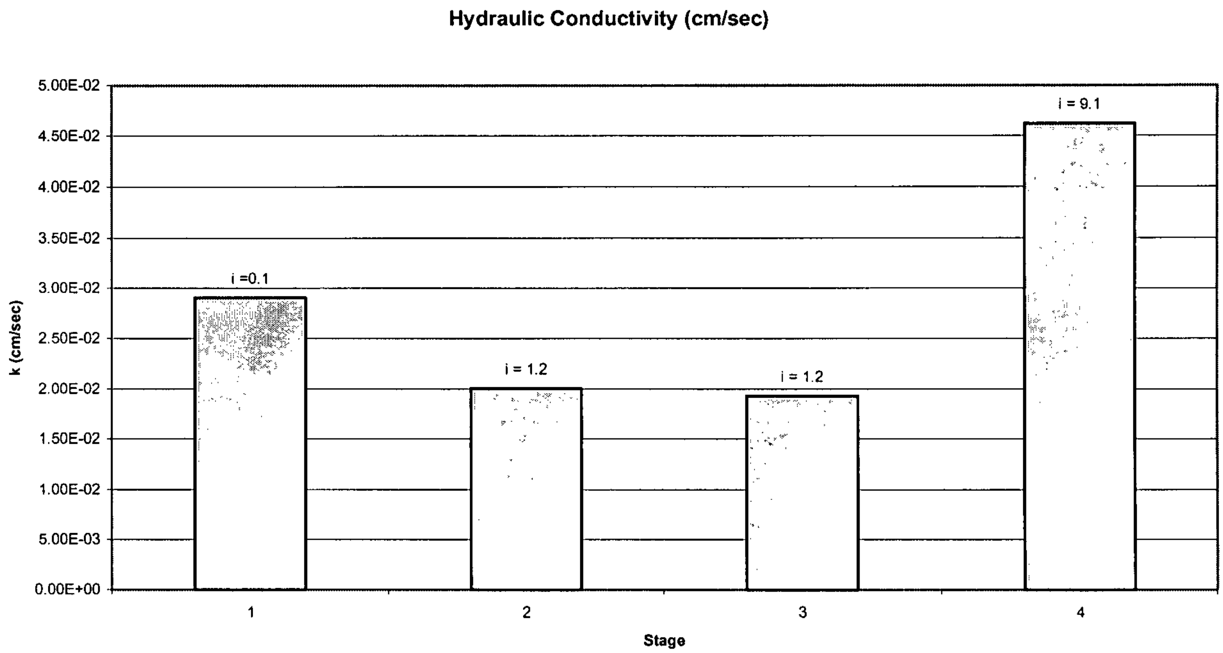

Figure 5.21 Hydraulic conductivity test Be#4

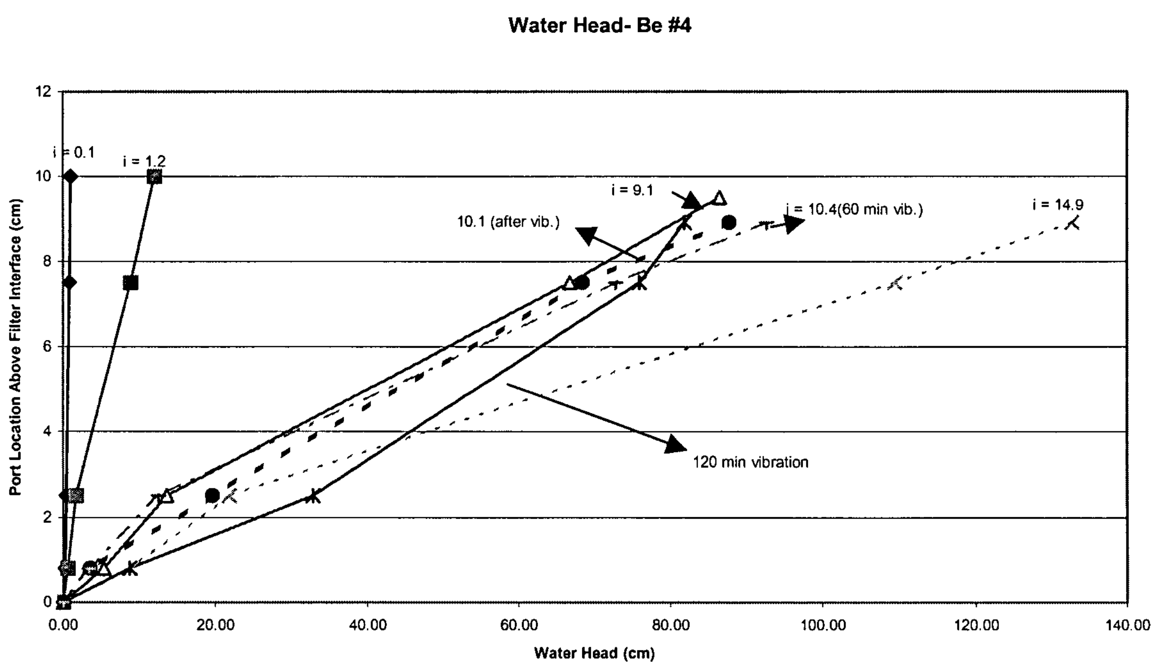

Figure 5.22. Water head test Be #4

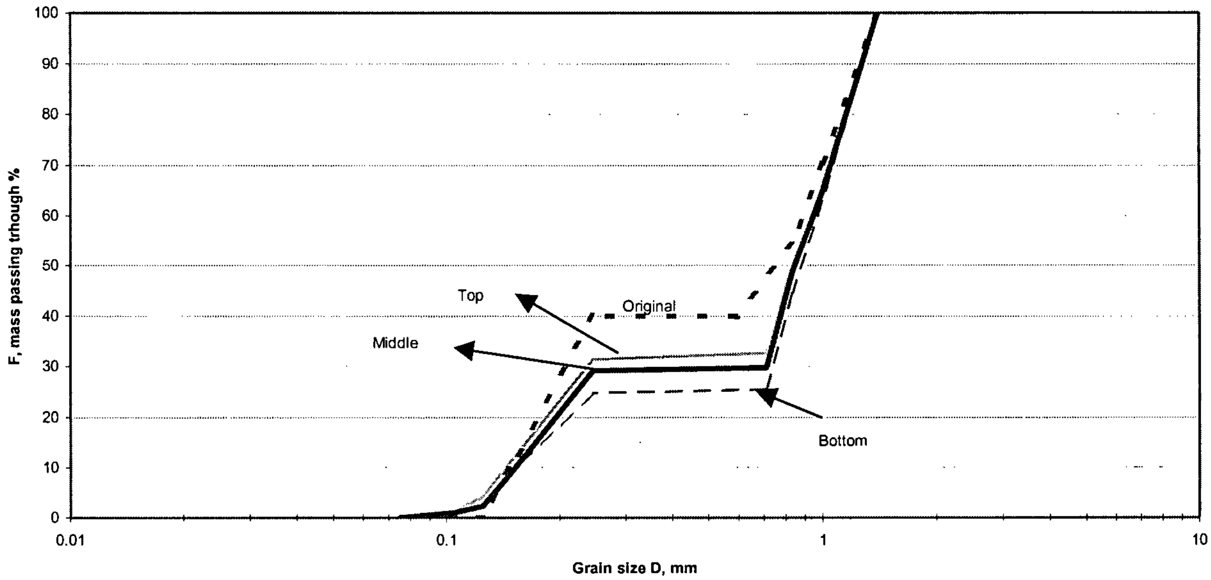

Figure 5.23. Sieve analysis test Be #4

Water Head- Be #5

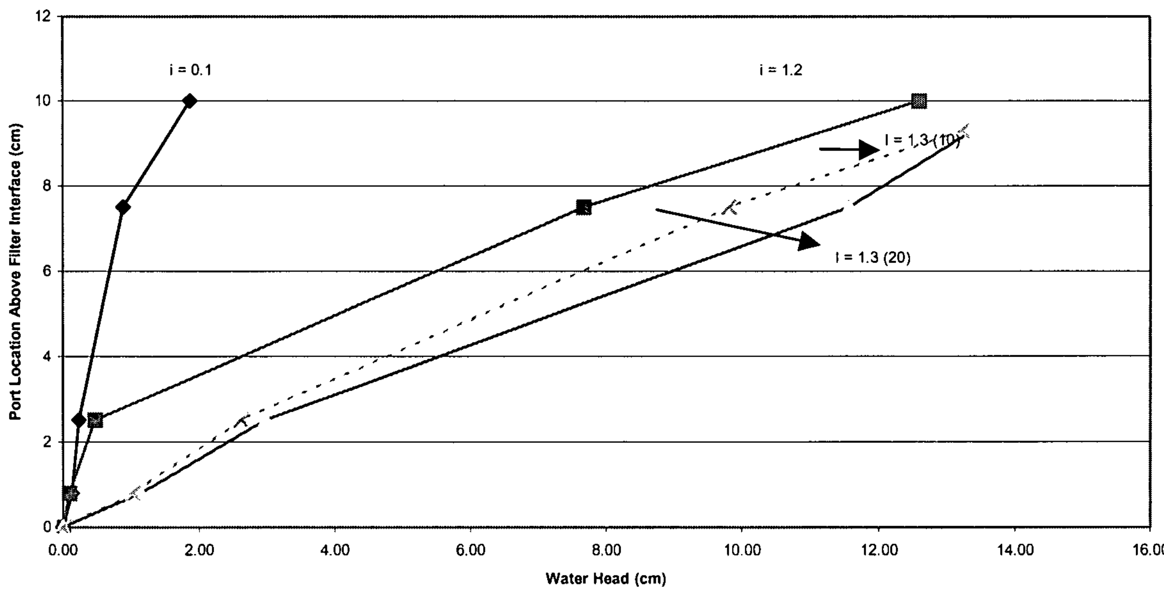

