# A study of soil to geotextile filtration behaviour in conjunction with Berea sand in South Africa

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# DECLARATION

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# ABSTRACT

Geotextiles perform a number of functions in various applications in civil engineering practise. It is often cost effective and more environmentally friendly versus conventional construction methods. One of the main functions of a geotextile is filtration whereby the geotextile is expected to hold back the soil particles and simultaneously has to allow sufficient water to pass through it. Soils are all different and can be problematic when it comes to designing geotextile filters. One such problematic soil is encountered in KwaZulu-Natal, situated along the east coast of South Africa. The Berea sand is problematic as it can highly variable in its engineering properties over a small area.

Geotextiles are becoming more and more common practice in South Africa and little is known about the filtration performance of commercially available geotextiles in conjunction with Berea sand. Local guidelines that are available are out of date and do not provide enough information to assist design engineers in decision making. Many international guidelines are available and it is difficult to choose which one is best suited to Berea sands.

This primary objective of this study is to investigate the filtration performance of four variants of commercially available geotextiles and three variants of Berea sand. The applicability of some of the international filter design criteria will also be assessed. The soil to geotextile compatibility testing was carried out as per ASTM D5101 (2006) - Standard Test Method for Measuring the Soil-Geotextile System Clogging Potential by the Gradient Ratio. In total 12 permutations were executed. The results showed that only 5 test permutations met the gradient ratio and permeability criteria. The test results also conclude that the permeability is just as important as the gradient ratio. Thick geotextiles should be considered when used as filters in Berea sands. The available international geotextile filter design criteria were assessed and all showed poor correlation between laboratory results and suggested criteria. Designing geotextile filters in conjunction with Berea reds is challenging and it is recommended that design engineers perform laboratory performance testing in conjunction with their designs

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# OPSOMMING

Geotekstiele verrig vir ’n aantal funksies in verskeie programme in die siviele ingenieurswese praktyk. Dit is dikwels meer koste-effektief en omgewingsvriendelik, teenoor konvensionele konstruksie metodes. Een van die belangrikste funksies van ’n geotekstiel is filtrasie, waardeur van die geotekstiel verwag word om van die grond terug te hou, en gelyktydig genoeg water daardeur te laat vloei. Grond verskil en dit kan problematies wees wanneer dit kom by die ontwerp van geotekstiel filters. Een so ‘n problematiese grond kom voor in KwaZulu-Natal, geleë langs die ooskus van Suid-Afrika. (Die) Berea sand is problematies, want dit verander geweldig baie ten opsigte van ingenieurseienskappe oor ’n redelike klein area. Gebruik van geotekstiele word al hoe meer ‘n algemene praktyk in Suid-Afrika, terwyl min bekend is oor die filtrasie prestasie van kommersieel beskikbare geotekstiele in samewerking met Berea sand.

Plaaslike riglyne wat beskikbaar is, is verouderd en onvoldoende inligting is beskikbaar aan ontwerpingenieurs vir besluitneming . Baie internasionale riglyne is beskikbaar en dit is moeilik om te besluit watter een die beste van toepassing is vir Berea sand. Die doel van hierdie studie is om die filtrasie prestasie van vier modelle van kommersieël beskikbare geotekstiele en voorbeelde van drie soorte Berea sand te ondersoek. Die toepaslikheid van ’n paar van die internasionale filter ontwerp kriteria sal ook beoordeel word. Die toetsing van grondverenigbaarheid met geotekstiel is uitgevoer soos aangedui in ASTM D5101 (2006 ) – Standaard Toets Metode vir die meet van die grond-Geotekstiel verstopping potensiëel deur die gradient verhouding. In totaal is 12 permutasies uitgevoer. Die resultate het getoon dat slegs 5 toetspermutasies beide gradiënt verhouding en permeabiliteit kriteria bevredig het. Dikker geotekstiele word ook aanbeveel vir gebruik as filters in Berea sand. Van die toets resultate kan ook afgelei word dat die permeabiliteit net so belangrik soos die gradiënt verhouding is. Beskikbare internasionalegeotekstiel filter ontwerp kriteria is nagegaan en al die metodes het swak korrelasie tussen laboratorium resultate en die voorgestelde kriteria getoon.

Om geotekstiel filters in samewerking met Berea Reds te ontwerp is ’n uitdaging en dit word aanbeveel dat ontwerpingenieurs laboratorium prestasietoetsing in samewerking met hul ontwerpe uitvoer.

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# LIST OF SYMBOLS

| | | |
|---|---|---|
| A | = | Cross sectional area of soil |
| AOS | = | Apparent geotextile opening size |
| Cu | = | Co-efficient of uniformity of soil |
| $D_f$ | = | Fibre diameter of nonwoven geotextile |
| $d_x$ | = | Particle size of soil whereby x percentage is smaller |
| $g/m^2$ | = | grams per square meter |
| Gy | = | Gray |
| H | = | Hydraulic head |
| Is | = | Hydraulic gradient within soil |
| $k_f$ | = | Permeability of geotextile filter |
| kN/m | = | kilo Newton per meter |
| Ks | = | Permeability of soil |
| L | = | Water flow path length |
| $\ell/s/m^2$ | = | litres per second per square meter |
| Ls | = | 50 mm |
| Lsf | = | 25 mm + the geotextile thickness |
| m/s | = | meters per second |
| $m^2/s$ | = | square meters per second |
| mm | = | millimetres |
| Mm | = | micrometers |
| MPa | = | Mega Pascal |
| N | = | Newtons |
| n | = | Nonwoven geotextile porosity |
| $N_{constr}$ | = | Number of constrictions within nonwoven geotextile |
| $O_{90}$ | = | Pore opening size of geotextile where 90% is smaller |
| $O_{95}$ | = | Pore opening size of geotextile where 95% is smaller |

| | | |
|---|---|---|
| OF | = | Largest geotextile pore opening size |
| Q | = | Quantity of water |
| $s^{-1}$ | = | Seconds to the negative power of one |
| Sn | = | the standpipe reading in cm for the standpipe numbered n |
| ρf | = | fibre density of nonwoven geotextile |
| ΔH | = | change in water head |
| ΔL | = | change in water flow path length |
| μGT | = | Nonwoven geotextile mass |

# LIST OF ABBREVIATIONS

| | |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| AOS | Apparent opening size |
| ASTM | American Society for Testing and Materials |
| COLTO | Committee of Land Transport Officials |
| $C_u$ | Co-efficient of uniformity of soil |
| $G_R$ | Gradient ratio |
| ISO | International Organization for Standardization |
| K5-6 | Permeability across standpipes 5 and 6 |
| K6-7 | Permeability across standpipes 6 and 7 |
| LTGRT | Long term gradient ratio test |
| NP | Non plastic |
| $N_s$ | Number of constrictions in geotextile |
| NW-N-CF-PET | Nonwoven needlepunched continuous filament polyester geotextile |
| NW-N-SF-PP | Nonwoven needlepunched staple fibre polypropylene geotextile |
| NW-N-HB-PP | Nonwoven needlepunched continuous filament heat bonded polypropylene geotextile |
| PI | Plasticity index |
| POA | Percentage open area |
| $R_D$ | Relative density |
| SABS | South African bureau of standards |
| SANS | South African national standards |
| TMH1 | Technical methods for highways |
| W-SLF-PP | Woven slit film polypropylene geotextile |

# Chapter 1 - Introduction

## 1.1 GENERAL

A geosynthetic is defined as a planar, polymeric (synthetic or natural) material, used in contact with soil or rock and/or any other geotechnical material in civil engineering applications (International Geosynthetic Society).

According to historical records, it is believed that the first applications of geotextiles were woven industrial fabrics used in 1950s. The Netherlands used fibre mats as separation and filtration adjuncts in sea defence structures just after the 1953 floods. Another one of the earliest documented cases was a sea revetment structure built in Florida (USA) in 1958, where a woven geotextile was used as a filter beneath pre-cast concrete blocks. The first nonwoven geotextile was developed in 1968 by the Rhone Poulenc company in France. It was comparatively thick needle-punched polyester, which was used as upstream and downstream chimney filters during the construction of a dam in France in 1970.

In reality, the use of geotextiles dates back to ancient Mesopotamia, where papyrus mats were used for soil reinforcement and to promote soil consolidation. In South Africa, geotextiles have been introduced and successfully used since the 1960s. Ever since, there has been a rapid growth towards the use of geotextiles for the reasons outlined below:

- They are manufactured in a quality controlled environment, usually according to International Standards Organisation (ISO) standards.
- Geotextiles are usually easy and quick to install.
- Geotextiles can be manufactured from recycled materials, which reduces carbon footprint.
- The use of geotextiles reduces the dependency on natural raw materials, which are becoming scarce and expensive.
- The use of geotextiles can replace difficult designs using soil or other construction materials.
- Geotextiles often offer considerable cost savings versus natural materials.
- The technical database on their testing and design is reasonably established.

This has been the case because methods for constructing natural filters are tedious, time consuming and often the availability of suitable natural materials has become scarce. If adequately designed, specified and installed, geotextiles can provide a cost effective solution for drainage and filtration in civil and environmental engineering works.

Designing with and specifying geotextiles by function has become specialised and this research will focus only on the filtration characteristics of the soil to geotextile behaviour systems within the Berea Red sands of the KwaZulu-Natal Coastal region. Often design engineers neglect the importance of inducing proper specifications. As a result the system could potentially fail. The behavioural mechanism of soil to geotextile flow compatibility is not always well understood. Where geotextiles are to perform as filters they must be adequately permeable and also have the ability to optimally retain soil particles. A third factor is also involved; a long term soil-to-geotextile flow compatibility that will not excessively clog the fabric during the lifetime of the system. Internationally, extensive research is continually being carried out on soil-to-geotextile filtration theory. This has formed a good basis for further local studies.

With regard to filtration theory, the most common failure mechanisms of a geotextile filter are blinding, blocking and clogging. These failure mechanisms and possible causes thereof will be discussed later.

It is therefore important to know and identify potentially problematic soils during the early planning stages of a project to eliminate or reduce the probability of failure occurring in the soil to geotextile filtration system. One such potentially problematic soil is the coastal sand also known as Berea sand which is, as previously mentioned, typically found along the KwaZulu-Natal (KZN) coastline, stretching northward into Mozambique.

Berea sand is Aeolian of origin. It has high variability in terms of its plasticity index which can be found to be non plastic to a plasticity index of approximately 12 percent. The sand has a relatively low grading modulus and has a soil classification of between G6 – G10 according to TRH 14 (1985). The Berea sand is also considered as highly erodible. The sand's colour varies between light yellow to orange and light brown to dark red.

Understanding the geotextile to soil behaviour with sand from the Berea formation will enhance the probability of better geotextile design and selection of filtration media for this region.

## 1.2 SCOPE OF STUDY

The area of study focused on soil-geotextile filtration and the current state of the art literature available to support the study. An evaluation was done of four commercially available geotextiles in conjunction with samples of three different types of Berea sand. The selection of the sand samples was based on the variability of their plasticity indices and their coefficients of uniformity. The study also focused on the pre and post permeability of the geotextiles subjected to these soils. Further outcomes of the study ascertained whether the co-efficients of uniformity of the soils and tensile strengths of the geotextiles had any significant impact on the soil to geotextile filtration system.

Soil to geotextile filtration tests were carried out using ASTM 5433. This test method is described in Chapter 3.

## 1.3 DISSERTATION STRUCTURE

The format of the thesis follows the chronological order illustrated in Table 1.1 below:

**Table 1.1: Thesis format**

| |
|---|
| CHAPTER 1: INTRODUCTION |
| CHAPTER 2: LITERATURE REVIEW |
| CHAPTER 3: METHOD |
| CHAPTER 4: RESULTS |
| CHAPTER 5: ANALYSIS |
| CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS |

Chapter 2 investigates the development of geotextiles, wherein filtration theory and various filter criteria will be discussed. Chapter 3 describes the research materials and their engineering properties. The research methodology and background of the computer model are discussed in Chapter 4. The analysis, results and findings of the laboratory testing are discussed in Chapter 5. Various international geotextile design criteria were also investigated, and compared to the laboratory tests as well as to the computerised predictive model results. Chapter 6 concludes this thesis with the conclusions drawn from the research and recommendations for further study.

## 1.4 AIMS AND OBJECTIVES

There was currently no South African code of practice that adequately addressed geotextiles in filtration applications. However, a filter design guideline did exist and was available from a few South African geosynthetics manufacturers (Kaytech and Fibretex SA). There was therefore a need for bridging the gap in order to understand the mechanism and behaviour of soil to geotextile filtration systems. This is critical in the design process, where insufficient knowledge could lead to potential failures and loss of life, which could be encountered on projects e.g. dams, harbours, jetties, breakwaters and retaining structures, to name a few.

This research was not aimed at replacing existing guidelines, but at augmenting these existing guidelines as needed for locally practising engineers. This would assist in making sound engineering decisions when it came to specifying geotextiles as filters in Berea sands.

The aim of this research was to further evaluate:

- The difference in behaviour as a filter of four variants of geotextiles in typical problem soil, such as Berea sand.
- The clogging potential of these geotextiles in conjunction with Berea sand.
- The influence that the co-efficient of uniformity (Cu) has on the soil to geotextile filtration system.
- The effect of geotextile fabric.

# Chapter 2 – Review of Geotextiles and Filtration Theory

## 2.1 INTRODUCTION

This chapter will discuss the different types of geotextile, their polymer makeup and manufacturing techniques, as well as soil-to-geotextile filter mechanisms and criteria. It is important to have a good understanding of how soil filtration theory relates to geotextiles, as many theories are common to both, but there are also differences. Furthermore, this chapter highlights the theories and criteria given in past research and also how these relate to natural soil and geotextile filters for present day consideration.

## 2.2 GEOTEXTILES

Geosynthetics comprise eight main categories, as follows:

1. Geotextiles
2. Geogrids
3. Geonets
4. Geomembranes
5. Geosynthetic Clay Liners
6. Geopipes
7. Geofoam
8. Geocomposites

Information is freely available on all the above geosynthetics. For the purpose of this thesis, only geotextiles will be discussed. Geotextiles form one of the two largest groups of geosynthetics, the other being geomembranes. (Koerner, R.M., 2005)

Geotextiles are, in reality, textiles but consist of synthetic fibres, rather than natural ones. This helps in the sense that synthetic fibres will probably better withstand the design life of any proposed structure. These synthetic fibres are made into flexible, porous fabrics, either by standard weaving machinery or by being matted together in a random, nonwoven manner. Some fabrics are also knitted. The major point is that geotextiles are porous to liquids, both perpendicular to their manufactured planes, as well as within their thickness. Not all geotextiles are able to allow liquids to permeate along their plane, as they are relatively thin as, for example, a flat woven geotextile. This will be explained in more detail in the later part of this chapter. A geotextile has numerous end uses, but performs at least one of four functions: filtration, drainage, separation and reinforcement.