Figure 5.24. Water head test Be #5

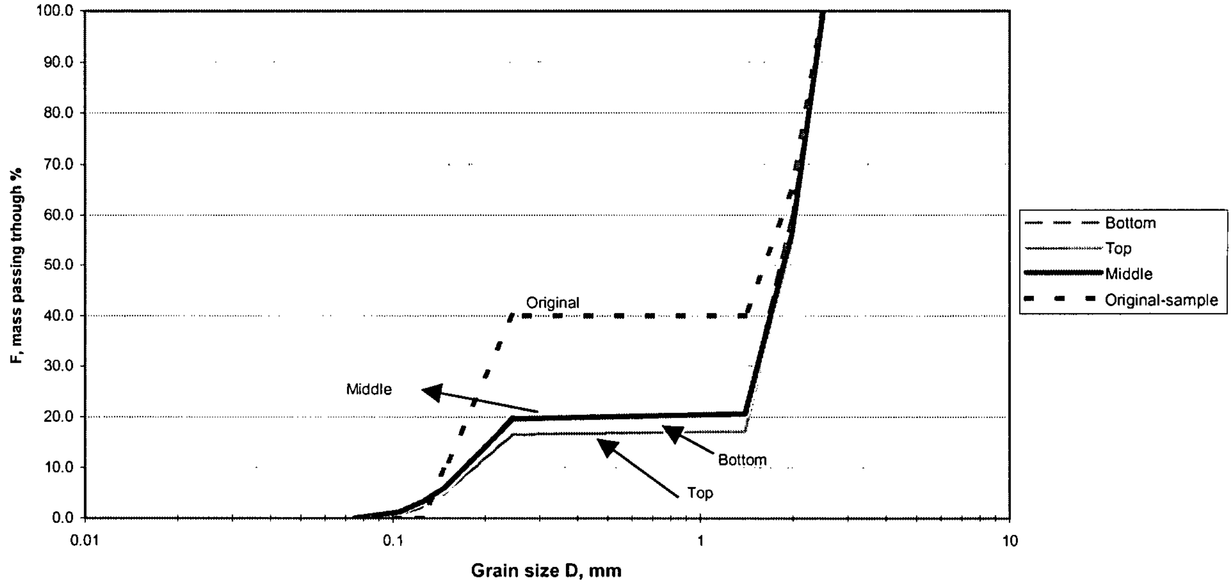

Figure 5.25. Sieve analysis test Be#5

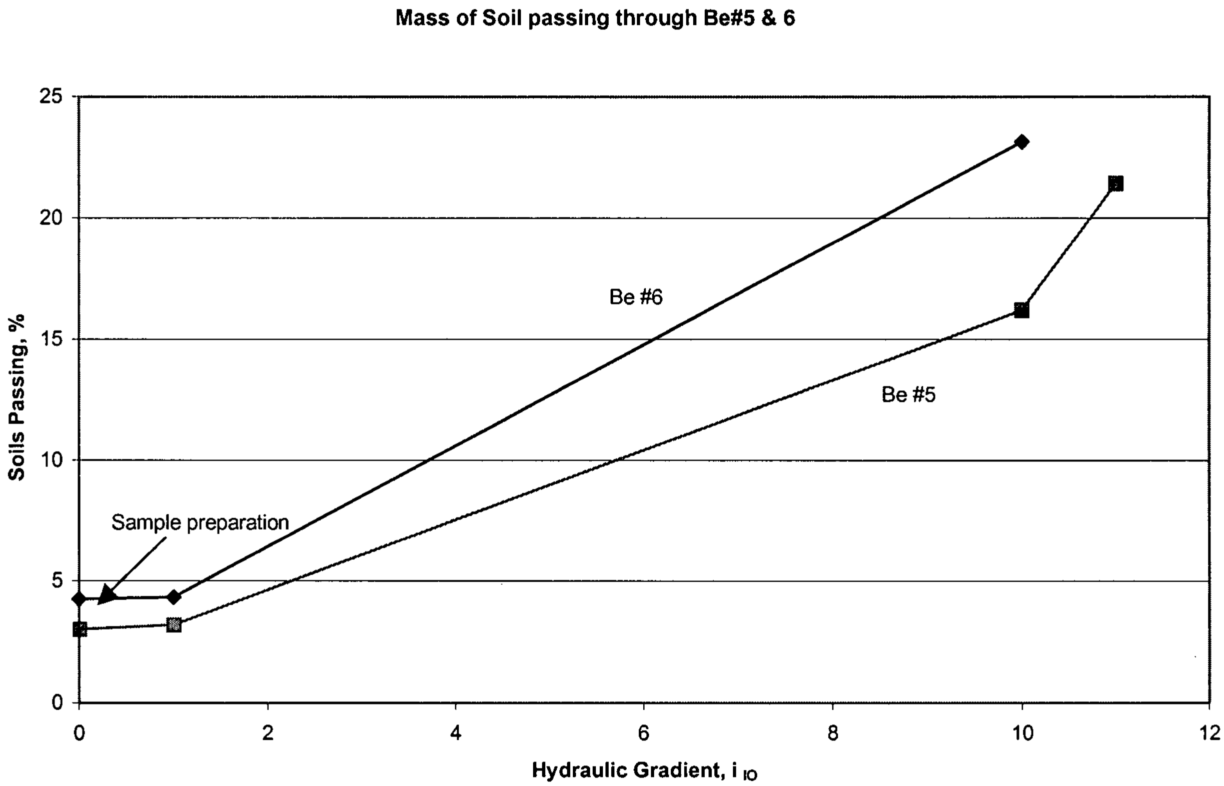

Figure 5.26. Soil passing test Be #5 and Be #6.

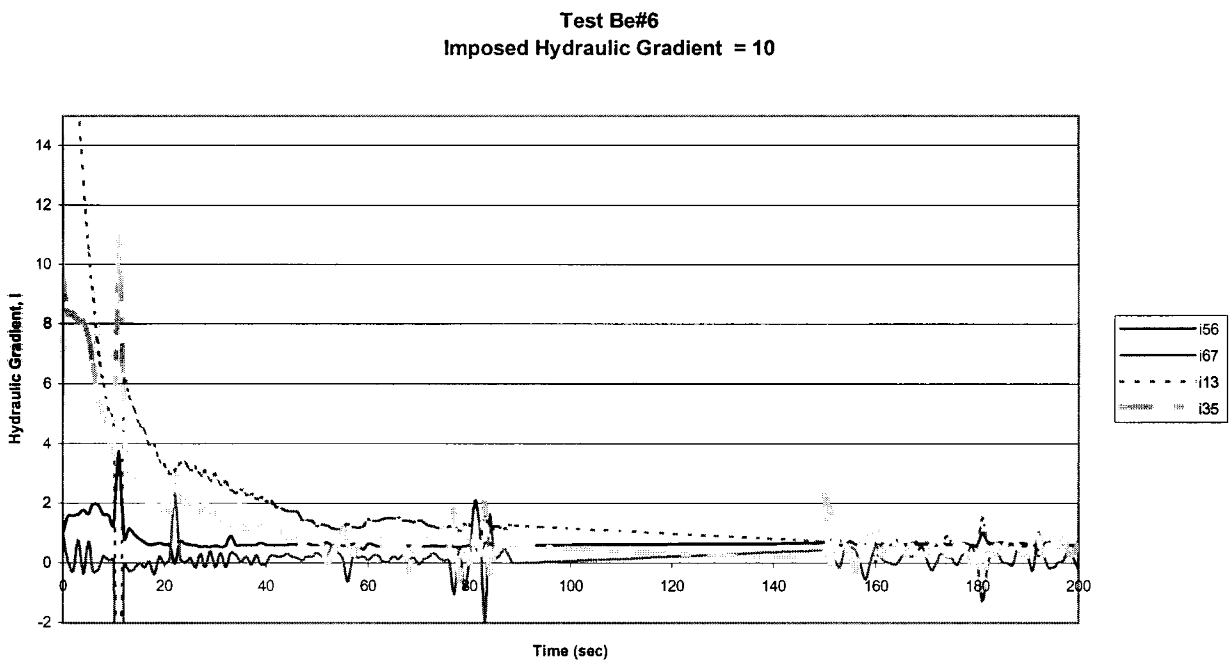

Figure 5.27. Internal variation of hydraulic gradient with elapsed time, test Be#6

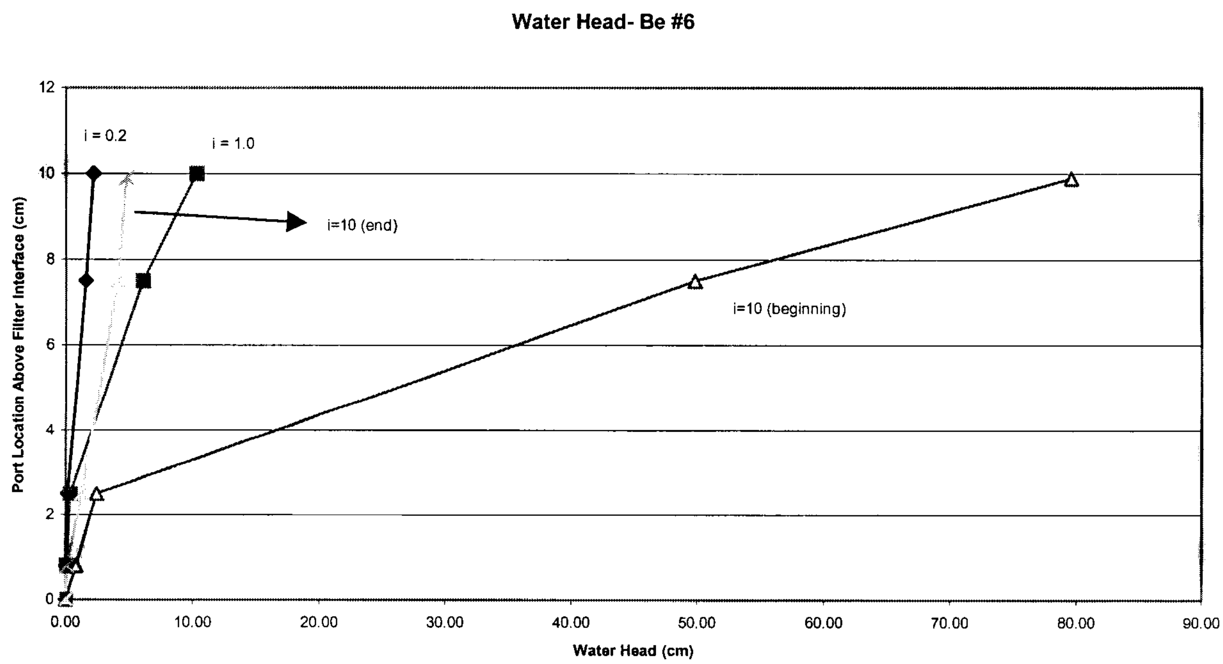

Figure 5.28. Water head test Be #6

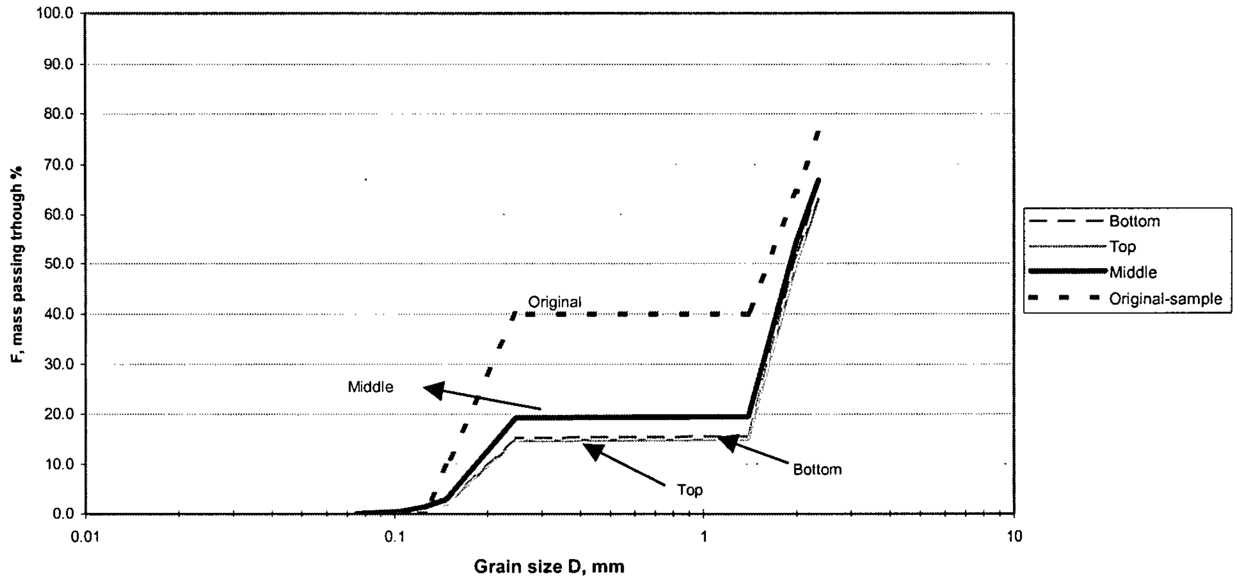

Figure 5.29. Sieve analysis test Be #6

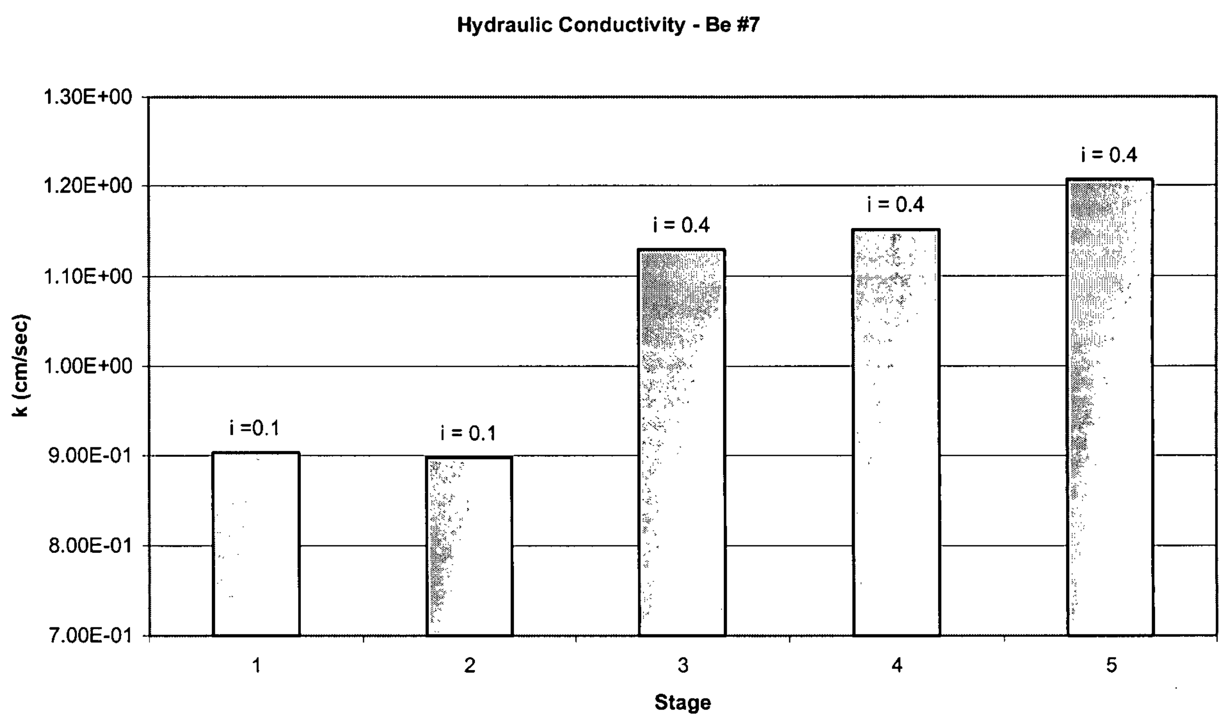

Figure 5.30. Hydraulic conductivity test Be#7

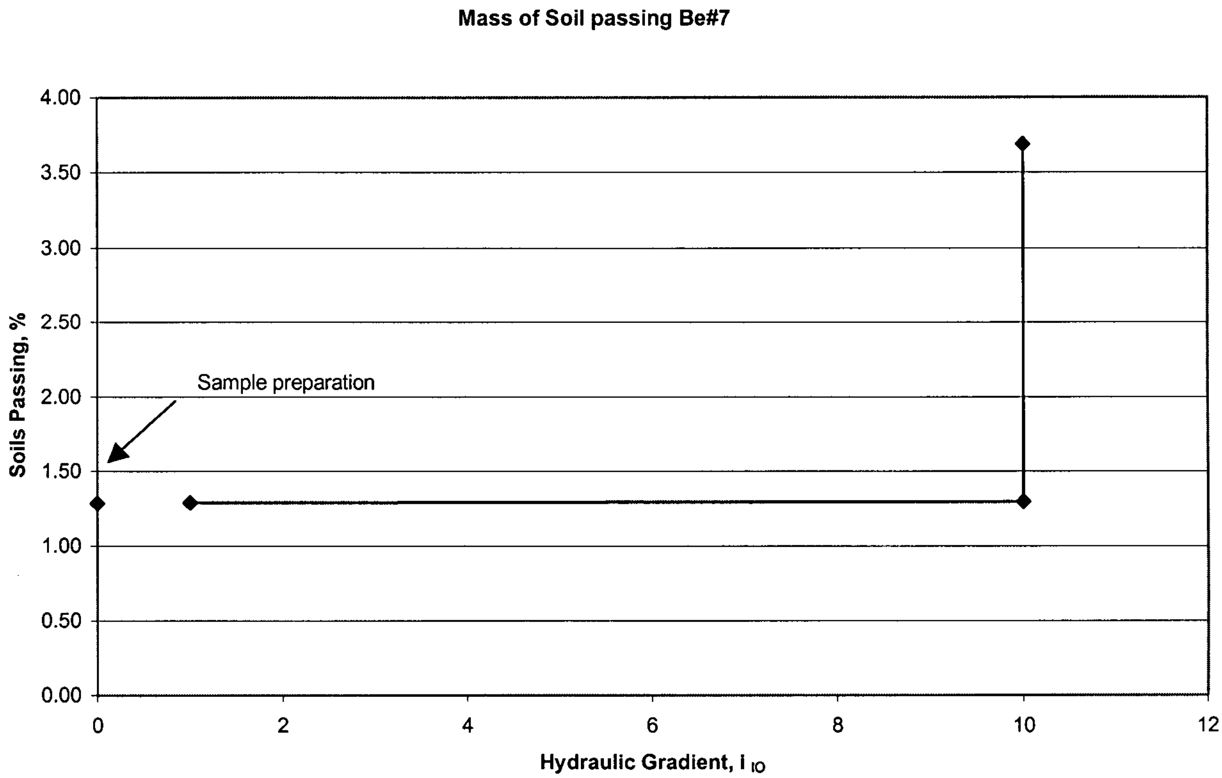

Figure 5.31 Soil passing test Be#7

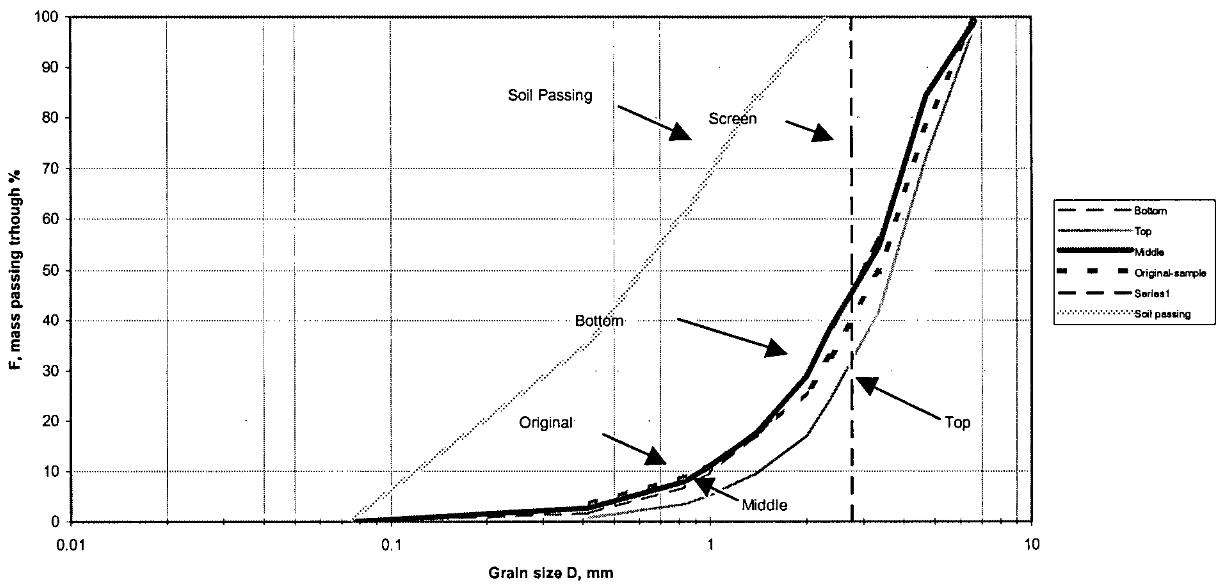

Figure 5.32 Sieve analysis test Be#7