## 2.3 POLYMERS

The following polymers are most commonly used in the manufacture of geosynthetics (Koerner, 2005):

- High Density Polyethylene (HDPE) – developed in 1941
- Linear Low Density Polyethylene (LLDPE) – developed in 1956
- Polypropylene (PP) – developed in 1957
- Polyvinyl Chloride (PVC) – developed in 1927
- Polyester (PET) – developed in 1950
- Polyamide (nylon) – developed in 1938
- Expanded polystyrene (EPS) – developed in 1950
- Chlorosulphonated polyethylene (CSPE) – developed in 1965
- Thermoset polymers such as ethylene propylene diene terpolymer (EPDM) – developed in 1960

Geotextiles are made of the following polymers (Koerner, 2005):

- Polypropylene (92%)
- Polyester (5%)
- Polyethylene (2%)
- Polyamide (nylon) (1%)

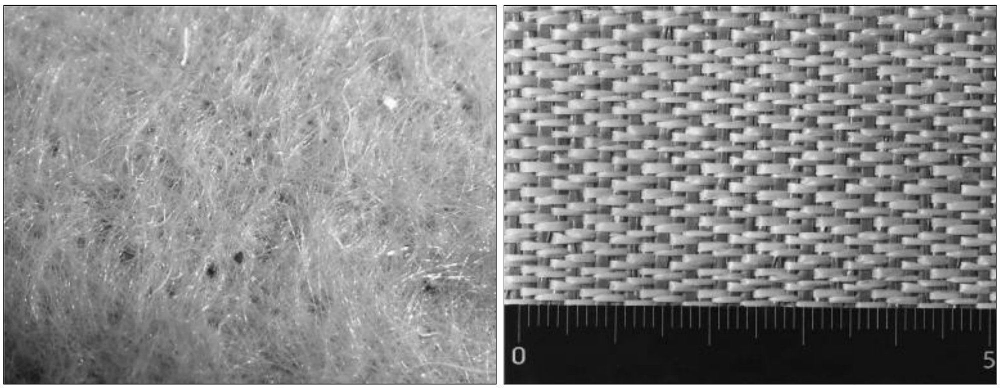
## 2.4 MANUFACTURING TECHNIQUES

Geotextiles are manufactured in different ways. Hence they are classified differently. The most common ones are briefly outlined below:

*Wovens:* there are two basic types of woven geotextiles: slit-films, which are made by weaving the flat strands that have been created by first slitting a plastic sheet; and monofilaments, which incorporate round strands that are extruded.

*Non-wovens:* these are flexible geotextiles which are manufactured by bonding together fibre mats which comprise layered, disarranged, spun fibres or filaments. The bonding may be achieved either mechanically (needle-punching), by adhesion (glueing) or by cohesion (melting).

*Composites:* these are multi-layered geotextiles comprising different layers of differing structure which are bonded together over their entire surface. The individual components may be woven, non-woven or other specially constructed geosynthetics. The individual layers may be bonded together by needle-punching, welding, sewing, glueing or a combination of these methods.




a b

**Figure 2.1: a) Nonwoven needle punched b) Woven slit film geotextile**

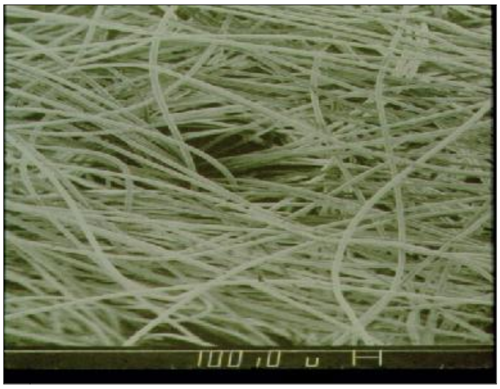


**Figure 2.2: 100 x magnification of nonwoven** (Kaytech)

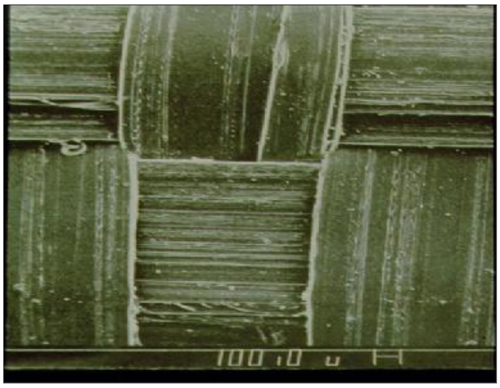


**Figure 2.3: 100 x magnification of woven geotextile** (Kaytech)

## 2.5 HISTORY

Geotextiles, as they are known and used today, are intended to be an alternative to natural, granular filters. Barret (1960) describes work originating in the 1950s using geotextiles behind precast concrete seawalls, under concrete erosion control blocks, beneath large stone rip rap, and in other erosion control situations. Dunham and Barret (1974) point out that when sand and gravel filters were installed to hold fine soils in place under erosion protection structures, scour and erosion sometimes undermined these protective structures, but woven plastic filter cloths have been used successfully. Barret was known as the 'ather of filter fabrics'. During 1957 and 1976 he also pioneered the use of filter fabrics in other applications such as French drains, scour protection around bridge piers, fabric wrapped around pipe, and the fabric encapsulated sand core breakwater. The need for both adequate permeability and soil retention, along with adequate fabric strength and proper elongation, set the tone for geotextile use in filtration applications. Agershou (1961) also describes the earlier use of geotextiles in a similar way.

Acceptance of geotextiles for civil engineering projects was at first slow as their use represented a relatively new concept. Over the years geotextiles have proven their good performance and their uses rapidly increased, as various suppliers started entering the market with a variety of geotextiles having various different properties.

## 2.6 CHARACTERISTICS OF WOVEN AND NONWOVEN GEOTEXTILES

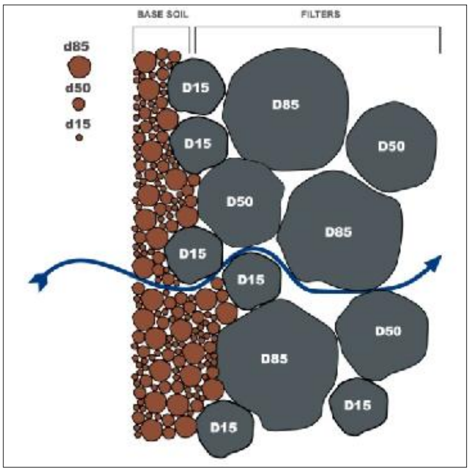



**Figure 2.4: Natural filter formation without geotextile** (Kaytech)

In Figure 2.4 above it can be seen that the filtration mechanism for a natural filter is formed when the bigger soil particles ($D_{85}$) hold back smaller soil particles ($D_{15}$), which in turn hold back even smaller particles. This illustrates a typical natural 'Terzaghi' type filter, where:

$D_{85}$ = particle size of the soil when 85% is smaller

$D_{15}$ = particle size of the soil when 15% is smaller

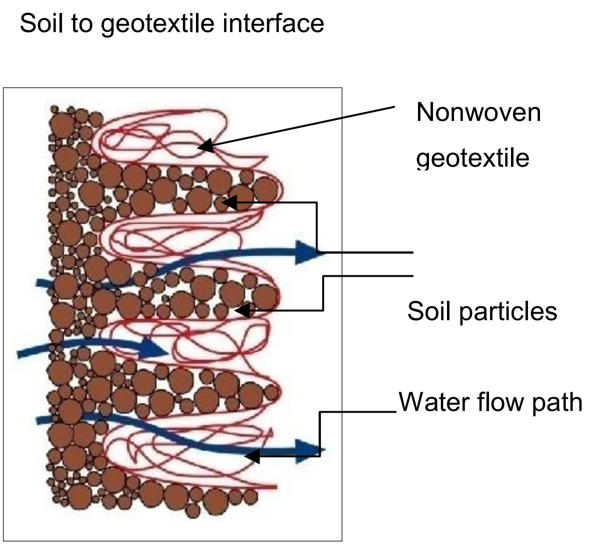



**Figure 2.5: Soil to geotextile system for nonwoven geotextile** (Kaytech)

When a nonwoven geotextile is to perform as a filter, as illustrated in Figure 2.5, there is some close interaction between the fibres of the geotextile and the soil. This is important for the promotion of a reverse filter, as will be discussed later in this chapter.

Figure 2.6 illustrates the soil geotextile interaction for a woven geotextile. This system does not support a reverse filter very well, as will be discussed later in this chapter.

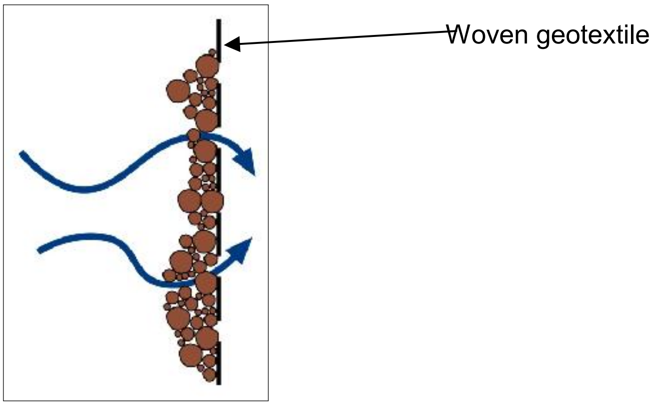



**Figure 2.6: Soil to geotextile system for woven geotextiles** (Kaytech)

### 2.6.1 Thickness

Woven geotextiles most often consist of a flat weave and often, by way of structure, are not considered three dimensional; they are usually less than 1 mm thick (Table 3.1). Thickness is an important parameter when in plane drainage is to be considered. In plane drainage is the ability of a geotextile to allow water to flow in the plane of the geotextile. In general, woven geotextiles do not exhibit good in-plane drainage characteristics.

This is important when a geotextile is used as a vertical drainage dissipater behind retaining structures. In other words, there is great dependency on the geotextile to drain water along its plane.

Nonwoven geotextiles, on the other hand, have thickness. According to standard laboratory testing, a typical commercially available nonwoven geotextile exhibits thickness ranging from approximately 1 mm to 10 mm under an exerted pressure of 2 kPa (Giroud 2010).

### 2.6.2 Elongation

Woven geotextiles do not exhibit high elongation characteristics and this characteristic lends itself to spanning across voids in the soil when under strain. Typically, woven geotextiles would strain approximately 10% to 20% before rupture. When a geotextile spans across a void where it is expected to perform as a filter, fine soil particles in suspension are deposited upstream, adjacent to the geotextile filter. This is due to the fact that water velocities are reduced by intersecting these voids and hence the depositing of the soil particles. This phenomenon can lead to the geotextile becoming blinded or blocked. This reduces the filtration characteristics of the system and could lead to possible failure of the structure.

## 2.7 GEOTEXTILES AS FILTRATION FABRICS

In simplistic terms, a filter must retain the soil and allow the water to pass through. These two requirements are contradictory if strictly formulated. If it were required for all soil particles to be retained, an impervious screen would be needed, in which case water would not be able to flow through it. Conversely, if it were required that the flow of the water was not to be impeded then the openings of the filter would be too large and no soil particles would be retained.

It is therefore important that there should be a balance between the two criteria. A good filter has openings both large and small enough. It needs openings large enough to let water flow freely but it should have openings small enough so that the soil skeleton, which gives the structure stability, is not disturbed as a result of a loss of fine particles. So to evaluate these two criteria, a filtration theory must be established (Giroud, 2010). This leads the researcher to say that a complete theory would be difficult to formulate, due to two reasons as outlined below (Giroud, 2010):

i) phenomena such as:

- *two phase flow and capillarity*
- *chemical and electrical interactions between filter and particles,*
- *erosion*
- *variation of the mechanical behaviour of soil as a function of water content and pore water pressures.*

ii) parameters such as :

- *geometrical conditions* (shape of the soil mass, location of the fluid, flow direction which may vary) and
- *mechanical conditions* (gravity, stresses)
- *properties of materials* such as the fluid (composition, density and viscosity),
- *the soil particles* (shape, dimension, mechanical properties such as friction and cohesion, permeability)
- *constituents of the filter* (shape, dimensions, distribution, density and chemical nature of solid fibres of the filter, void distribution of filter)
- *filter* (continuity, permeability, mechanical properties such as compressibility).

At the present time, there is still a way to go before having a complete theory dealing with the above phenomena and parameters. A simple approach is used for granular filters as well as for geotextiles, which consists of two criteria, the permeability criterion and the retention criterion. Soil filtration with geotextiles is neither better nor worse understood than soil filtration with granular filters. Giroud (2010) proposed that criteria for geotextiles are probably as valid as the classical criteria used for granular filters.

One of the most common applications of a geotextile is as a filter (Koerner, 2005). A geotextile filter is often used in subsurface drainage applications (Figure 2.8). The purpose of a subsurface drain is to have a draw down effect on the water phreatic surface in order to increase the bearing capacity and performance of soils. In this application, the geotextile is expected to hold the sand particles behind, as well as to let enough water flow perpendicularly through the plane of the geotextile.

### 2.7.1 Roadside subsurface drain

In Figure 2.7, the geotextile acts as a filter between the in-situ soil and the drainage stone. The geotextile will stop fine soil particles from migrating into the stone drain and eventually causing failure of the subsoil drain.

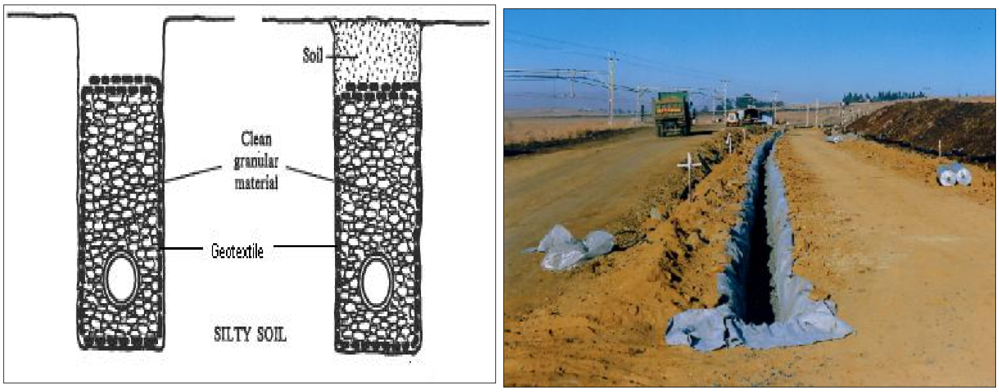



**Figure 2.7 Geotextile acts as a filter in subsurface drainage application alongside a road** (Kaytech)

### 2.7.2 Drainage in railway construction

In Figure 2.8, the geotextile acts as a filter below railway ballast and in the side subsoil drains. Effective drainage is critical, as subsidence due to moisture and cyclic loads can be catastrophic for rail superstructure.