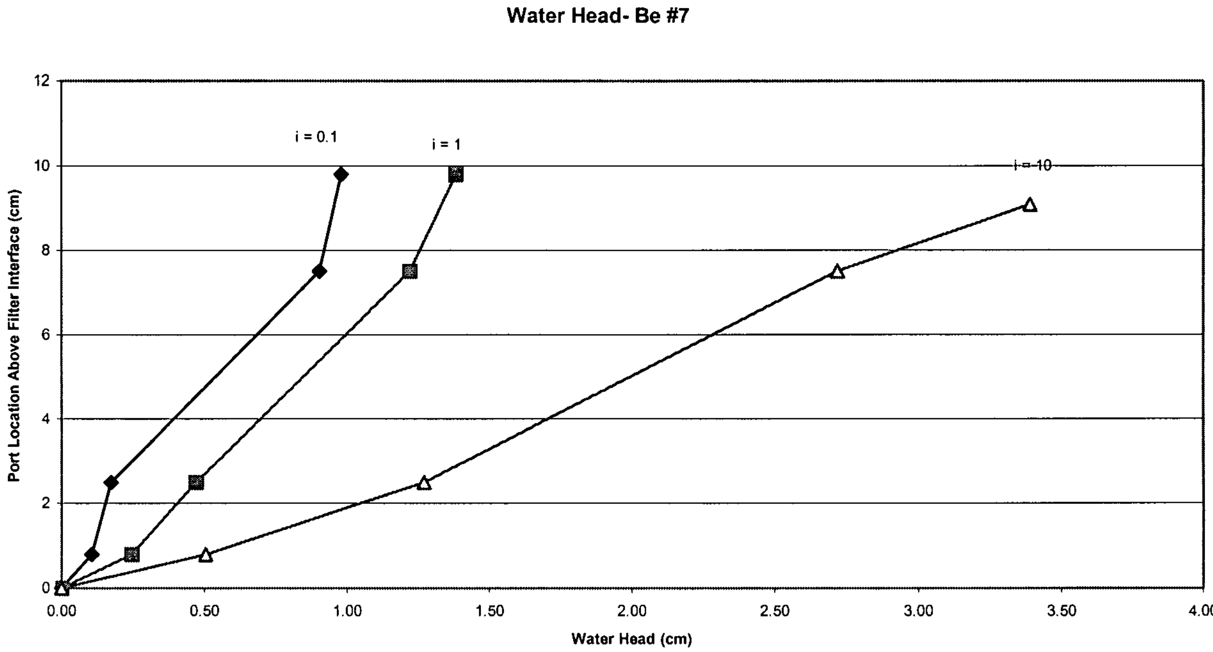

Figure 5.33 Water head test Be#7

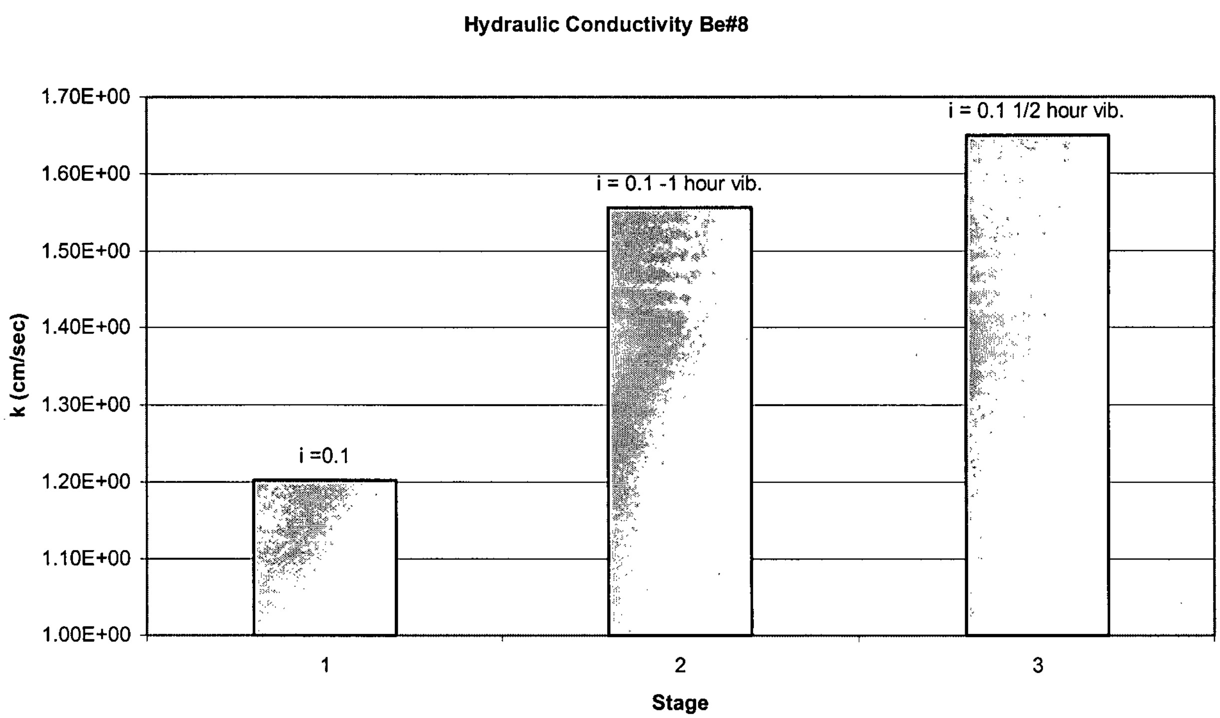

Figure 5.34. Hydraulic conductivity test Be#8

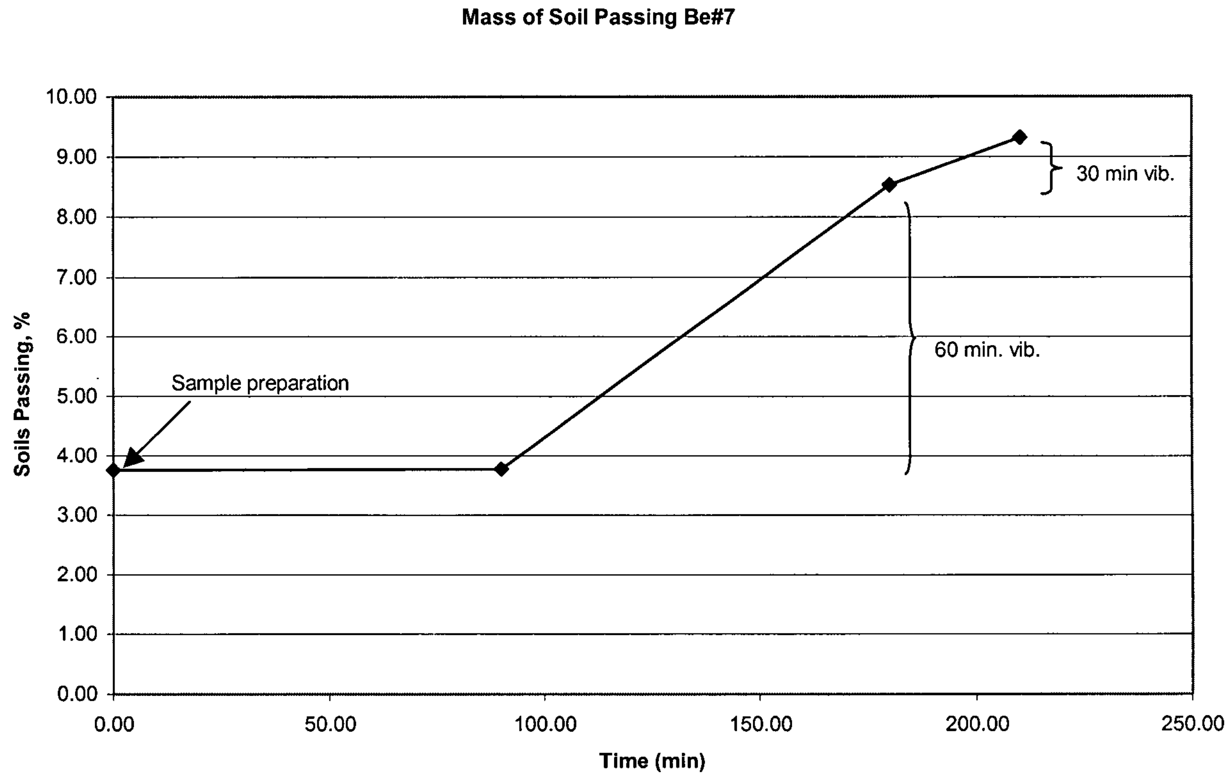

Figure 5.35. Soil passing test Be#8

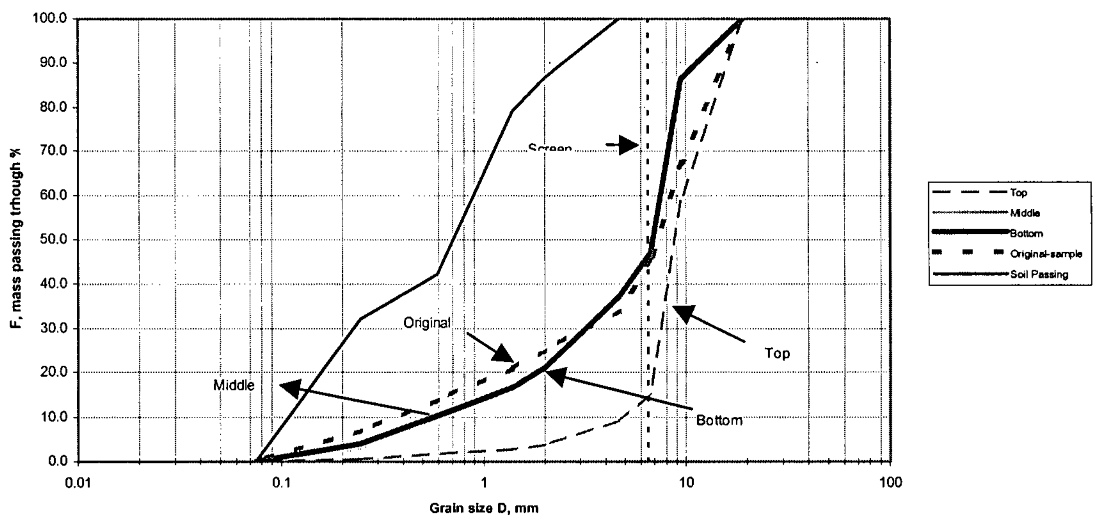

Figure 5.36. Sieve analysis test Be#8

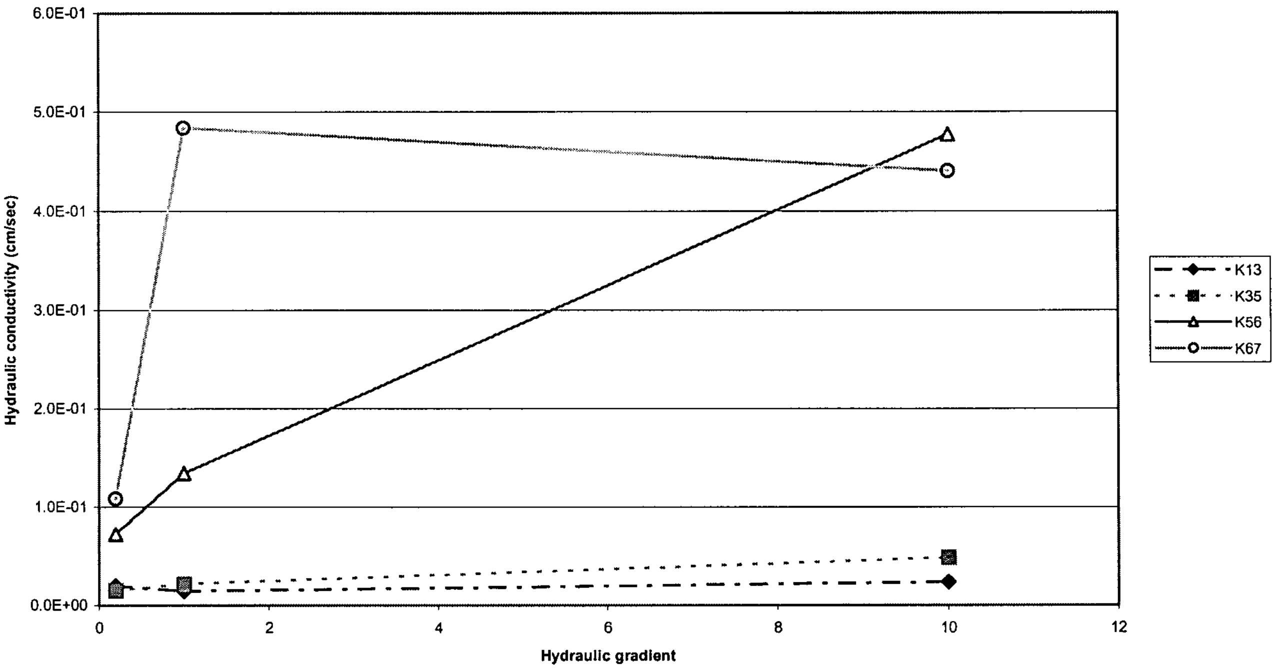

Figure 6.1. Variation pf hydraulic conductivity (test Be#6)

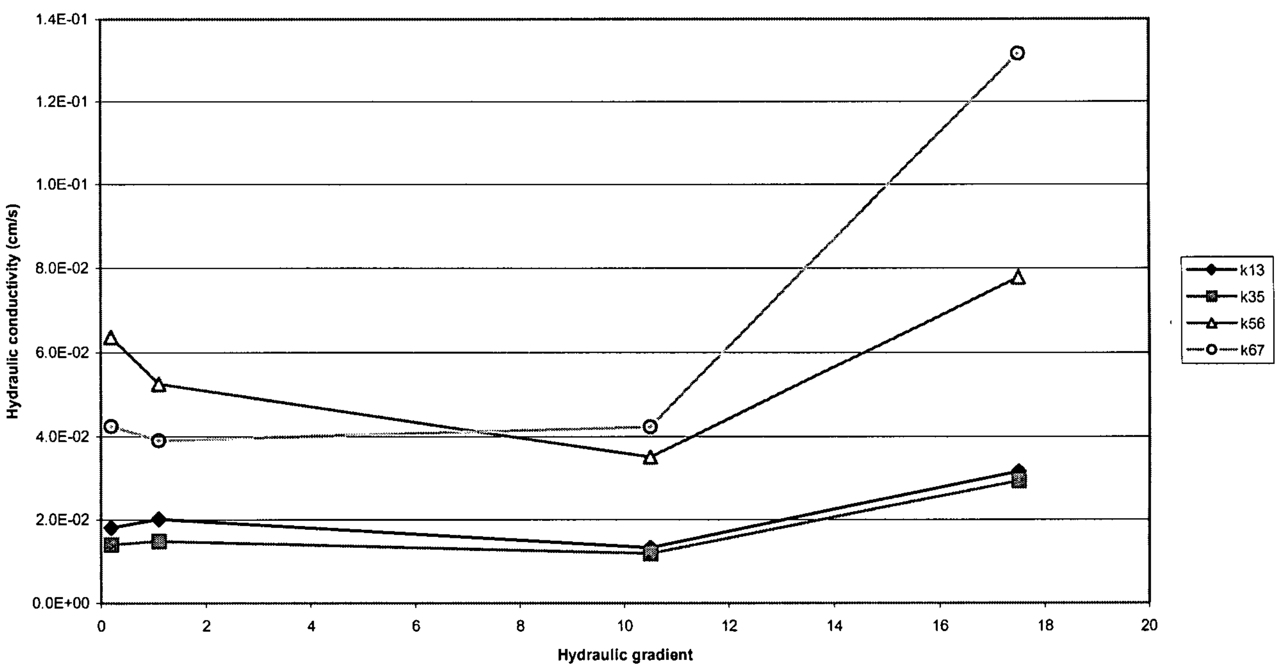

Figure 6.2. Variation pf hydraulic conductivity (test Be#1)

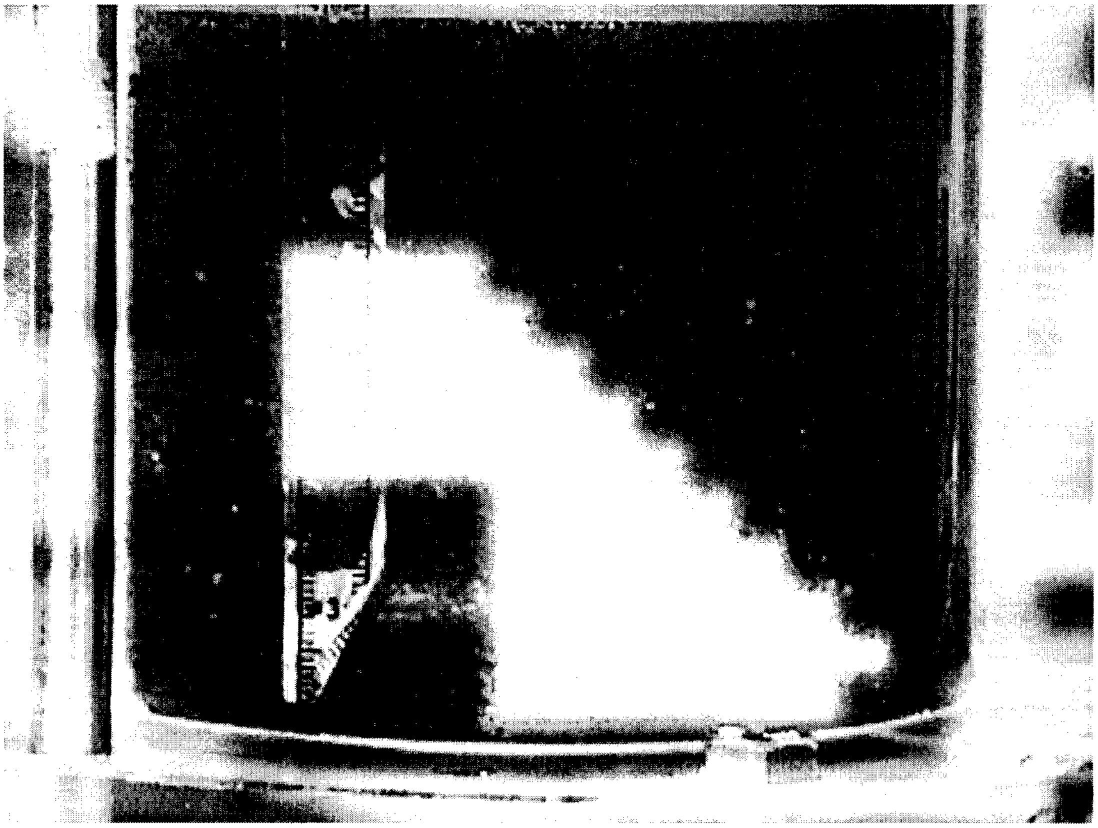

Figure 6.3. Test Be#1 (after specimen reconstitution)

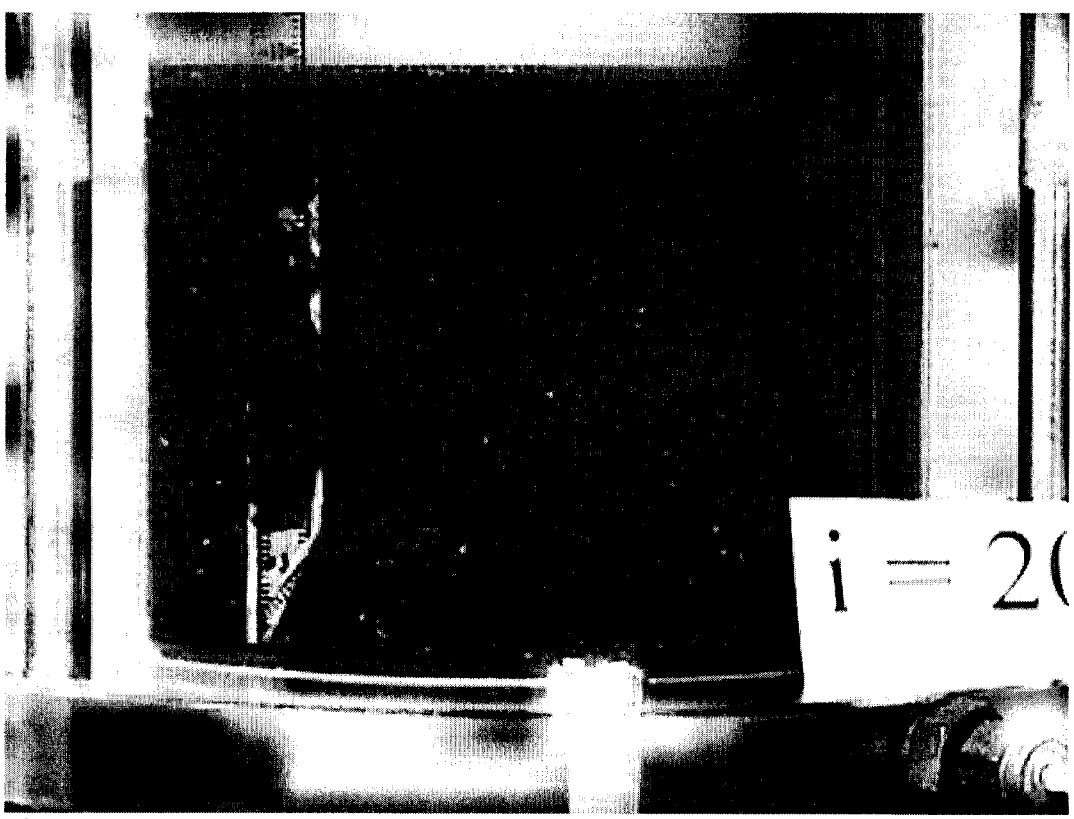

Figure 6.4. Test Be#1 (at i = 16.9, no vibration)

Figure 6.5. Test Be#1(during vibration)

Figure 6.6. Test Be#1 (after vibration)