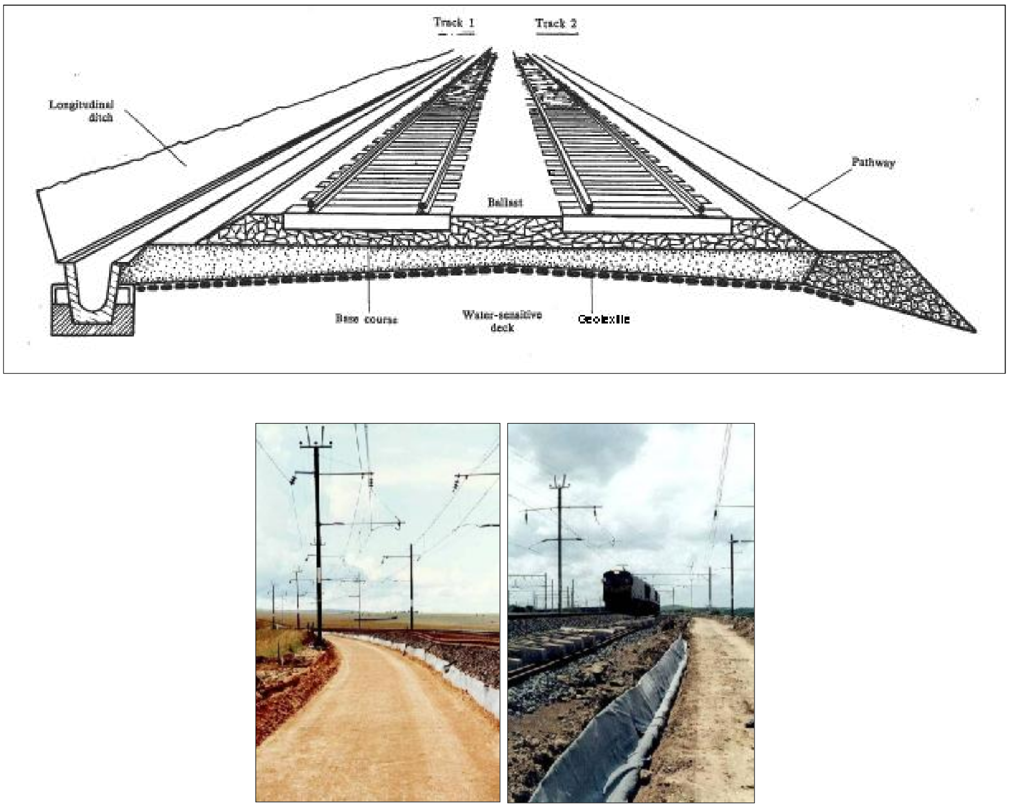



**Figure 2.8: Geotextile as a filter medium under and next to railway** (Kaytech)

### 2.7.3 Drainage of embankment dams

Geotextiles are also often used as filters during dam wall construction. In Figure 2.9 below, the geotextile acts as a filter under the rock rip rap that is adjacent to the clay core of the dam. The geotextile filter is intended to prevent the washing out of fines and thus causing rock settlement on the upstream side of the dam wall. Secondly, geotextile also acts as a filter adjacent to the clay core on both the upstream and downstream side. These filters are critical as they maintain the integrity of the clay core, as well as prevent the building up of excess pore water pressure adjacent to the clay core. Thirdly, the geotextiles act as a filter between the in-situ and imported fill for the subsoil drainage blanket adjacent to the downstream side of the clay core, as well as under the downstream fill embankment. Finally, the geotextile would act as a filter around the toe collector drain, which often forms part of the blanket drain. Also refer to Figure 2.10 below, where the rock rip rap embankments of a dam are underlain with a geotextile filter.

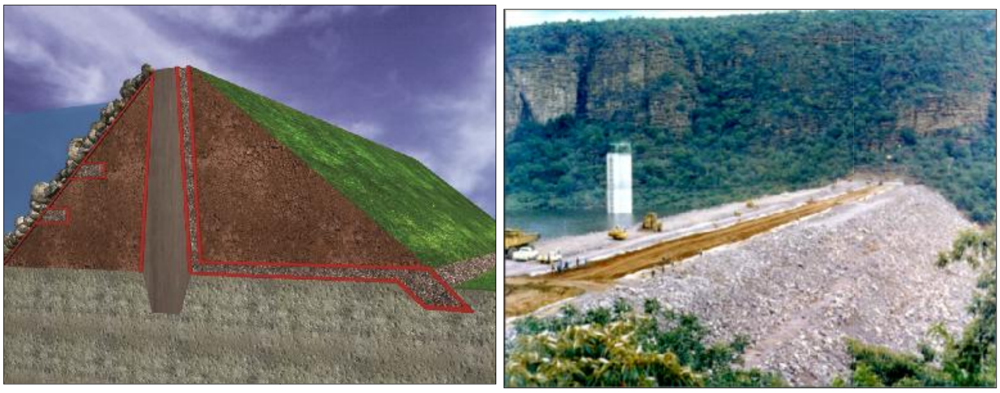


**Figure 2.9: geotextile filter under upstream rip rap, in adjacent chimney-and downstream drains for earth fill dam** (Kaytech)

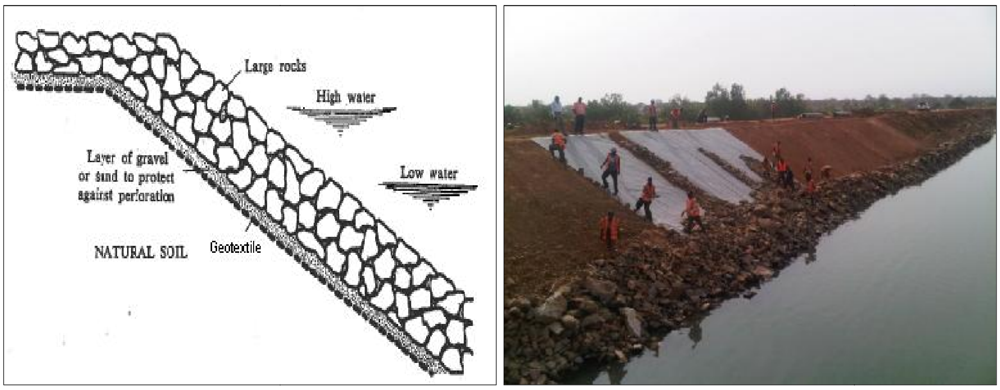



**Figure 2.10: Geotextile filter under rip rap scour protection** (Kaytech)

### 2.7.4 Gabion structures

A gabion structure should always have a geotextile filter incorporated on its upstream side, as in Figure 2.11. The voids in a gabion structure are often big and will easily let soil behind it pipe through the structure. A geotextile filter will prevent the piping of the soil from behind the gabion structure and allows the gabion to effectively perform its retaining function.

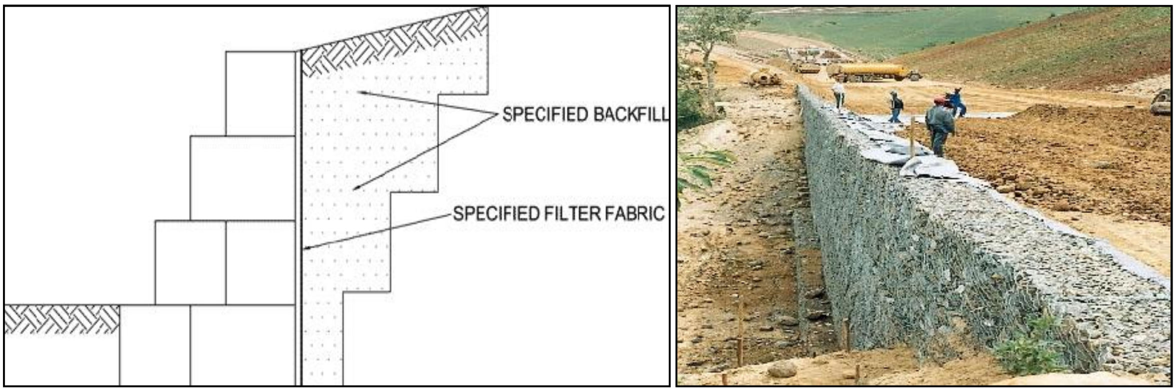



**Figure 2.11: Geotextile as a filter behind gabion and gabion mattresses** (Kaytech)

#### 2.7.5 Sports field drainage

In this application the geotextile acts as a filter between the imported stone drainage medium and the in-situ soil. The drainage system can form a herringbone system under the field or a blanket drain. The latter is recommended, although more expensive, because of the greater quantity of stone and geotextile that is used (Figure 2.12). The herringbone system is justified on cost, and also where it is accepted that a pool of standing water on the sports field will not be catastrophic, for example on a school sports field.

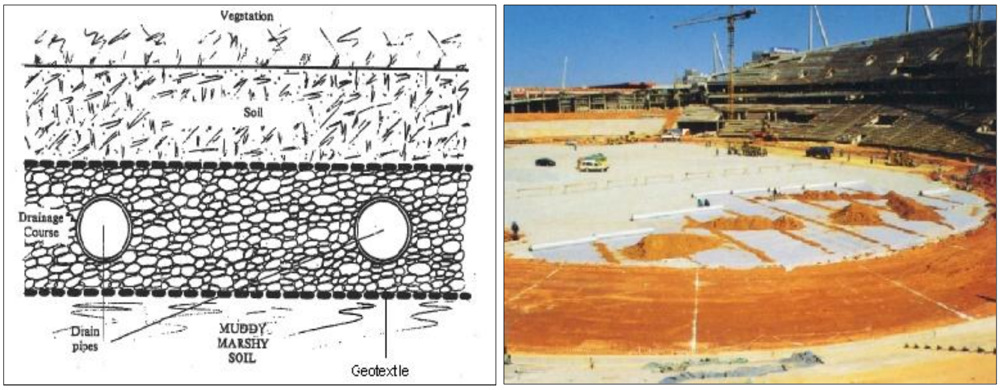



**Figure 2.12: Geotextile performing as a filter under sports field drainage** (Kaytech)

### 2.7.6 Drainage in tailings dams

Figure 2.13 shows that thick nonwoven geotextiles are often used a filter between the in-situ and imported tailings materials. Because of the fine nature of tailings, it is critical to dimension the filter correctly to prevent potential clogging of the geotextile filter and subsequent failure of the drain.

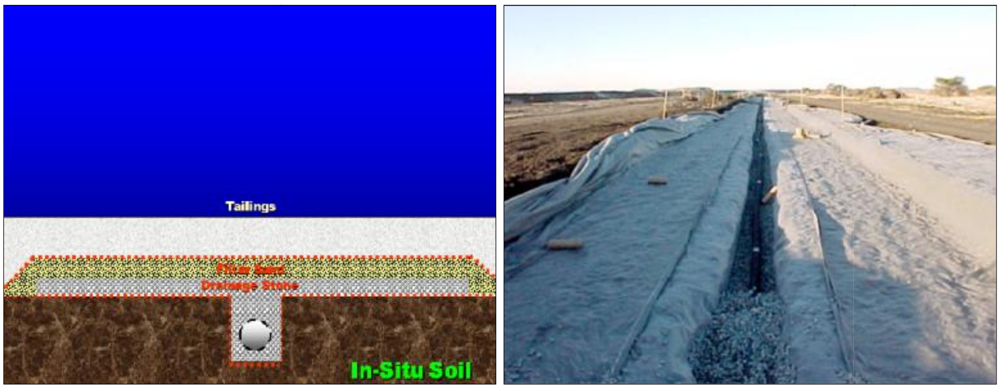



**Figure 2.13: Geotextile used as a filter medium below the toe drain of tailings dam**
(Kaytech)

## 2.8 GEOTEXTILE FILTRATION MECHANISMS

### 2.8.1 Geotextile to soil contact

When geotextiles are used in earth structures to restrain soil migration and to retain particles in water three distinct filtration mechanisms are prevalent: first, the geotextile itself, which is in contact with the soil; secondly, particles in suspension and, thirdly, the migration of soil particles under cyclic loading.

The drainage of a saturated soil is usually achieved by constructing a trench and subsequently lining the trench with a geotextile. A stone drainage medium, in conjunction with a perforated pipe, is installed and the entire system is enveloped with the geotextile. The entire system is buried where it performs its drainage function.

The water flow rate within the soil can be estimated using Darcy's law:

$$Q = k\,(\Delta H / \Delta L)\,A$$

where:

Q = water flow rate in $m^3/s$

k = soil permeability in m/s

A = soil cross sectional flow area in $m^2$

H = hydrostatic head in m

L = water flow path length in m

The permeability and pore size distribution of geotextiles are selected to restrain particle migration while allowing the water to percolate through. These properties are a function of the structure of the geotextile and must remain constant throughout the service life of the system.

The long term performance of geotextile filters can be affected by soil conditions, the design of the system, installation procedures and hydraulic conditions. It is therefore important to understand that the successful performance of a geotextile filter includes it encouraging the formation of a natural upstream filter. This usually occurs by either one or a combination of the following two mechanisms:

#### *2.8.1.1 Self-filtration*

Under hydraulic gradient, soil particles will migrate towards the drain. The particles with smaller diameter than the geotextile pore opening size and located adjacent to the geotextile lining the drainage trench will be carried into the geotextile by the flowing water. These particles will either be flushed into the drainage pipe system or will remain trapped permanently between the fibres. As the finer particles are removed from the soil, the coarser soil fraction will migrate towards the geotextile where it will be stopped. This is assuming that the coarser fraction is larger than the pore opening size of the geotextile. These larger particles will, in turn, stop finer particles from migration, which will in turn stop even smaller particles. As a result, coarse particles will form a layer at the geotextile interface and the soil migration will be stopped. This phenomenon is favoured in well graded soils (Rollin and Lombard, 1988).

#### *2.8.1.2 Vault network formation*

In soils that are not well graded, the geotextile can be properly selected to favour vault formation (Figure 2.14). Particles adjacent to the geotextile can rearrange themselves as they migrate toward the filter interface to form vaults. This is believed to be as a result of the electrical and adsorption forces between the organic lubricant or anti-static agent on the geotextile fibres and soil particles and also between the soil particles themselves. Upon formation of this vault network, the geotextile will stop particles with a smaller particle size than that of the geotextile, from migrating through it (Rollin & Lombard, 1988). This is favourable, as no further piping of finer soil through the geotextile can occur.

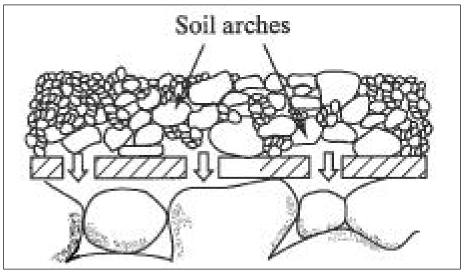



**Figure 2.14: Upstream soil particles forming vaults or arches over geotextile openings** (after McGown 1985)

### 2.8.2 Particles in suspension

Besides subsurface drainage applications, geotextiles are also frequently used as filters in silt fences, sea barriers, tailings storage facilities and other places. Geotextiles are in these instances used to protect rivers and lakes from being contaminated by silt.