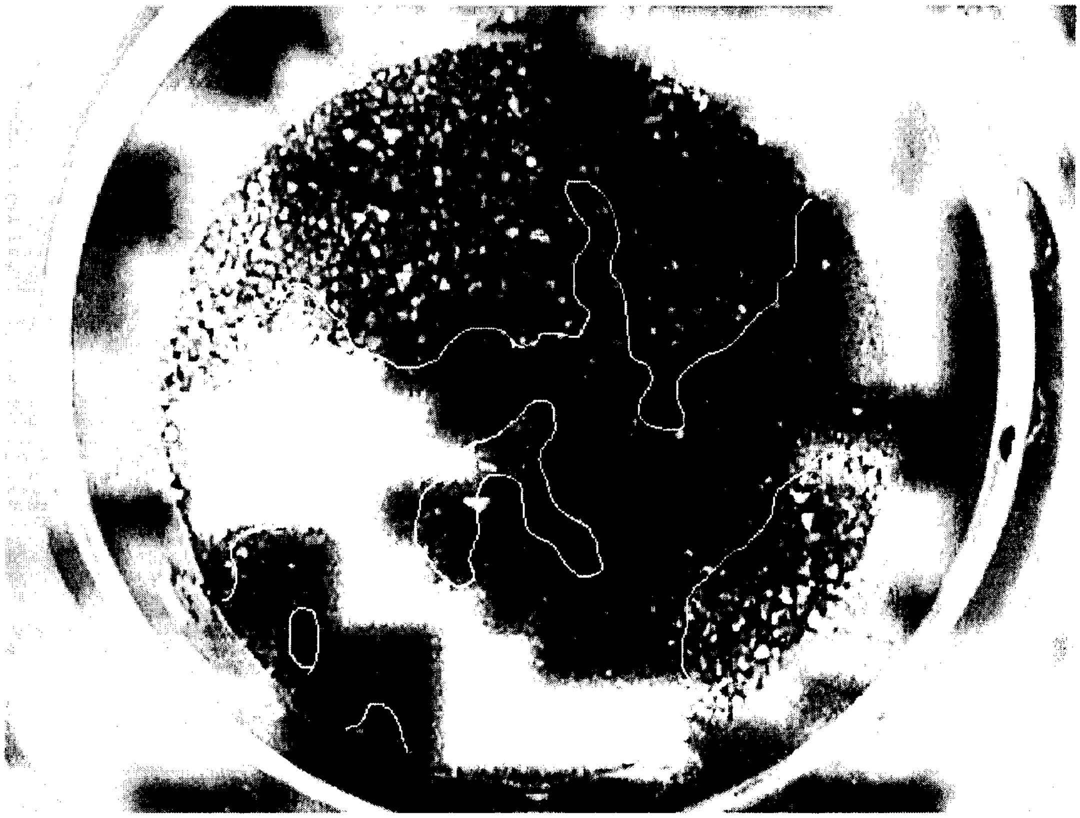

Figure 6.7. Test Be#1 (plan view of top of specimen)

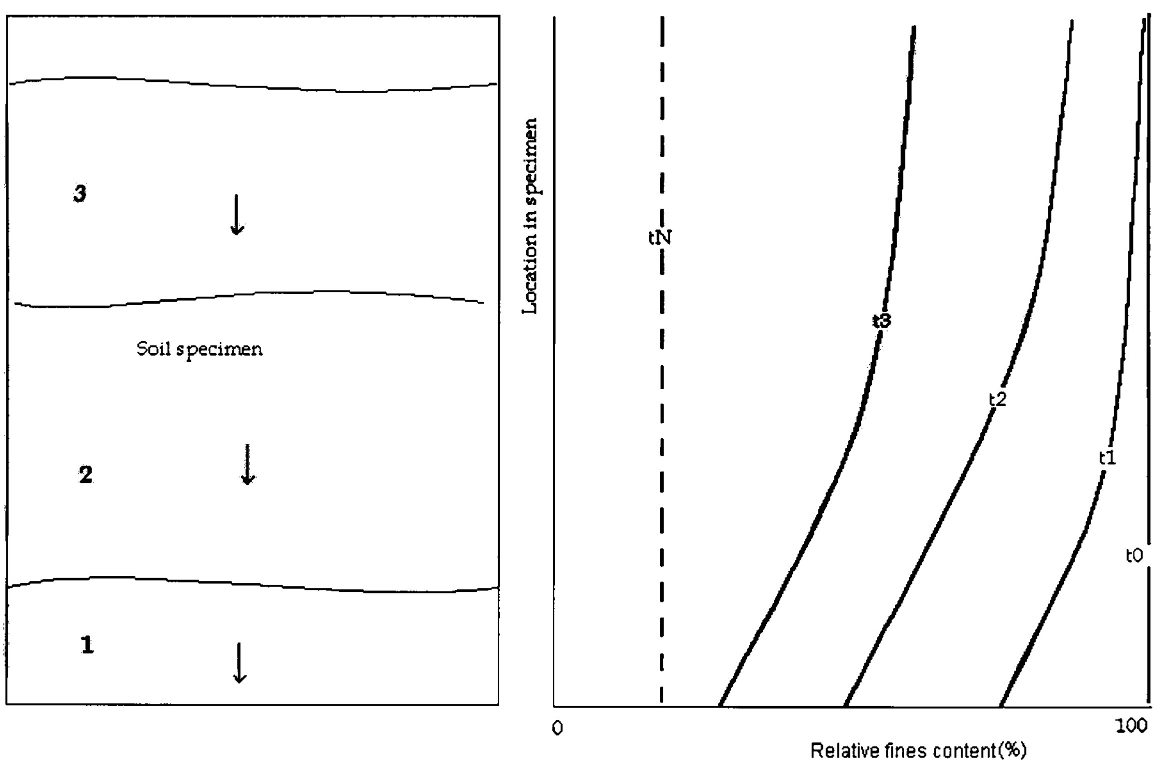

Figure 6.8. Expected mode of soil migration

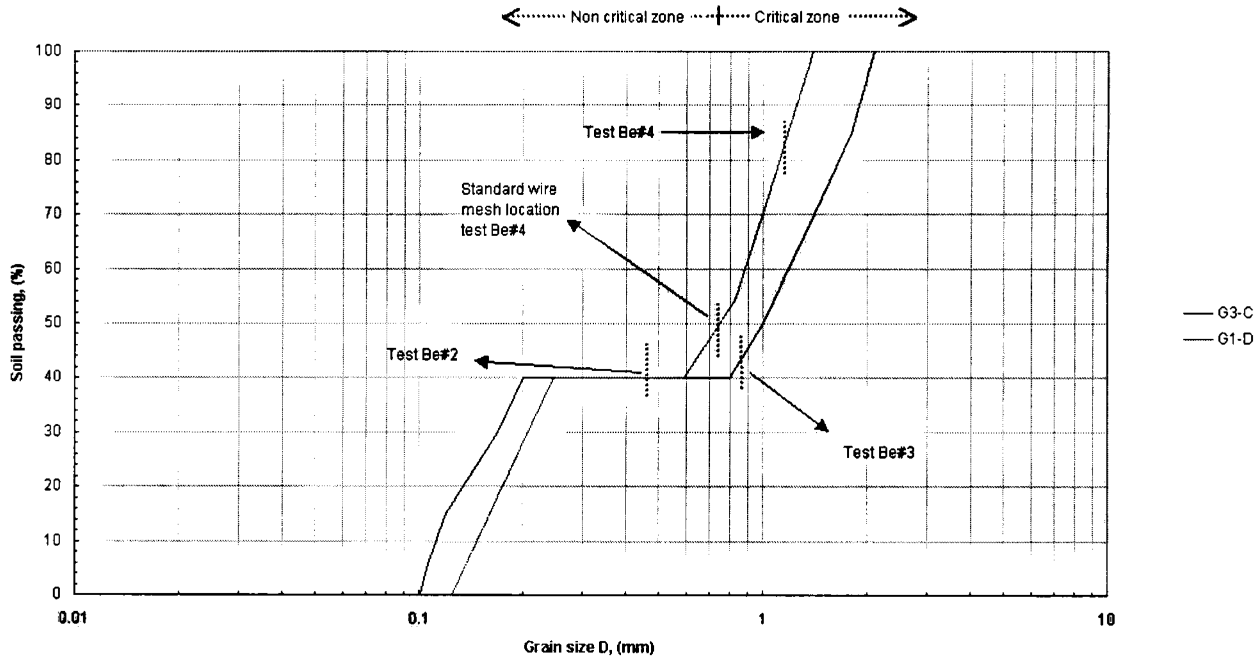

Figure 6.9. Opening wire-mesh size.

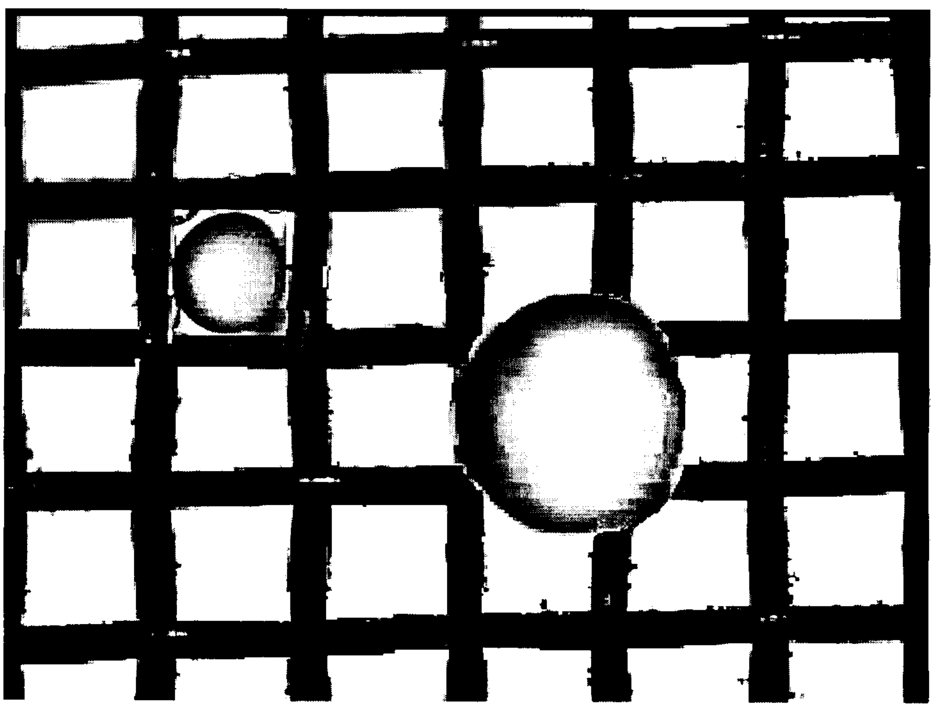

Figure 6.10. Obstruction on wire mesh. Wire-mesh 0.86mm "A". Wire-mesh 0.45mm "B".

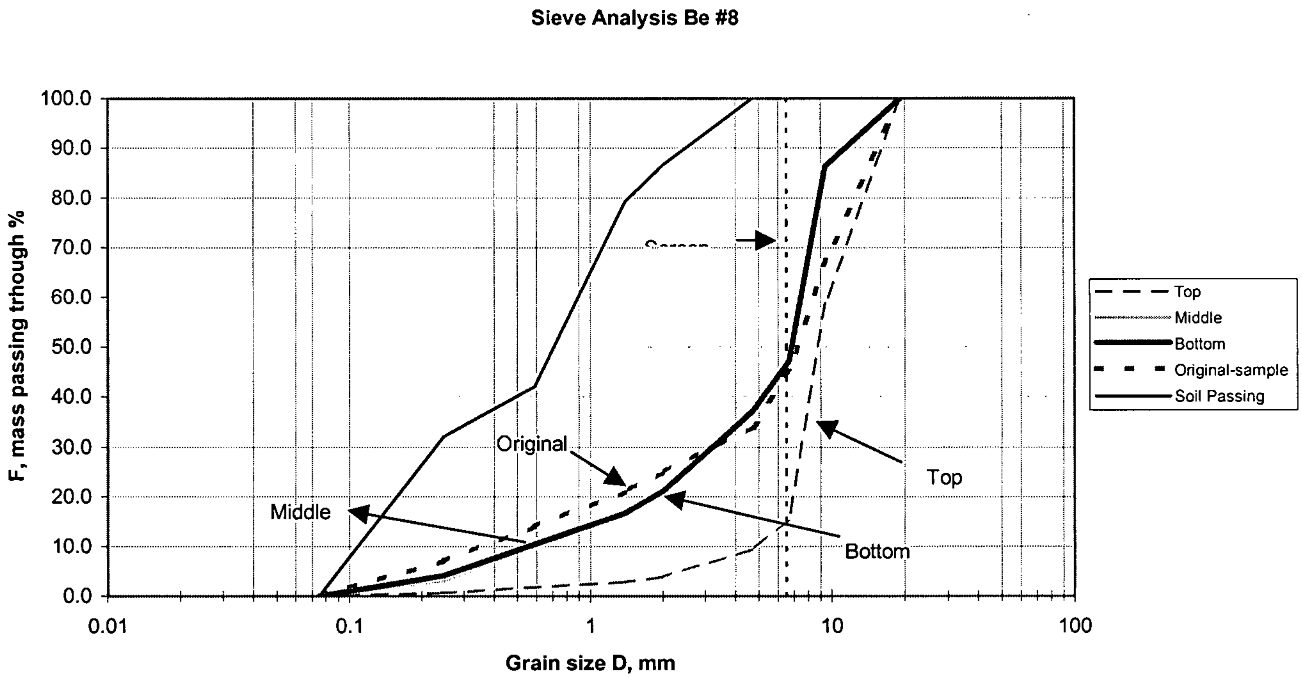

Figure 6.11. Test Be#8 (sieve analysis)

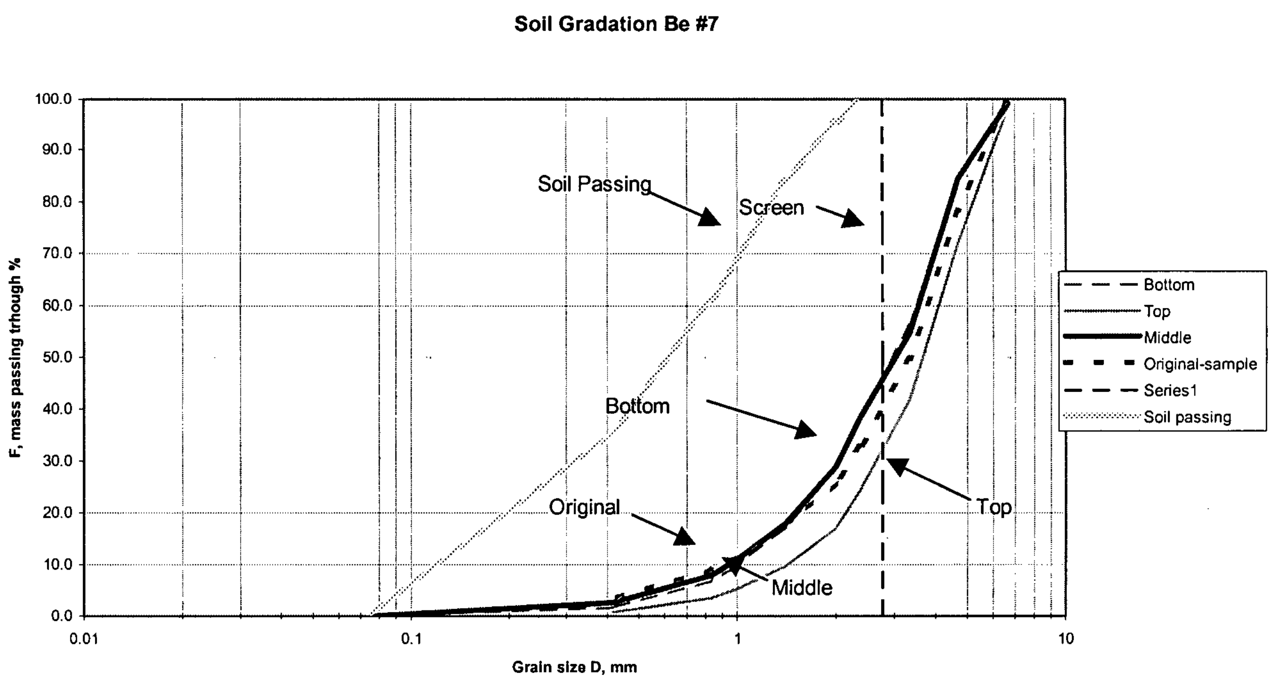

Figure 6.12. Test Be#7 (sieve analysis)

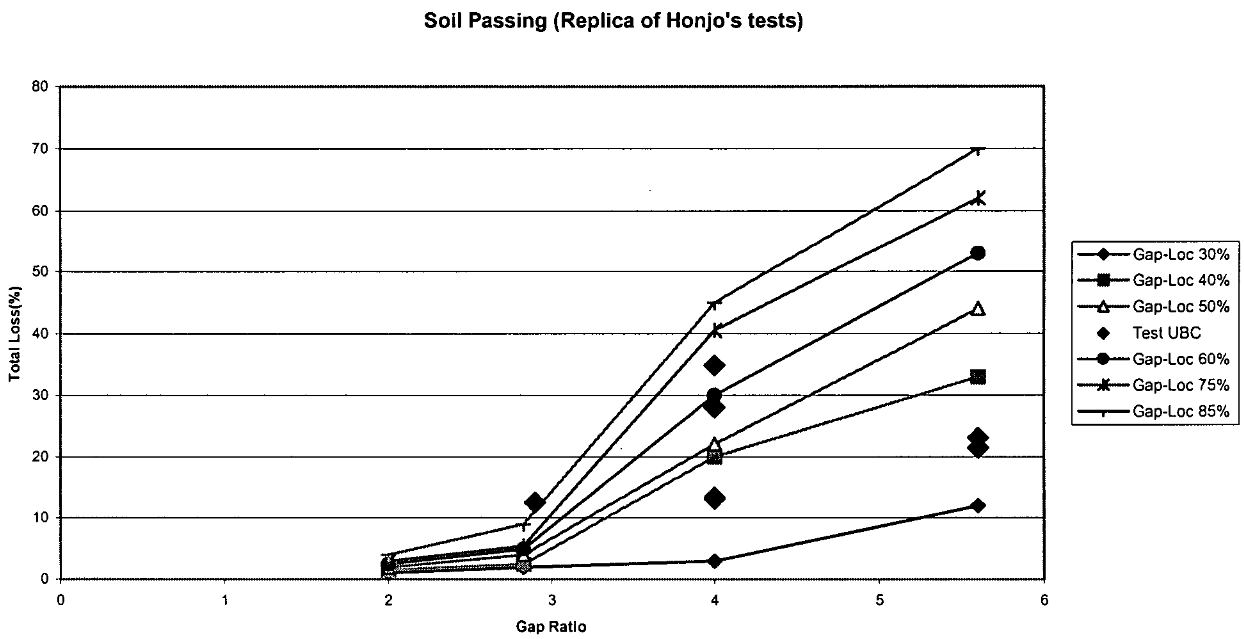

Figure 6.13. Comparison between Honjo et al. (1996) and UBC tests.