If care is not taken during installation, fine soil particles may be carried in by the run of water and settle on top of, or adjacent to, the geotextile filter. A soil particle carried in suspension will remain so if its velocity is higher than that of the gravitational forces acting on it. Clay and silt particles will remain in suspension if their velocity is greater than 0.01 m/s. These particles reach the geotextile interface at relatively high velocity. In this instance the filtration mechanism reacts totally differently than in the case previously mentioned, as the formation of an impervious layer on the upstream side of the geotextile is highly probable (Rollin & Lombard, 1988). This phenomenon is known as blinding or blocking, in the cases of nonwoven and woven geotextiles respectively.

Under relatively high flow rates, particles will block the pores of the geotextile and slowly start building up an impervious layer on the adjacent upstream side of the geotextile. Over time the geotextile filter will inevitably stop functioning and will need replacement. In these cases the filter behaves similarly to the way it would in industrial filtration. In this instance, geotextile filter selection should be based on the life expectancy of the structure or system (Rollin & Lombard, 1988).

### 2.8.3 Soil particles migration under cyclic loads

In applications such as railways and access roads, cyclic loads are exerted onto the geotextile due to the passage of heavy trains and vehicles. Under these conditions, smaller particles tend to migrate towards the geotextile due to the pumping effect on them. This effect is a function of the soil type, particle distribution, soil water content, and the amplitude and frequency of the applied load. For silty and silty clay soils, the geotextile must be able to stop the flow of slurry, consisting of the in-situ water and the soil that will flow as a result of cyclic loading (Rollin & Lombard, 1988).

## 2.10 FURTHER DEVELOPMENTS FOR GEOTEXTILE FILTERS

Earlier work was undertaken by Giroud (1982 and 1986) to develop the basic criteria for geotextile filters. These criteria are also applicable to granular filters. The classical Terzaghi's criteria in terms of permeability and retention for granular filters are expressed by the following equations:

$$\mathbf{d_{15F} \geq 4 \text{ or } 5d_{15S}} \qquad \textbf{(1)}$$

$$\mathbf{d_{15F} \leq 4 \text{ or } 5d_{85S}} \qquad \textbf{(2)}$$

where:

$d_{15F}$ = $d_{15}$ of the filter where 15% of the particle size are equal or smaller

$d_{15S}$ = $d_{15}$ of the soil where 15% of the particle size are equal or smaller

$d_{85S}$ = $d_{85}$ of the soil where 85% of the particles size are equal or smaller

Terzaghi (1922) and Bertram (1940) identified the $d_{85}$ of the soil as a suitable parameter in their filter criteria, and this has been universally adopted. Lund (1949) and Soares (1980) carried out tests on soils using a series of sieves, each sieve acting as a single filter. They confirmed an abrupt loss of fines if the sieve sizes were bigger than $d_{85}$ of the base soil. Therefore the $d_{85}$ is a good parameter to use against the excessive loss of fines.

Equation 1 means that the $d_{15}$ of the filter must not be too small, which is the permeability criterion. Equation 2 means the $d_{15}$ of the filter must not be too large, which relates to the retention criteria. So Terzaghi's filter criteria comprise a permeability criterion and a retention criterion. However, for geotextile filters, two additional criteria are needed (Giroud, 2010).

Since there is an abundance of geotextiles that are commercially available, it is possible that some geotextiles may not perform adequately as filters. These are those which are very thin and those with very few openings. Hence criteria needed to be developed to avoid the use of these types of geotextiles as filters, especially in critical applications. These two extra criteria for ensuring sufficient number of openings and sufficient thickness were called the porosity and thickness criteria (Giroud, 2010).

Terzaghi's criteria for granular filters are applicable only to cohensionless soils. This thesis was limited to the Berea sands, where plasticity ranged between non-plastic and a plasticity index of 7.

## 2.11 PERMEABILITY CRITERION

The presence of a filter disturbs the flow of the water in the soil upstream of the filter. The selected filter must offer as little disturbance as possible, as it can affect the flow rate and the pore pressure. Therefore the permeability criterion must consider pore pressure requirement and flow rate requirement (Giroud, 2010).

### 2.11.1 Pore pressure requirement

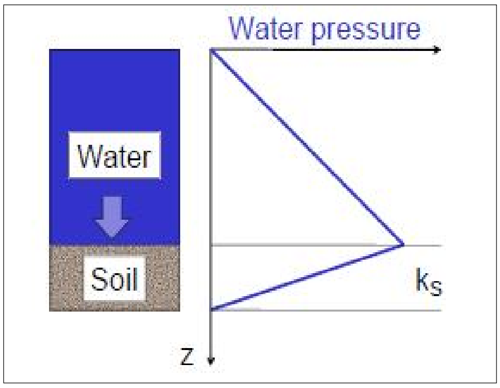



**Figure 2.15: Pressure diagram showing pore water pressure with depth and no geotextile present** (Giroud 2010)

In Figure 2.15, there is an increase of pore pressure with depth. From the soil-to-water interface, to a depth z, the pressure decreases. This is the case where the permeability of the soil ($k_s$) is sufficiently high not to allow the build-up of excess pore water pressure at the water to soil interface. Note that in the above case the soil at depth (z) forms part of the free draining boundary.

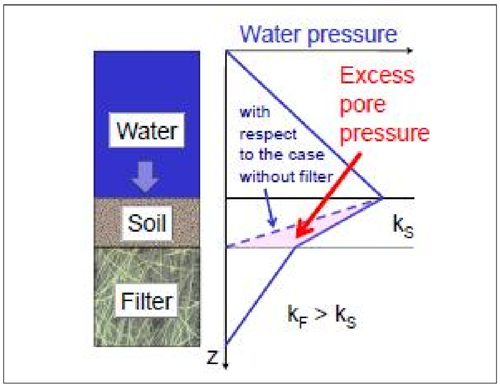



**Figure 2.16: Initial build-up of excess pore water pressure where geotextile is present** (Giroud, 2010)

When incorporating a geotextile, there initially is a build-up of pore water pressure between the soil to geotextile filter interfaces at the start of water flow through the system as shown in Figure 2.16. The excess pore water pressure is a function of the geotextile filter's permeability.

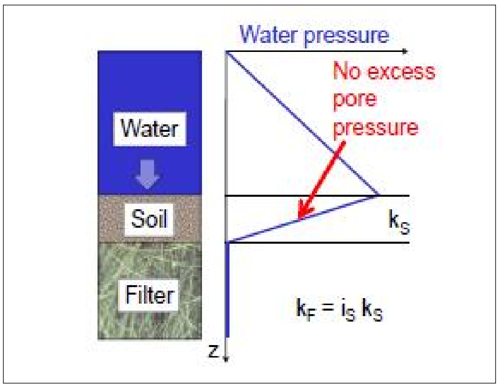



**Figure 2.17: Equilibrium flow conditions with geotextile present** (Giroud, 2010)

After some time, if the geotextile filter's permeability is sufficiently higher than that of the adjacent soil, the excess pore water pressure will be reduced to zero at the soil to geotextile interface (Figure 2.17), where:

$K_f$ = permeability of geotextile filter

$K_s$ = permeability of adjacent soil

$I_s$ = hydraulic gradient within soil

| Drainage Applications | Typical Hydraulic Gradient |
|---|---|
| Channel Lining | 1.0 |
| Standard Dewatering Trench | 1.0 |
| Vertical Wall Drain | 1.5 |
| Pavement Edge Drain | 1.0 |
| Landfill LCDRS | 1.5 |
| Landfill LCRS | 1.5 |
| Landfill SWCRS | 1.5 |
| Shoreline Protection | |
| Current Exposure | 1.0[(b)] |
| Wave Exposure | 10[(b)] |
| Dams | 10[(b)] |
| Liquid Impoundments | 10[(b)] |

**Table 2.1: Hydraulic gradient as measured within the soil** (Giroud, 2010)

Table 2.1 illustrates the typical hydraulic gradients present in soil drainage and filtration systems, depending on the application. This thesis evaluates the soil to geotextile filtration characteristics at hydraulic gradients of ≤1, which are commonly encountered in roadside dewatering trenches and edge drains.

### 2.11.2 The flow rate requirement

When a geotextile filter is introduced into a soil system, the flow rate of water is impeded and reduced, no matter how permeable the geotextile is. Calculations done with Darcy's equation show that the reduction in flow rate is less than 10% of the flow rate without a filter, barring the following conditions are met (Moraci 2008):

$\mathbf{k_F \geq k_S}$ for filter thickness 1 to 10 mm (for geotextile filters only)

Similarly,

$\mathbf{k_F \geq 25\ k_S}$ for filter thickness 250 mm to 2500 mm (for granular filters only).

### 2.11.3 Comparison of the two requirements (Giroud 2010):

For geotextile filters: $\mathbf{k_F \geq max\ (i_s.k_s\ ,\ k_s)}$ **(3)**

Similarly,

For soil filters: $\mathbf{k_F \geq max\ (i_s.k_s\ ,\ 25\ k_s)}$ **(4)**

In the case of geotextile filters, the flow rate requirement ($k_F \geq k_S$) may be more or less stringent, depending on the adjacent soil's hydraulic gradient, $i_s$. In the case of granular filters, the flow rate requirement ($k_F \geq 25\ k_S$) is generally more stringent than the pore pressure requirement ($k_F \geq i_s.k_s$), because the hydraulic gradient, $i_s$, is generally less than 25 (Giroud, 2010).

| Author | Permeability criterion |
|---|---|
| Schober & Teindl (1979) / Heerten (1981) | $K_g > k_s$ |
| Giroud (1982) | $K_s > 0.1k_s$ |
| Loudiere (1982) | $K_g > 10^2 k_s$ |
| Gourc (1982) | $K_g > 0.32k_s$ |

**Table 2.2 Review of main permeability criteria in unidirectional flow conditions**
(Moraci, 2008)

The trend in various different design criteria is to design the geotextile filter so that the long term permeability of the filter is at least one order of magnitude higher than the permeability of the base soil (Table 2.2).

The permeability criterion in terms of pore pressure requirements is generally verified for the geotextile filters, owing to their high permeability and limited thickness (Palmeira and Fannin, 2002).

Therefore, to satisfy the permeability criteria, the presence of a filter should not increase the pore water pressure at the soil geotextile interface. The flow rate was analysed between two soil layers, first with and then without a geotextile filter between the layers. If the flow rates differ less than 10%, then the geotextile filter is deemed to be satisfactory (Fannin, 2010). The long term permeability of the geotextile filter must be at least one order of magnitude higher than the permeability of the base soil (Fannin, 2010).

Furthermore, fluid flow through soils finer than course gravel is believed to be laminar (Mitchell and Soga, 2005). Equations have been developed to relate the hydraulic conductivity to the properties of the soil and permeating fluid.

Poiseuille's law for flow through a round capillary tube gives the average flow velocity $V_{ave}$ according to:

$$v_{ave} = \frac{\gamma_p R^2}{8\mu} i_h$$

where μ = viscosity of permeating fluid

$\gamma_p$ = unit weight of permeating fluid

R = tube radius

For a circular tube flowing full, Poiseuille's equation becomes:

$$q_{cir} = \frac{1}{2}\frac{\gamma_p}{\mu} R_H^2 i_h a$$

where a = the cross-sectional area of the tube.

For other shapes of cross-section, an equation of the same form will hold, differing only in the value of a shape coefficient Cs, yielding:

$$q = C_s \frac{\gamma_p R_H^2}{\mu} i_h a$$

For a bundle of parallel tubes of constant but irregular cross section contributing to a total cross sectional area A (solids plus voids), for which $S_0$ is the wetted surface area per unit volume of soil particles, then:

$$q = C_s\left(\frac{1}{S_0^2}\right)\left(\frac{\gamma_p}{\mu}\right)S^3\left(\frac{e^3}{1+e}\right)i_h A$$

By analogy with Darcy's law:

$$k_h = C_s\left(\frac{\gamma_p}{\mu}\right)\frac{1}{S_0^2}\left(\frac{e^3}{1+e}\right)S^3$$

for the case of full saturation (S = 1) and denoting Cs by ($1/k_0T2$), where:

$k_0$ = pore shape factor

T = tortuosity factor, the previous equation becomes:

$$K = k_h\left(\frac{\mu}{\gamma_p}\right) = \frac{1}{k_o T^2 S_o^2}\left(\frac{e^3}{1+e}\right)$$

The expression relates absolute or intrinsic permeability (K) to the fabric of the porous medium (void ratio, pore shape, and tortuosity), and is commonly termed the Kozeny-Carman equation. It explains the dependency of permeability on void ratio in uniformly graded sands and some silt (Mitchell and Soga, 2005).

The Kozeny-Carman equation also implies that the permeability of a soil is governed by the finer particles within the soil matrix, rather than the larger particles (Fannin *et al.*, 2006).

## 2.12 RETENTION CRITERION

The retention criterion is arguably the most important function of the filtration function (Giroud 1996). Soil retention does not require that the migration of all soil particles is prevented (Watson and John, 1999; Moraci, 1992). Soil retention only requires that the soil behind the geotextile filter remain stable. It is therefore important that the retention modes are understood, as described below.

### 2.12.1 Retention modes

Three types of soil retention mode (Fry, 2007) can be considered for filter applications:

#### *2.12.1.1 Total retention*

This retention mode is typically used on the downstream side of clay zones, as in earth dam construction, where the clay core should be kept intact by not losing too many fine particles due to internal erosion, piping, dispersion or cracking of the clay. These filters become progressively clogged, but in this case clogging is not detrimental because these filters are part of a system whose function is to retain water (Fry, 2007).

#### *2.12.1.2 Optimum retention*

These are typical filters used around structures such as drainage trenches and blankets in a variety of applications as outlined previously in the chapter. These filters should function for the design life of the structure and should remain permeable. Their function is to retain the soil as a whole, but not necessarily all the particles. This form of retention should exhibit a good balance between permeability and soil retention (Fry, 2007).

#### *2.12.1.3 Partial retention*

These filters are used in embankment protection systems where the geotextile is exposed to turbulent, intermittent and multi-directional flow. Such geotextile filters should remain unclogged to prevent instability in the case of rapid drawdown.

This thesis is exclusively devoted to filters providing optimum retention. These filters are assumed to be subject to non-turbulent flow in one direction and they are intended to function permanently. Total and partial retention are not considered.

Lafleur (1999) has proposed an approach that takes into account the rearrangement of particles within the interface zone. It is based on the retention ratio ($R_R$) as follows:

$R_R$ = Filtration Opening Size of the geotextile (FOS) / Indicative Size of the retained soil ($d_I$), where the FOS value is the $O_{95}$ value obtained from hydrodynamic sieving and $d_I$ is determined from the shape of the grading curve of the base (Lafleur *et al.*, 1989)

The $O_{95}$ value, typical of nonwoven geotextiles, is the opening size of which 95% of the openings are smaller.

Giroud (2010) shows that retention criteria depend on soil density and the soil particle size distribution curve which describes the coefficient of uniformity (Cu).

$$\mathbf{Cu = d_{60} / d_{10}}$$

While granular filters benefit to a certain degree from their thickness, geotextile filters on the other hand are thin, and the retention criterion must therefore take into account the internal stability of the soil (Moraci, 2002).

Internally stable soils have particles of a certain size that form a continuous skeleton. The successful performance of any soil to geotextile filtration system is the ability of the system to form an upstream reverse natural filter (Giroud, 2010).

The potential for grading instability increases as the width of the grading increases and the severity of seepage and vibration increases. Grading instability is described as the inability of the larger particles of a soil to hold back its smaller particles whilst water is flowing through it. This can usually be identified by studying and analysing the soil's particle size distribution curve. One might tend to think that grading instability would occur only in gap graded soils. Kenney and Lau (1984) and Scheurenberg (1986) have done experiments to prove that even well graded soils can prove to be unstable.

Earlier studies dating back to the 1930s were done on suffusion, which is the transportation of small particles from a soil. Research by de Mello (1975) and Sherard (1979) suggested that the potential of a material for 'elf-filtering' might be evaluated by estimating whether or not the soil's coarse fraction could act as a filter to retain the fine fraction.

Kenney and Lau (1985) did further studies on the influence on the shape of the grading curve on the internal stability of filter materials.

A series of permeability tests was conducted using material with $C_u$ ranging from 3 to 35 and particle sizes from 0.1 to 100 mm. The samples were vibrated to create flow conditions more severe than they would encounter in practice. Specimens were compacted to 80% relative density. By comparing normalised grading curves, a boundary line can be defined to separate stable from unstable gradings (Figure 2.18)

Sherard and Dunningan (1986) reviewed the findings above and found the results conservative, because the flow conditions do not represent flows experienced in real practice. Wolski (1987) suggested that a simple criterion of $C_u < 20$ may be used to define a stable grading. Widely graded filters are excluded, where the risk of becoming unstable is higher.

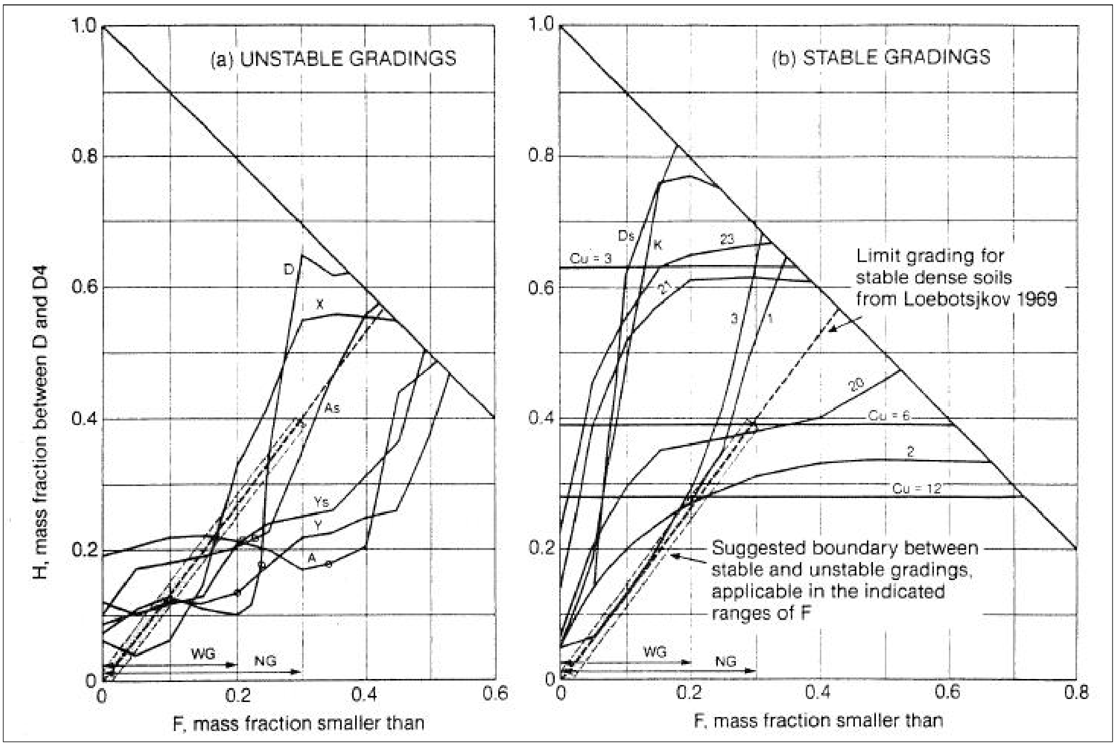



**Figure 2.18 Shape curves of stable and unstable gradings** (Kenney and Lau, 1984)

where:

D = soil particle grain size, D4 = 4 times soil particle size,
WG = widely graded ($C_u > 3$), NG = Narrowly graded ($C_u < 3$)

In an ideal situation, water flows together with suspended soil particles via a preferential flow path and under a certain hydraulic head towards the geotextile filter. The water will pass through perpendicular to the plane of the geotextile. Some of the finer suspended soil particles will pass through the geotextile and some will become lodged within the fibrous structure of the geotextile, assuming it is a nonwoven geotextile. Some particles will be blocked up against the upstream side of the geotextile, where the particle size is slightly bigger than the adjacent pore opening size of the geotextile.

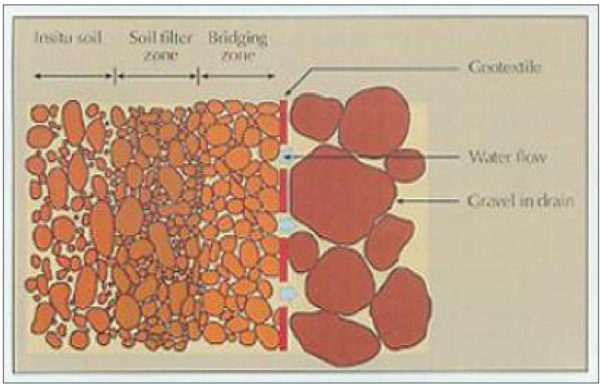



**Figure 2.19: Equilibrium soil conditions following formation of a soil filter** (Lawson, 1982)

After a time, as the system stabilizes, only bigger particles will be positioned immediately adjacent to the upstream side of the geotextile. These bigger particles will in turn hold back the smaller particles, which will in turn hold back even smaller particles (Figure 2.19). In this way, a stable reverse filter is formed, and under steady state flow conditions.

It should be noted that geotextiles for filtration functions do not perform as true filter fabric. They function for an indefinitely long time without becoming blinded or clogged. True filters have limited functionality as they would eventually become blinded or clogged by continually accumulated particles. Geotextiles function properly by restraining the soil and keeping it from being moved by water. Actually, the soil body being restrained by the fabric holds the soil into place (Giroud, 2010).

### 2.12.2 Size of soil skeletal particles

Internal stability depends on the co-efficient of uniformity ($C_u$). Soils that exhibit a co-efficient of uniformity of approximately 3 or less prove that particles are tightly packed together and the soil is considered internally stable (Figure 2.20). The closer the $C_u$ of the soil is to 1, the more uniformly graded the soil is. In this case there is no need for the soil to hold back any finer particles, which therefore makes the soil inherently stable.

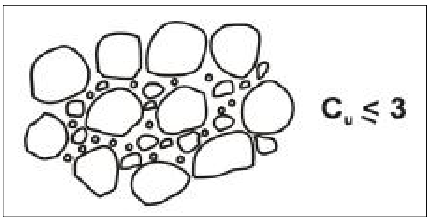



**Figure 2.20: Schematic representation of soil with Cu equal to or less than 3**

(Giroud, 2010)

If soils have co-efficient of uniformity of more than 3, it means that the coarsest particles are generally not in contact with the each other and are floating in the fine soil matrix, as seen in Figure 2.21, below. It also means that the coarsest particles do not have the ability to hold back the fine soil fraction and do not form a continuous skeleton that would entrap other particles.

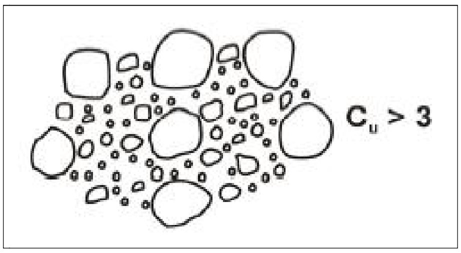
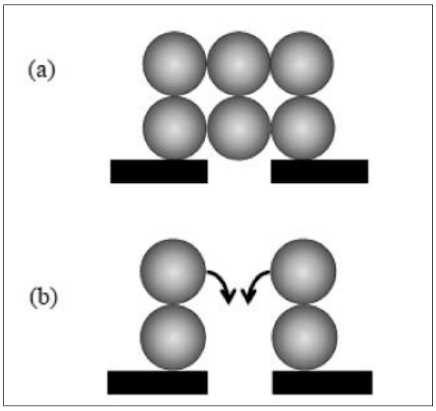



**Figure 2.21: Schematic representation of soil with a co-efficient of uniformity above 3** (Giroud, 2010).

Therefore as seen above, it is critical to know what the adjacent soil characteristics are, in order to help predict the soil's ability to form of a natural upstream reverse filter.

### 2.12.3 Maximum allowable geotextile filter opening size

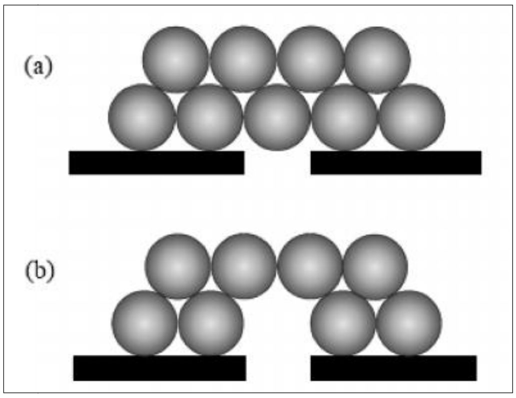
The maximum opening size of a filter to retain a soil skeleton of a certain size depends on the soil density. If the soil is in a loose state, all particles will pass through the filter if its opening size is larger than the particle size, as illustrated in Figure 2.22 below.




(a) cubic arrangement on a filter with openings as large as soil particles;
(b) most of the particles pass through the geotextile filter

**Figure 2.22: Schematic representation of a loose soil by a cubic arrangement** (Giroud, 2010)

If the soil is in a dense state, particles do not pass through the filter if the opening size is as large as the soil particle size (Figure 2.23). The internal stability therefore does not only depend on its co-efficient of uniformity but also its density (Giroud, 2010).




(a) hexagonal arrangement on a filter with openings as large as particles;
(b) formation of a stable bridge after one particle has passed through the filter

**Figure 2.23: Schematic representation of a dense soil by a hexagonal arrangement** (Giroud, 2010)

Mathematical analysis (not presented in this thesis) leads to the following equations representing the retention criterion (Giroud, 2010):

**For Cu ≤ 3**:

| | | |
|---|---|---|
| $\mathbf{O_F \leq (Cu)^{0.3} d_{85}}$ | for a loose soil | **(5)** |
| $\mathbf{O_F \leq 1.5(C_u)^{0.3} d_{85s}}$ | for a medium dense soil | **(6)** |
| $\mathbf{OF \leq 2(Cu)^{0.3} d_{85s}}$ | for a dense soil | **(7)** |

**For Cu ≥3**:

| | | |
|---|---|---|
| $\mathbf{O_F \leq 9 d_{85s}}$ / | $\mathbf{(Cu)^{1.7}}$ for a loose soil | **(8)** |
| $\mathbf{O_F \leq 13.5 d_{85s}}$ / | $\mathbf{(Cu)^{1.7}}$ for a medium dense soil | **(9)** |
| $\mathbf{O_F \leq 18 d_{85s}}$ / | $\mathbf{(Cu)^{1.7}}$ for a dense soil | **(10)** |

where:

$O_F$ = largest single opening size of geotextile filter

$C_u$ = co-efficient of uniformity of the soil

$d_{85}$ = particle size of soil fraction whereby 85% is smaller

## 2.13 POROSITY CRITERION

It is necessary, in the case of geotextile filters, to have a criterion to ensure that the number of filter openings is sufficient. This results in a porosity criterion (Giroud 2010).

Many geotextiles are so permeable that even if they have a small number of openings per unit area, they might still meet the permeability criterion. Therefore the permeability criterion is not sufficient to eliminate those geotextile filters that do not have enough openings.

The flow of liquids through a porous medium, such as granular soil or fibrous filters, takes place in channels. These channels are referred to as flow channels. The number of flow channels per unit area is greater in the soil than in a filter that meets the retention criteria for that soil. Therefore it can be expected that there will be a disturbance of flow at the soil-filter interface (Giroud 2010). For the purposes of this thesis, the filter will be the geotextile.

Disturbance at the soil-geotextile interface could cause displacement of fine soil particles in the vicinity of the filter, which could result in an accumulation of fine particles at the surface of the geotextile or inside the geotextile. Therefore the number of flow channels per unit area in the filter should be as large as possible, in order to minimise disturbance of the flow of liquid from the soil to the geotextile.

The number of openings per unit area for a granular filter is expressed by the following equation (Giroud 1996):

$$\boxed{N_o \approx \frac{0.1}{O_F^2}} \qquad \textbf{(11)}$$

The number of openings per unit area for a woven geotextile filter is expressed by the following equation (Giroud 2010):

$$\boxed{N_o = \frac{A_R}{O_F^2}} \qquad \textbf{(12)}$$

where $A_R$ = relative opening area of a woven geotextile.

$A_R$ is only measured in flat woven geotextiles, as it does not exhibit thickness such as the nonwovens have. The $A_R$ is measured by placing a woven geotextile is onto a light table; the percentage of light shining through the voids is then expressed as a percentage of the entire geotextile area. $A_R$ is synonymous with percentage open area (POA).

Therefore $A_R$ for woven geotextile filters:

$$A_R \geq 0.1 \tag{13}$$

Similarly, the lower and upper limit opening sizes of a non woven geotextile are expressed by the following equation (Giroud 2010):

$$\frac{\left(1-\sqrt{1-n}\right)^2}{O_F^2} \leq N_o \leq \frac{4\left(1+0.4n-\sqrt{1-n}\right)^2}{\sqrt{3}\,O_F^2} \tag{14}$$

where n = porosity of the nonwoven geotextile

Experimenting with a wide range of values of porosity, n, gives a conservative criterion, which ensures that the number of openings in the nonwoven geotextile is at least equal to the number of openings in a granular filter having the same opening size. The conservative criterion is that the porosity of a nonwoven geotextile should be equal to or greater than 0.55.

In summary the two criteria are (Giroud, 2010):

For woven geotextiles $A_R = 0.1$ (10%)

For nonwoven geotextiles $n \geq 0.55$ (55%)

Woven geotextiles with a relative opening area less than 0.1 should not be used as filters, as they pose a high risk of clogging (Giroud, 2010).

## 2.14 THICKNESS CRITERION

A soil particle that travels through a filter must go through passages called constrictions (Kenny and Lau 1985). In the case of a geotextile, a constriction is the passage between fibres. These are typical of nonwoven geotextiles. Woven geotextiles are not considered in this case, as they are usually thin. The size of a constriction is defined as the diameter of the largest sphere that can pass through the constriction as shown in Figure 2.24 below.