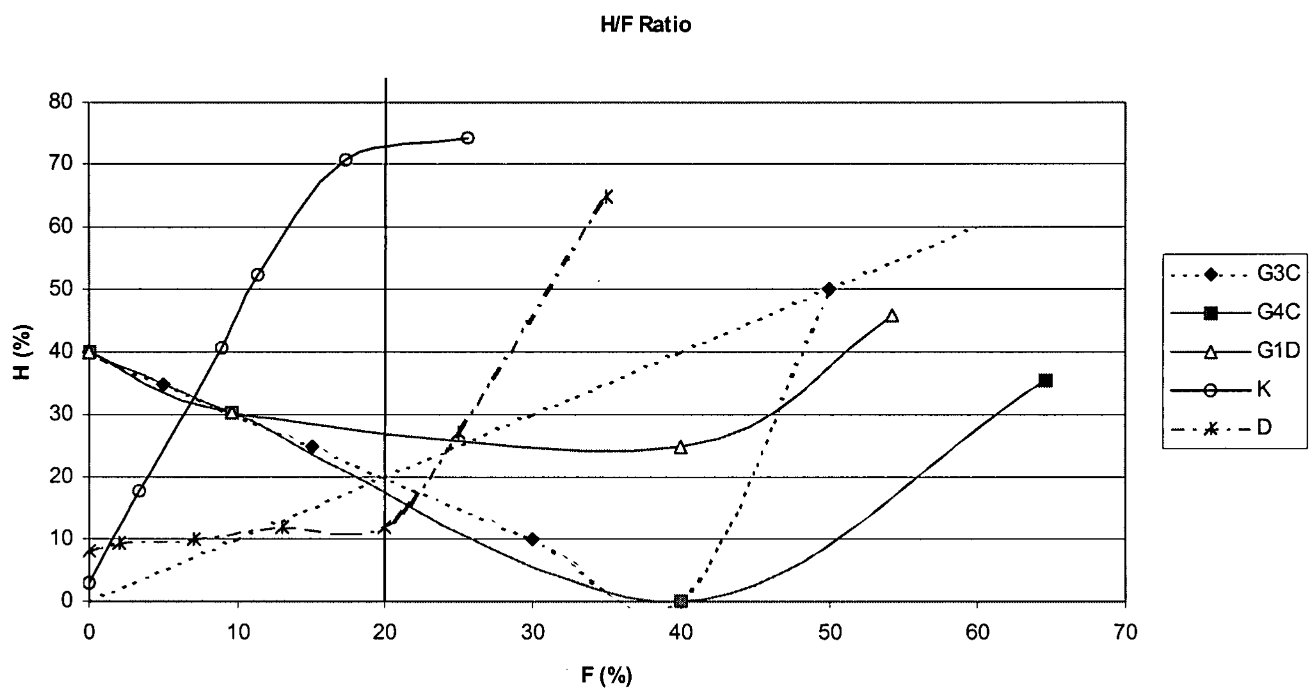

Figure 6.14. Evaluation of soil gradations using Kenney and Lau criteria

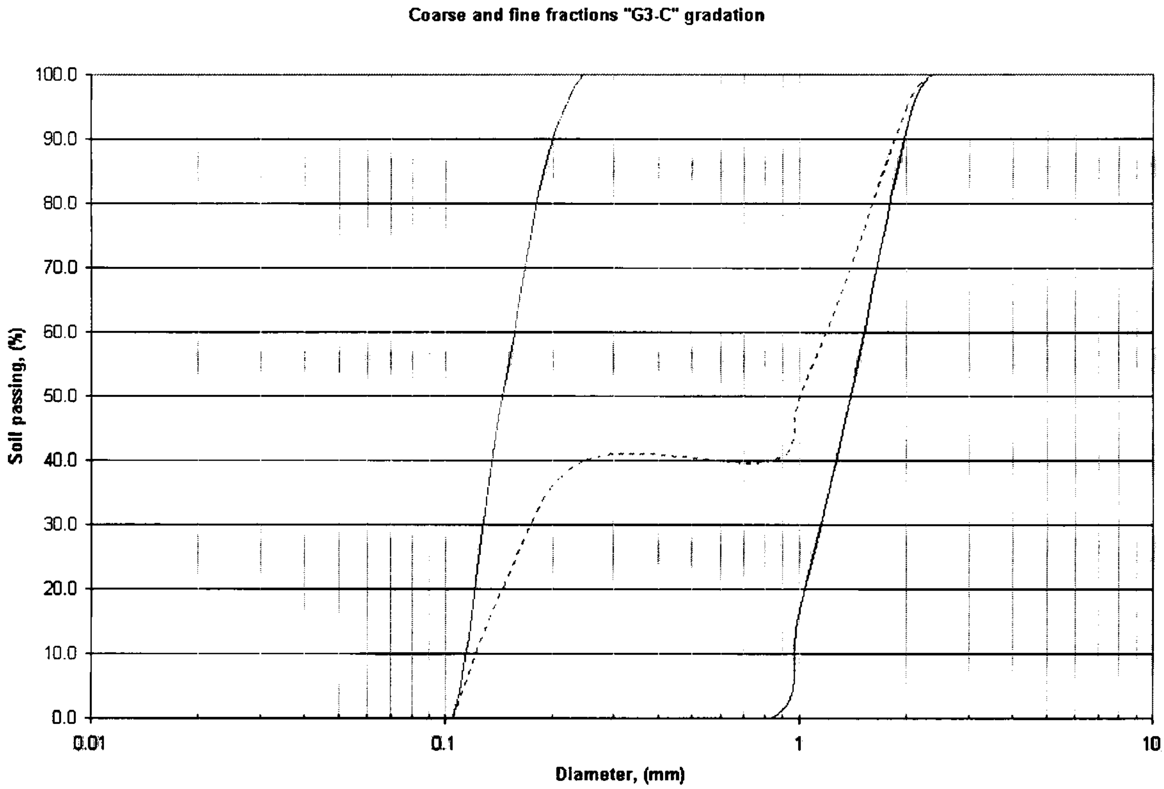

Figure 6.15. Kézdi's criterion, the split of gradation "G3-C"

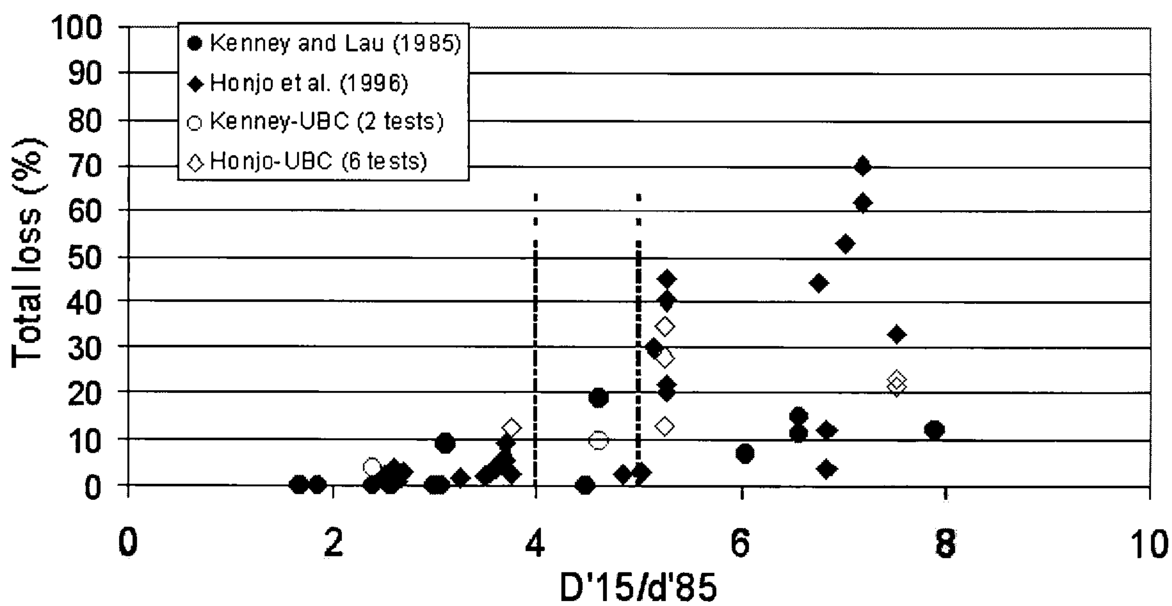

Figure 6.16. Summary of loss of soil versus D_{15}'/d_{85}'

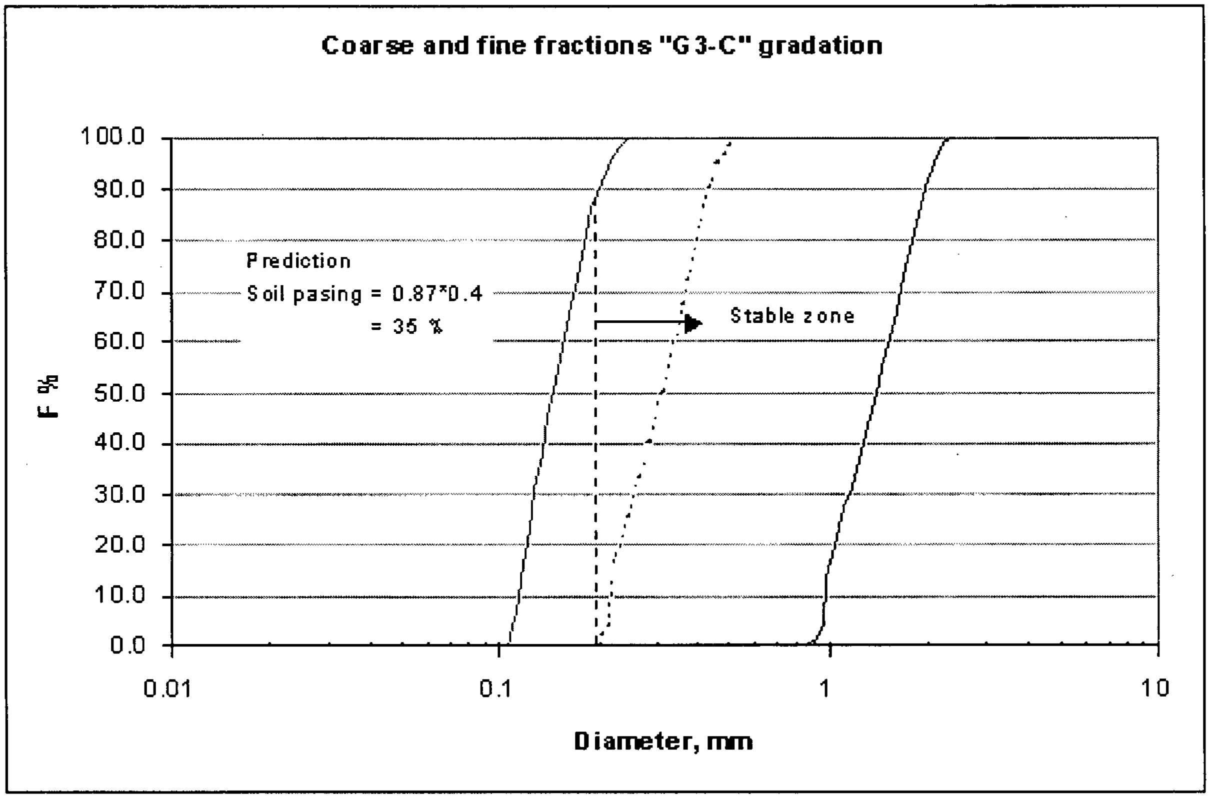

Figure 6.17. Soil passing prediction gradation G3-C (for original G3-C, see Figure 4.5)

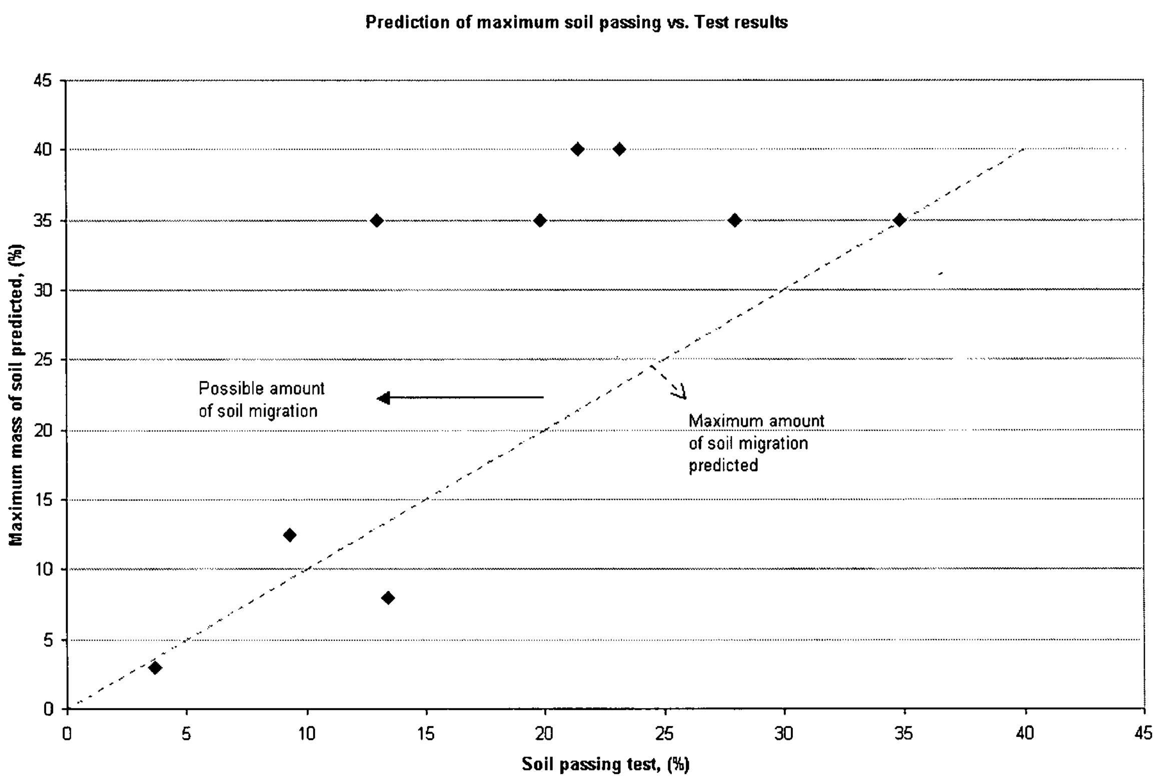

Figure 6.18. Summary of soil passing using split method