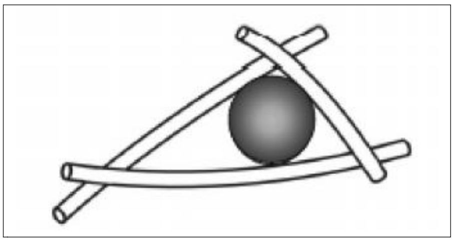


**Figure 2.24: Soil particles passing through a constriction** (Kenny and Lau 1985)

The soil particle that travels through a filter moves from one constriction to another, following a filtration path, which is identical to a flow channel. The soil particle will either pass through or be trapped, depending on the size of the constrictions along the filtration path. In practice, a soil particle can be stopped at the soil-filter interface or inside the filter, or can pass through the filter as shown in Figure 2.25 below.

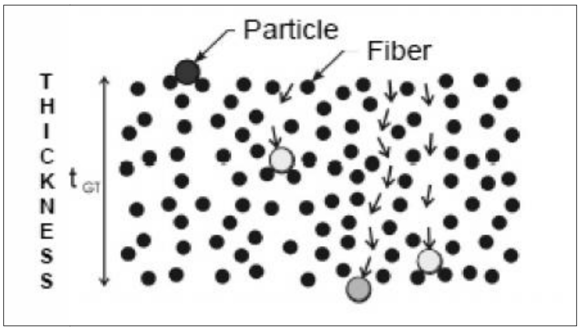


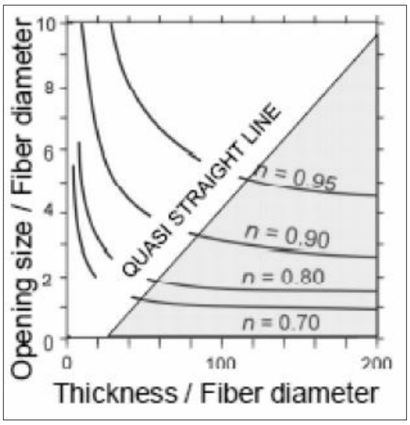

**Figure 2.25: Schematic cross section of a nonwoven geotextile** (Kenny and Lau, 1985)

This shows a particle stopped at the soil-geotextile interface, two particles trapped inside the geotextile filter and a particle passing through the geotextile filter (Kenny and Lau, 1985).The number of constrictions ($N_{constrictions}$) through a nonwoven geotextile filter is given by the following approximate equation (Kenney *et al.*, 1985):

$$N_{constrictions} \approx \frac{\mu_{GT}}{\rho_f \, d_f \sqrt{1-n}} \quad \textbf{(15)}$$

| | | | |
|---|---|---|---|
| where: | $\mu_{GT}$ | = | nonwoven geotextile mass per unit area |
| | ρf | = | fibre density of nonwoven geotextile |
| | df | = | fibre diameter of nonwoven geotextile |
| | n | = | porosity of nonwoven geotextile |

Studies by Kenney *et al*. (1984) have shown that beyond a geotextile thickness containing approximately 25 constrictions, the opening size is not significantly affected by changes in the geotextile thickness. In other words, a geotextile thickness that contains more than 25 constrictions is approximately an infinite thickness from the viewpoint of opening size.




Thickness, geotextile opening size and fibre diameter are all measured in mm.

**Figure 2.26: Zone of the graph with more than 25 constrictions** (Kenny and Lau, 1985)

To be reliable, a nonwoven geotextile filter should have a thickness that corresponds to at least 25 constrictions (Kenney *et al.* 1984). This concludes the explanation of the thickness criterion.

## 2.15 MODES OF FAILURE

Too often, when there is a failure in a drainage and filtration application where geotextiles are used, the first assumption is that the geotextile is the cause of such failure. In order to understand the cause of failure it is important to understand the mechanism of failure.

### 2.15.1 Base soil failure

The soil referred to as base soil is that soil which needs to be drained and is usually in intimate contact with the geotextile. As earlier discussed in this chapter, it is quite possible that the base soil is unstable and it is quite possible for the failure to occur internally as a result thereof. Too much fine material can be lost to seepage and under low hydraulic gradients these fine soil particles settle at the soil-geotextile interface. This can cause a phenomenon known as blinding, in the case of a nonwoven geotextile, and blocking in the case of a woven geotextile (Figures 2.27 to 2.30)

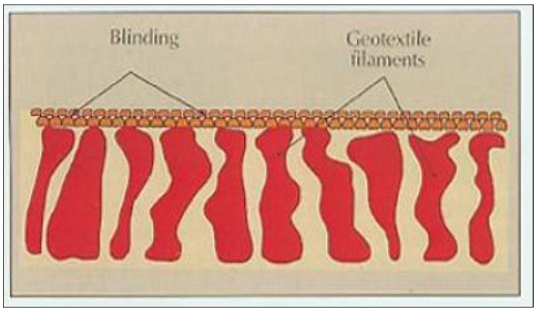



**Figure 2.27: Caked layers of fine particles blind a nonwoven geotextile** (McGown, 1982)

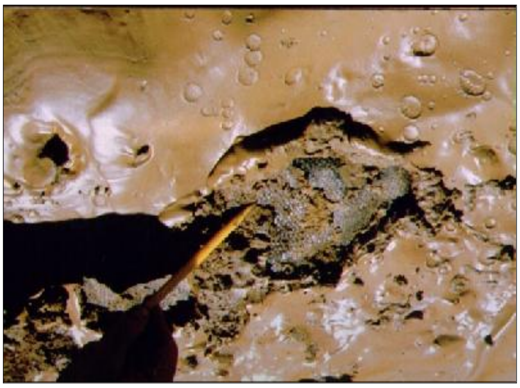


**Figure 2.28: Cake forming on a nonwoven geotextile also known as blinding** (Kaytech)

In Figure 2.28 above, when blinding occurs, the flow of water through the geotextile is impeded. This is not because the geotextile has clogged but due to the impervious silty soil layer which has settled on top or adjacent to the geotextile. If this cake layer is removed, as illustrated in Figure 2.29, quite often one would find that the geotextile is still functional.

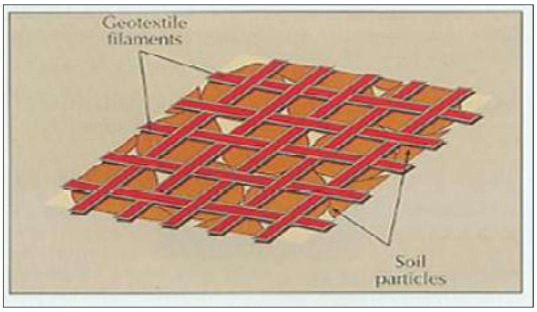



**Figure 2.29: Soil particles blocking and reducing the pore openings of a woven geotextile** (McGown, 1985)

**Figure 2.30: Woven geotextile showing openings being blocked by soil particles**
(Courtesy: Kaytech)

### 2.15.2 Physical Clogging

Physical clogging is a primary result of base soil characteristics. It is inevitable that, in the case of nonwoven geotextiles, a percentage of the fine particles in suspension will either pass through the geotextile into the drain or get trapped into the porometry of the geotextile. If the percentage of trapped particles is too high, the geotextile is said to be clogged (Figure 2.31).

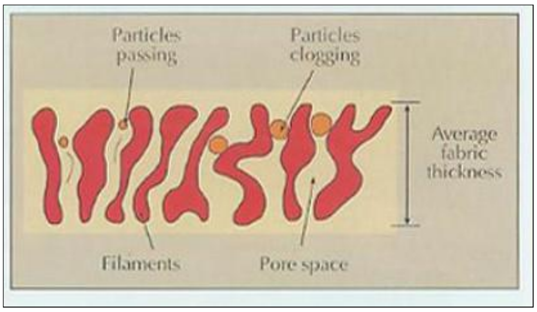



**Figure 2.31: Geotextile clogging** (Hoare, 1982)

Furthermore, clogging of a geotextile can also be in response to chemical, biological and physical mechanisms. The influence of stress level, pore fluid and integrity of the soil to geotextile contact also have an influence on the geotextile's clogging potential (Moraci, 2010).

### 2.15.3 Biological clogging due to permeating fluid

Biological clogging occurs in municipal solid waste landfills (Brune *et al.* 1991; McBean *et al.*1993) due to the flow through the geotextile of leachate. Giroud (1996) explains the biological process further below.

Bacteria reacting to an excess of nutrients (in the case of leachate that contains organic compounds) secrete fibrils of polysaccharide. In this case bacteria become entrapped in the gelatinous matrix formed by these entangled fibrils. The gelatinous matrix adheres to the surface of sand particles or geotextile fibres and a biofilm is formed. As bacteria continue to be supplied with nutrients, they reproduce and secrete more extracellular polysaccharide. The presence of the network of biofilm decreases the pore space available for flow and can cause clogging of filters with small openings, such as sands and geotextiles. Leachate therefore contains suspended and microbial contaminants which are subjected to varying degrees of temperature and availability of oxygen which result in either anoxic or aerobic conditions. A combination of all these can create a biological regime that could promote biological clogging of geotextiles (Figure 2.32).

**Figure 2.32: Ferric oxide (biological) clogging of a nonwoven geotextile**
(Courtesy: Kaytech)

Mendoca *et al.* (2003) reported that the anoxic/aerobic interface in drainage systems presents a favourable location for the development of iron bacteria and the formation of iron ochre biofilm on geotextile filters. Palmeira *et al.* (2008) also did laboratory permittivity tests on nonwoven geotextiles, using leachate from a waste containment facility. Reduced normal and in-plane permeability through the candidate geotextiles occurred as a result of biochemical clogging. Rapid reductions in permeability to unacceptable levels were observed in as little as 60 days. Normal permeability, in-plane hydraulic conductivity which is also known as transmissivity, and permittivity of nonwoven geotextiles are all interrelated as follows:

$$\boldsymbol{\Psi = K_n / t}$$

$$\boldsymbol{\theta = K_p . t}$$

where:

| | | |
|---|---|---|
| $\Psi$ | = | permittivity of the geotextile |
| $t$ | = | thickness of geotextile |
| $K_n$ | = | permeability normal to the plane of the geotextile |
| $K_p$ | = | permeability perpendicular to the plane of the geotextile |
| $\theta$ | = | transmissivity of the geotextile |

Different studies have shown the potential risk of clogging and blinding for the geotextile filters interacting with leachate (Cancelli & Cazzuffi 1987, Fourie *et al.* 1987, de Mendonca *et al.* 2003). Therefore the use of geotextile filters in waste disposal applications must be accurately evaluated by means of performance tests.

### 2.15.4 Stress levels

Knowledge of the vertical effective stress is important, since an increase of the vertical effective stress produces a decrease in soil porosity (n). In addition, an increase in vertical effective stress also involves a decrease in the pore size distribution in the geotextile filter, especially for needle-punched nonwoven geotextiles. Therefore, for a specific nonwoven geotextile, a vertical effective stress increase involves a decrease in porosity (n) that also produces a reduction of thickness (tgt) and of geotextile filtration opening size ($O_F$). The same effect was observed also by Palmeira and Gardoni (2002), using the bubble point method, relative to pore size distribution and filtration opening size $O_{95}$ values. For woven geotextiles, owing to the intrinsic structure of the material itself, an increase in vertical effective stress is not associated with a corresponding variation of the filtration opening size (Moraci, 2010).

Most geotextile filter design criteria do not consider the effect of the effective vertical stress, despite the fact that the increase in vertical effective stress involves a decrease in the filtration opening size of geotextiles. The influence of normal stress on the hydraulic characteristic of nonwoven geotextiles was studied, using a different experimental procedure, by Gardoni *et al.* (2000). Moreover, they also compared the test results with existing theoretical methods of predicting geotextiles' permeability. It was observed that even for rather large normal stresses the porosity and the permeability of the geotextile might still be greater than those values for typical sandy soils. The permeability coefficient normal to the geotextile plane can be reduced about 10 times in the range of pressures between 0 and 200 KPa.

#### 2.15.5 Type of contact

The continuity of soil to geotextile filter contact at the interface also plays an important role in the filter design. This continuity depends on the building procedure used, the density of the base soil and the stiffness of the geotextile filter. For instance, in the case of river bank revetments, the impact energy due to the placing of rip-rap blocks could produce large deformations in the base soil, if the latter is constituted of loose granular materials. In these cases, surface irregularities are generated in the base soil and the geotextile filters may follow these deformations, depending on their stiffness characteristics. For needle-punched nonwoven geotextiles, the adjustment occurs without large tensile stresses, and consequently without variations of filtration opening size. For woven geotextiles, the tensile stress becomes important and could induce changes in the filtration opening size whereby it becomes smaller.

### 2.16 LONG TERM DURABILITY OF GEOTEXTILES

Two phenomena have been identified as being responsible for the loss in mechanical strength of geotextiles:

a) Mechanical degradation of the fibres themselves and/or

b) Degradation of the bonds between the fibres, due to microbiological and chemical actions.

Degradation is a result of mechanical, environmental, chemical and bacterial actions. The polymeric structure alteration or destruction is a slow process unless the fibres are submitted to extreme conditions such as temperatures that increase the brittleness of the polymer. Any sharp objects or materials that could potentially tear or puncture the geotextile are also a factor.

### 2.16.1 Mechanical degradation

This commonly occurs during geotextile installation, when sharp edged materials are in direct contact, which could possible tear or puncture the geotextile.However, it is not limited to the installation process. Cyclic contact with a highly abrasive material can also erode and puncture fibres over time. When holes occur in the geotextile, the geotextile's ability to perform as a filter is reduced. Localised flow would occur and smaller particles could migrate through the geotextile and potentially become blocked and render the drain dysfunctional.

### 2.16.2 Microbiological degradation

Most polymers used today to manufacture geotextiles (polyesters and polyolefins) are resistant to microbiological attack (Rankilor, 1981). Polyamides are known to be attacked by mildew and bacteria. In the case of the latter it was observed that moulds, mildew and fungi were adhering to some of the finished coatings applied to the geotextile during the manufacturing process, but without attacking the fibres themselves. Ionescu *et al.* (1982) immersed 1 400 samples of six geotextiles in eight types of soil containing different bacterial media for five to seventeen months. Results showed no significant negative signs of bio-degradation and no significant reduction in mechanical properties. Biological activity is more likely to occur near the surface, rather than at the depth where geotextiles are often utilised.

### 2.16.3 Chemical degradation

Moncrieff (1975) has documented the performance and resistance of polymers with specific chemicals. Most geotextile polymeric fibres have high resistance to chemical degradation. Polyester can be degraded by strong alkalis, polyamides undergo hydrolysis and polypropylene oxidises. It is advisable to do accelerated laboratory testing for chemical compatibility for the specific application.

Troost and den Hoedt (1984) investigated the reaction of geotextiles made of polyester, polyamide and aramid, submersed for up to thirty months in solutions ranging from pH 5 to pH 9. None of the fabrics lost more than 10% of their inherent strength.

Further tests conducted by Halse *et al*. (1987a and 1987b), submersed geotextiles composed of polypropylene, polyvinyl chloride and polyester for 120 days in alkaline solutions ranging from pH 10 to pH 12. No significant loss of strength was observed for polypropylene and polyvinyl chloride, but a 53% loss of strength was observed for the polyester geotextile.

The above investigations have shown that, barring very aggressive conditions, polymers are generally resistant to chemical attack. Table 2.3 below shows the performance of different geotextile polymers when exposed to acidic and alkaline environments.

| Polymer Type | Resistant to | | Stable between (°C) | Remarks |
|---|---|---|---|---|
| | Acid Conditions | Alkali Conditions | | |
| Polypropylene | pH ≥ 2 | All | -15 to 120 | Attacked at elevated temperatures by hydrogen peroxide, sulphuric acid and nitric acids.<br>Weakened by certain solvents, e.g. diesel fuel.<br>Insignificant change in strength between 20°C and 35°C. |
| Polyester | pH ≥ 3 | pH ≤ 10 | -20 to 220 | Degrades by hydrolysis under strongly alkaline conditions. Therefore, concrete must not be cast directly against it.<br>Insignificant change in strength between 20°C and 35°C. |
| Polyamide (nylon 6.6) | pH ≥ 3 | pH ≤ 12 | -20 to 230 | Degrades by hydrolysis under strongly acidic conditions.<br>Reduces in strength by up to 30% when immersed in water or used in a saturated environment.<br>Insignificant change in strength between 20°C and 35°C. |
| Polyethylene | pH ≥ 2 | All | -20 to 80 | Same as polypropylene, except strength at 35°C is lower than that at 20°C by about 25%. |

**Table 2.3 Chemical and thermal stability of synthetic fibers used in the manufacture of geotextiles** (Cooke & Rebenfield (1988), Lawson & Curiskis (1985) and van Zanten (1986))

### 2.16.4 Environmental degradation

Environmental factors which could negatively affect the properties of unprotected or uncovered geotextiles are ultra-violet light, extreme temperatures, oxidation and polluted atmospheres. In general, the most frequent risk for an uncovered geotextile is ultra-violet exposure. Raumann (1982) reported outdoor exposure on a range of polypropylene and polyester geotextiles for up to thirty six weeks. All samples showed significant strength loss under outdoor exposure. Some samples lost all their strength in from sixteen to twenty four weeks.

Polypropylene geotextiles are most susceptible to sunlight exposure, although chemical stabilising agents have been added during the fibre manufacturing process. Polyester is known to melt at temperatures of around 260 degrees Celsius and polypropylene at about 180 degrees Celsius. The extreme temperatures around the world are well within an acceptable range and geotextile polymers will not degrade in the short term. During storage it is advisable to keep all geotextile in its original wrapping and stored under cover or under tarpaulins.

Geotextiles may also be placed near or in contact with radioactive waste. Van de Voorde (1972) indicated that the elasticity modulus of the polymers is not affected, but the elongation to break is decreased in all cases. The quantity of radiation required to reduce the elongation of some industrial polymers by 50% has been measured: $3 \times 10^4$ Gy for polypropylene, $5 \times 10^5$ Gy for polyethylene and $1 \times 10^8$ Gy. Gy stands for 'gray' which is the standard unit of measurement unit for absorbed radiation.

## 2.17 TESTS ON EXHUMED SAMPLES

A few studies have been carried out by examining samples of geotextiles buried for a number of years. Colin, Mitton *et al*. (1986) carried out experiments where geotechnical fabrics have been exposed to accelerated soil burial testing for up to seven years and the recovered fabrics then examined for changes using burst strength testing, optical microscopy and infrared spectroscopy. By analytical testing the test specimens were shown to be based on monofilaments of polypropylene, polyethylene terephthalate and a mixture of polypropylene and bi-component fibres respectively. None of the samples showed a significant decrease in burst strength beyond the deviation of the experimental data. No oxidation was observed.

Van Zanten and Thabet (1982) exhumed polypropylene, polyethylene and polyamide geotextiles that had been buried for ten years, from the canal banks of the Netherlands. They found the loss of strength and elongation to rupture respectively as 26% and 62% for polypropylene, 11% and 24% for the polyethylene, and 23% and 0% for polyamide. These results are somewhat of a contrast to those of Colin, Mitton *et al*. (1986). Van Zanten and Thabet (1982) could not ascertain whether the loss of strength in their research was due to environmental factors or installation damage.

The most significant of exhumed tests were commissioned by the French Ministry of Industry and reported upon by Sotton *et al*. (1982). Approximately two hundred samples consisting of polypropylene, polyester and polyester/polyamide were exhumed from over thirty sites in France. The geotextiles had been buried in various soil types for up to twelve years. Testing of exhumed samples included thickness, tensile strength, permittivity and fibre properties. Losses in tensile strength of up to 30% were observed. No changes in polymer fibres were observed to be due to soil conditions. No micro bacterial damage was observed upon visual inspection.

Based on the above studies, it can be deduced that geotextiles are very robust and durable against chemical attack and micro bacteria. Care should be exercised in leaving the geotextile exposed to factors such as direct sunlight and oxidative environments.

## 2.18 INSTALLATION CONSIDERATIONS

The successful performance of geotextile filters is also a function of their survivability during installation. It is desirable for soils to be relatively dry where geotextiles are to be installed. Care should also be taken not to damage the geotextile by contact with sharp objects or protrusions during installation.

Because of its relatively thin planar structure, geotextiles could be prone to rupture during installation, which might result in the opposite of the desired outcomes of the performance of the geotextile.

## 2.19 EVALUATION OF DRAINAGE SEVERITY

It is highly recommended that all drainage applications should start with an evaluation of how critical or severe the application is (Table 2.4). Geotextiles should not be selected on costs alone. The cost of a geotextile is often negligible in comparison to that of other components and actual construction cost of the drain itself.

| A. Critical Nature of the Project | | |
|---|---|---|
| Item | Critical | Less Critical |
| 1. Risk of loss of life and/or structural damage due to drain failure: | High | None |
| 2. Repair costs versus installation costs of drain: | > > > | = or < |
| 3. Evidence of drain clogging before potential catastrophic failure: | None | Yes |
| **B. Severity of the Conditions** | | |
| Item | Severe | Less Severe |
| 1. Soil to be drained: | Gap-graded, pipable, or dispersible | Well-graded or uniform |
| 2. Hydraulic gradient: | High | Low |
| 3. Flow conditions: | Dynamic, cyclic, or pulsating | Steady state |

**Table 2.4: Guidelines for evaluating the critical nature or severity of a drainage application** (Carroll, 1983)

## 2.20 INTERNATIONAL GEOTEXTILE FILTER DESIGN CRITERIA

Many filter criteria have been developed internationally as regards uni-directional flow in geotextiles.. Some of the more commonly researched criteria are highlighted in Table 2.4.

| Source | Criterion | Remarks |
|---|---|---|
| Bergado et al. (1992) | $O_{90}/D_{85} \leq$ 2 to 3<br>$O_{50}/D_{50} \leq$ 18 to 24 | Nonwovens, clay recommended |
| Ogink (1975) | $O_{90}/D_{90} \leq 1.8$ | Nonwovens, type of soil not specified |
| Carroll (1983) | $O_{95}/D_{85} \leq$ 2 to 3 | For both wovens and nonwovens, type of soil not specified |
| Christopher and Holtz (1985) | $O_{95} \leq 1.8\ D_{85}$<br>Steady state<br>$AOS < 0.3\ D_{85}$ | Nonwovens, for soils with greater than 50% particles passing the 75 µm sieve |
| Holtz and Christopher (1987) | For steady state<br>$O_{95} \leq 0.5, D_{85} \leq 0.3$ mm<br>For dynamic flow<br>$O_{50} \leq 0.5\ D_{85}$ | Nonwovens, for silts and clay |
| Calhoun (1972) | $O_{95}/D_{85} \leq 1$ | Suitable for geotextile filters with a high percentage of large pores |
| Chen and Chen (1986) | $O_{90}/D_{85} \leq$ 1.2 to 1.8<br>$O_{50}/D_{50} \leq$ 10 to 12 | |
| Sweetland (1977) | $O_{15}/D_{85} \leq 1$<br>$O_{15}/D_{85} \leq 1$ | Nonwovens, soils with $C_u = 1.5$<br>Nonwovens, soils with $C_u = 4$ |
| Rankilor (1981) | $O_{50}/D_{85} \leq 1$<br>$O_{50}/D_{50} \leq$ 25 to 37<br>$O_{15}/D_{15} \leq 1$ | Nonwovens, soils with $0.02 \leq D_{85} < 0.25$ mm<br>Nonwovens, cohesive soil<br>Nonwovens, soils with $D_{85} > 0.25$ mm |

**Table 2.4: Retention criteria based on previous research** (Bergado *et al*., 1996)

Although many researchers have derived their own interpretation of what the retention and permeability criteria should be, some countries have decided to adopt what has proved to be best practice for themselves for their local conditions.

## 2.21 REGIONAL GEOTEXTILE FILTER DESIGN CRITERIA

Furthermore, various countries have adopted geotextile filter design criteria as follows:

### 2.21.1 Dutch practice (after N.W.M. John, 1989):

For static unidirectional flow, originally $O_{95} < d_{90}$ for wovens, and $O_{90} < 1.8d_{90}$ for nonwovens, both of these are released by the Dutch Coastal Works Association to $O_{90} < 2d_{90}$.

Where:

$O_{90}$ = opening size of geotextile where 90% are smaller

$d_{90}$ = particle size of soil where 90% are smaller

### 2.21.2 German Practice (after NWM John, 1989):

| Soil Description | Geotextile Criteria |
|---|---|
| $d_{40}$<0.06mm, stable soil | $O_{95}<10d_{50}$ and $O_{95}<2d_{90}$ |
| $d_{40}$<0.06mm, problem soil | $O_{95}<10d_{50}$ and $O_{95}<d_{90}$ |
| $d_{40}$>0.06mm, stable soil | $O_{95}<5d_{10}.Cu^{0.5}$ and $O_{95}<2d_{90}$ |
| $d_{40}$>0.06mm, problem soil | $O_{95}<5d_{10}.Cu^{0.5}$ and $O_{95}<d_{90}$ |

**Table 2.5: German practice in terms of geotextile filter criteria** (NWM John, 1989)

where:

$d_{40}$ = particles size of soil whereby 40% is smaller

$O_{95}$ = characteristic pore size of the geotextile

and where problem soils are defined as those falling into any one of the following categories:

i) Fine-grained soils with a plasticity index of less than 15%

ii) Soils whose average particle size (d50) lies between 0.02 and 0.1mm

iii) Soils with a coefficient of uniformity of less than 15, which also contains clay or silt-sized particles

### 2.21.3 American Practice (after N.W.M. John, 1989):

| Soil Description | Geotextile Criteria |
|---|---|
| $d_{50}$>0.075mm | 0.297mm $\leq O_{95} \leq d_{85}$(wovens)<br>0.297mm $\leq O_{95} \leq 1.8d_{85}$(nonwovens) |
| $d_{50}$<0.075mm | |
| $Cu \leq 2$ | $O_{95} \leq d_{85}$ |
| $2 \leq Cu \leq 4$ | $O_{95} \leq 0.5Cu.d_{85}$ |
| $4 \leq Cu \leq 8$ | $O_{95} \leq 8.d_{85}/Cu$ |
| $Cu \geq 8$ | $O_{95} \leq d_{85}$ |

**Table 2.6: American practice in terms of geotextile filter criteria** (NWM John, 1989)

### 2.21.3 French Practice (after N.W.M. John, 1989):

These criteria recognise the base soil's coefficient of uniformity (U); soil density and hydraulic gradient (i).

| Soil Description | Geotextile Criteria |
|---|---|
| Well Graded (Cu > 4) and dense | $4d_{15} \leq O_f \leq 1.25d_{85}$ |
| Well Graded (Cu > 4) and loose | $4d15 \leq O_f \leq d_{85}$ |
| Uniformly Graded (Cu≤4) and dense | $O_f \leq d_{85}$ |
| Uniformly Graded (Cu≤4) and loose | $O_f \leq 0.8d_{85}$ |

**Table 2.7: French practice in terms of geotextile filter criteria** (NWM John, 1989)

where:

$d_{15}$ = particle size of soil whereby 15% is smaller

$O_f$ = geotextiles characteristic pore size as measured by the French AFNOR 38017 (Wet Sieve Test Method)

When the hydraulic gradient (i) in the vicinity of the geotextile lies between 5 and 20, then the geotextile pore sizes specified above should be reduced by 20%. Similarly, if it exceeds 20, or reversing flow conditions are present, then the pore size should be reduced by 40% (Table 2.7).

### 2.21.4 English Practice (N.W.M. John, 1989):

| **Minimum size of soil particle to be positively restrained** | **Maximum value for $O_{95}$** |
| --- | --- |
| $d_5$ | $d_{50}Cu'^{-0.9}$ |
| $d_{15}$ | $d_{50}Cu'^{-0.7}$ |
| $d_{50}$ | $d_{50}$ |
| $d_{60}$ | $d_{50}Cu'0.2$ |
| $d_{85}$ | $d_{50}Cu'0.7$ |
| $d_{90}$ | $d_{50}Cu'0.8$ |
| $d_{95}$ | $d_{50}Cu'0.9$ |

**Table 2.8: English practice in terms of geotextile filter criteria** (NWM John, 1989)

English Practice is based on the principle that if a characteristic particle size is retained, a reverse filter will form. Even for a broadly graded soil, having a higher coefficient of uniformity ($C_u$), a reverse filter will form (Table 2.8).

Arguably, one of the most comprehensive geotextile filter design methods developed was based on work done by Luettich, Giroud and Bachus (1993). The research brought together already developed design concepts and filter design criteria into a comprehensive nine-step design methodology. These nine steps are listed as follows (Luettich *et al.,*1993):

*Step 1: to define the application filter requirements*

This is in order to understand what the requirements are for the filter for the intended application. Furthermore it also involves the type of drainage material that will be used adjacent to the geotextile filter.

*Step 2: defining boundary conditions*

This stage evaluates the confining pressure in the vicinity of the geotextile filter. Flow conditions are further defined as either uni-directional or dynamic. This thesis evaluates only unidirectional flow conditions.

*Step 3: determining soil retention requirements*

A chart which provides numerical retention criteria was developed in terms of uni-directional flow (Table 2.9). This chart takes into account the soil's particle size distribution, Atterberg limits, dispersivity and density. It also correlates these aforementioned parameters to a candidate geotextile's pore opening size ($O_{95}$).

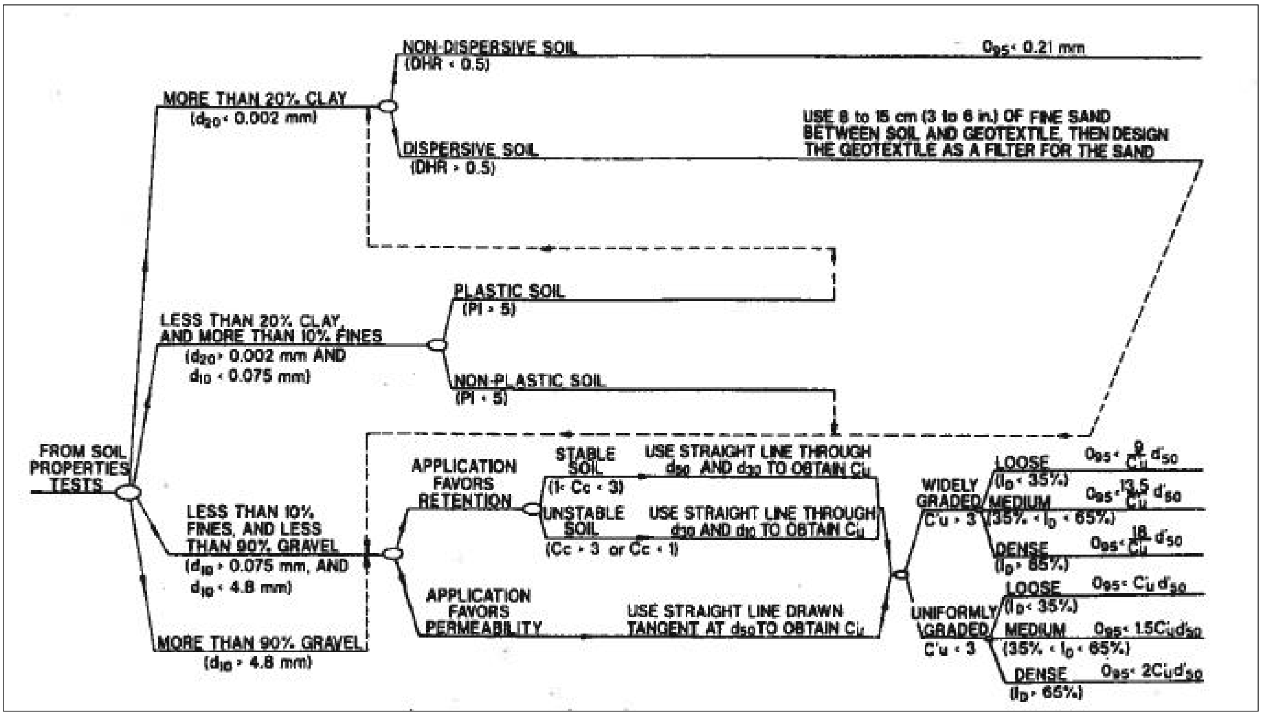



**Table 2.9: Geotextile filter criteria under uni-directional flow conditions** (Luettich *et al*., 1992)

where $O_{95}$ = pore size of geotextile where 95 per cent are smaller.

*Step 4: determining geotextile permeability requirements*

Based on the soil's permeability, the geotextiles permeability requirements can be determined. The minimum allowable geotextile permeability can then be calculated (Giroud, 2010):

$$\mathbf{k_g > i_s k_s}$$

where:

$k_g$ = permeability of the geotextile

$i_s$ = hydraulic gradient

ks = permeability of the soil

*Step 5: determining anti-clogging requirements*

To minimise the risk of clogging the following criteria should be met (Giroud 2010):

- Use the largest opening size ($O_{95}$) that satisfies the retention criteria.
- For nonwoven geotextiles, use the largest porosity (n) available, but not less than 30%
- For woven geotextiles, use the largest open percentage area ($A_R$) available, but not less than 4%

*Step 6: determining survivability requirements*

As previously mentioned, construction techniques and the type of drainage aggregate placed adjacent to the geotextile, have a potential damaging effect on it. Minimum strength indices should be specified to mitigate risk of damaging the geotextile (Table 2.10).

| | | GRAB STRENGTH (LBS) | ELONGATION (%) | SEWN SEAM STRENGTH (LBS) | PUNCTURE STRENTH (LBS) | BURST STRENTH (LBS) | TRAPEZOID TEAR (LBS) |
|---|---|---|---|---|---|---|---|
| SUBSURFACE DRAINAGE | HIGH CONTACT STRESSES (ANGULAR DRAINAGE MEDIA) (HEAVY COMPACTION) or (HEAVY CONFINING STRESSES) | 247 | < 50% * | 222 | 90 | 392 | 56 |
| | | 157 | ≥ 50% | 142 | 56 | 189 | 56 |
| | LOW CONTACT STRESSES (ROUNDED DRAINAGE MEDIA) (LIGHT COMPACTION) or (LIGHT CONFINING STRESSES) | 180 | < 50% * | 162 | 67 | 305 | 56 |
| | | 112 | ≥ 50% | 101 | 40 | 138 | 40 |
| ARMORED EROSION CONTROL | HIGH CONTACT STRESSES (DIRECT STONE PLACEMENT) (DROP HEIGHT > 3 FT) | 247 | < 50% * | 222 | 90 | 392 | 56 |
| | | 202 | ≥ 50% | 182 | 79 | 247 | 79 |
| | LOW CONTACT STRESSES (SAND OR GEOTEXTILE CUSHION) and (DROP HEIGHT < 3 FT) | 247 | < 50% * | 222 | 90 | 292 | 56 |
| | | 157 | ≥ 50% | 142 | 56 | 189 | 56 |

**Table 2.10: Survivability strength requirements** (AASHTO, 1986)

*Step 7: Determining durability requirements*

Reference is made to section 2.16 of this thesis.

*Step 8: miscellaneous design considerations*

Other geotextile filter design considerations, in no specific order:

- Intimate contact between the soil and geotextile
- Abrasion of the geotextile due to dynamic water action
- Extrusion of fine soil particles through geotextile when exposed to high confining pressures
- Intrusion of the geotextile into stone drainage layer
- Geotextile structure
- Biological and bio-chemical clogging
- Safety factors

*Step 9: selecting a geotextile filter*

The final step is to select a geotextile filter using the required material properties. Any geotextile which meets these properties, irrespective of type, can be chosen as a filter. For some applications, these properties are to be verified through third party conformance testing.

## 2.22 AASHTO M288 GEOTEXTILE CRITERIA

In 1982, an American task force of representatives of the geotextile industry, private contractors, and state and federal transportation agencies reviewed tables of suggested geotextile property values for the Federal Highway Association Geotextile Manual that was being prepared at the time. The resulting work was published in the AASHTO Specification Book as Specification M-288 on Geotextiles. It was established that geotextiles need to have a strength criteria, irrespective of their application (see Table 2.11). Geotextiles are classified into different classes according to their strength. The geotextiles are also classified in terms of their elongation characteristics. High elongation geotextiles (>50%) are normally classified under the nonwoven group and low elongation (<50%) are usually classified under woven geotextiles. The elongation of the geotextile is the strain measured at rupture, expressed as a percentage, using standard test methods.

| | | | Geotextile Class | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Class 1 | | Class 2 | | Class 3 | |
| | | | Elongation | | Elongation | | Elongation | |
| | **Test Methods** | **Units** | **<50%** | **≥50%** | **<50%** | **≥50%** | **<50%** | **≥50%** |
| Grab Strength | ASTM D 4632 | N | 1400 | 900 | 1100 | 700 | 800 | 500 |
| Sewn Seam Strength | ASTM D 4632 | N | 1260 | 810 | 990 | 630 | 720 | 450 |
| Tear Strength | ASTM D 4533 | N | 500 | 350 | 400 | 250 | 300 | 180 |
| Puncture Strength | ASTM D 4833 | N | 500 | 350 | 400 | 250 | 300 | 180 |
| Burst Strength | ASTM D 3786 | kPa | 3500 | 1700 | 2700 | 1300 | 2100 | 950 |

**Table 2.11: Geotextile strength requirements** (AASHTO M288)

Furthermore, a table was developed relating to a geotextile to be used as a filter in subsurface drainage applications (Table 2.12). Note that class 2 geotextiles, in terms of strength, are recommended across the spectrum.

| | | | **Requirements** | | |
|---|---|---|---|---|---|
| | | | **Percent silt and clay (<0.075 mm)** | | |
| | **Test Methods** | **Units** | **<15** | **15 to 50** | **>50** |
| Geotextile Class | | | | Class 2 | |
| Permittivity | ASTM D 4491 | $sec^{-1}$ | 0.5 | 0.2 | 0.1 |
| Apparent Opening Size | ASTM D 4751 | mm | 0.43 max. avg. roll value | 0.25 max. avg. roll value | 0.22 max. avg. roll value |
| Ultraviolet Stability (Retained Strength) | ASTM D 4355 | % | | 50% after 500 hrs of exposure | |

**Table 2.12: Subsurface drainage geotextile requirement** (AASHTO M288)

## 2.23 SOUTH AFRICAN DESIGN GUIDELINES

Other than a few manufacturers' recommendations, the standards and specifications for geotextiles in South Africa are limited and out of date. A SABS code of practice for the testing of geotextiles exists (SANS 10221:2007), which lists five in-isolation tests, serves as a means to classify geotextiles. The code furthermore defines a list of geotextile grades, but does not provide input into the actual design process nor does it consider the soil to geotextile interface performance (Table 2.13).

| Test | Unit | GRADES | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Thickness | mm | As specified by Manufacturer | | | | | | | | | |
| Mass per unit area | $g/m^2$ | 100 | 100 | 135 | 135 | 200 | 200 | 250 | 250 | 300 | 300 |
| Penetration load (CBR) | kN | 0.8 | 0.8 | 1.2 | 1.2 | 2 | 2 | 2.4 | 2.4 | 3 | 3 |
| Tensile Strength | kN/m | 6 | 6 | 8 | 8 | 12 | 12 | 16 | 16 | 22 | 22 |
| Elongation | % | 50 | 10 | 50 | 10 | 50 | 10 | 50 | 10 | 50 | 10 |
| Water Permeability | $l/s.m^2$ | 130 | 20 | 130 | 20 | 100 | 15 | 70 | 10 | 40 | 5 |

**Table 2.13: Geotextile drainage and filtration specification** (SABS 1200)

*TRH15:1994 - Subsurface Drainage for Roads* and *COLTO: 1998 Section 2100 – Drains* aims to address some of the design inputs that are required. These specifications may be deemed to follow most of the design requirements but fall short, as they do not reflect the current state of practice and the possible performance of geotextiles is ignored. Referring to Table 2.13 as in COLTO 1988 below, no mention is made of geotextile pore size and it also does not take geotextile to soil interaction into account. The geotextiles, although for drainage and filtration, are classified according their mechanical characteristics, whereas reference to their hydraulic characteristics would be more appropriate (Table 2.14).

| Property | GRADE | | Test Method |
|---|---|---|---|
| | 2 | 3 | |
| **Penetration Load (minimum), N** | 2400 | 1500 | 3.5 of SABS 0221-1988 |
| **Puncture Resistance (maximum), mm** | 26 | 32 | Clause 8114 |
| **Water Percolation (minimum), l/m$^2$/s** | 20 | 20 | 3.7 of SABS 0221-1988 |

**Table 2.14: Geotextile classification for drainage and filtration** (COLTO, 1988)

As an example of the shortcomings, one should note that the geotextile strengths are specified at 10% elongation, while geotextiles are known to function beyond this limit and are particularly of benefit where large deformations must be accommodated. In view of these shortcomings, the intention is to adopt international specifications rather than trying to update existing specifications. SABS is a participating body of the International Organization for Standardization (ISO). This provides the opportunity to adopt ISO standards and provide them as South African National Standard (SANS) at relatively small cost. SABS has recently formed *Working Group 5: Geotextiles* under *Sub Committee 59J*: *Geosynthetics,* to investigate the ISO standards and to adopt them in a structured fashion.

## 2.25 CONCLUSION

This chapter has reviewed and discussed the different polymers and types of commercially available geotextiles available in modern times. Their characteristics and their expected performance as filtration media were discussed. Examples of geotextiles as filters were illustrated, as well as the theoretical mechanisms of filtration. Further review were done on the important criteria needed for geotextiles to function as filters, with reference to the theory of natural soil filters. These criteria are permeability, retention, porosity and thickness. Together, going forward, these criteria would form a coherent set of criteria to allow for the safe design of geotextile filters. Long term durability of geotextiles, as well as various filter criteria, were discussed.

Soils are infinitely different and so are their behaviour. It is good to bear cognisance of the theories and criteria presented. When it comes to geotextile filter design, it is not to say that these theories will present a definite answer to all soil types. The research contained in this thesis is assessing only Berea sand's compatibility with various commercially available geotextiles commonly found in South Africa.

The aforementioned criteria given in this chapter are set to give a broad description of the expected behaviour of soil in various category envelopes, but are not confined to a single soil type, which in this case is the sands of the Berea formation. It is therefore critical, depending on the nature of the project, to assess the filter criteria of each soil type in isolation, and also to carry out further laboratory testing to substantiate the theory in every case.

The next chapter will discuss the test method used for conducting the research, as well as the engineering properties of the candidate soils and geotextiles. A computer spreadsheet predictive model for the comparison of laboratory tests will also be briefly discussed.

# Chapter 3 - Method

## 3.1 INTRODUCTION

The purpose of the laboratory study was to evaluate and report on the behaviour of three different types of Berea sand found along the KwaZulu-Natal coastline when in intimate contact with four commercially available geotextiles. Test outcomes would assist in determining the following:

a) The clogging potential of the soil to geotextile system.
b) How the soil's co-efficient of uniformity (Cu) influences the selection of the geotextile filter.
c) The evaluation of a geotextile type in terms of its fabrication.
d) The design methods available that can be considered in a South African context.

In addition, a computer prediction model was incorporated into the study in order to compare its results with the laboratory results. Furthermore, in this chapter, the test methodology and engineering properties of the materials used in this research are discussed. The computer spreadsheet used for predictive modeling is also referred to during the latter part of this chapter.

## 3.2 RESEARCH DESIGN

Based on previous research done on soil to geotextile compatibility testing, it was decided to carry out laboratory testing according to internationally accepted ASTM D5101, *Standard Test Method for Measuring the Soil-Geotextile System Clogging Potential by the Gradient Ratio* also known as the long term gradient ratio test (LTGRT). The LTGRT has been developed by the US Army Corps engineers. The LTGRT measures the performance of soil to geotextile permeability systems in cohesionless soils under unidirectional flow conditions. It is designed to detect clogging, if any, of soil to geotextile systems. The general assumption of the gradient ratio test is that a geotextile clogging level can be inferred from the gradient ratio of hydraulic gradients in different zones of the soil to geotextile system. The test apparatus is illustrated in Figure 3.1.

## 3.3 TESTING FACILITIES

The LTGRT was carried out at Geolaboratory in Durban, South Africa. Geolaboratory is a third party quality control geosynthetics testing facility that carries out both internal and external commercial testing. The apparatus at Geolaboratory is illustrated in Figure 3.2. The apparatus measures approximately 1.5 m in length, 0.8 m in depth and 2 m in height. A total of eight test permeameters can be operational simultaneously. To accommodate both research and commercial testing it was decided to occupy only four permeameters at any given time

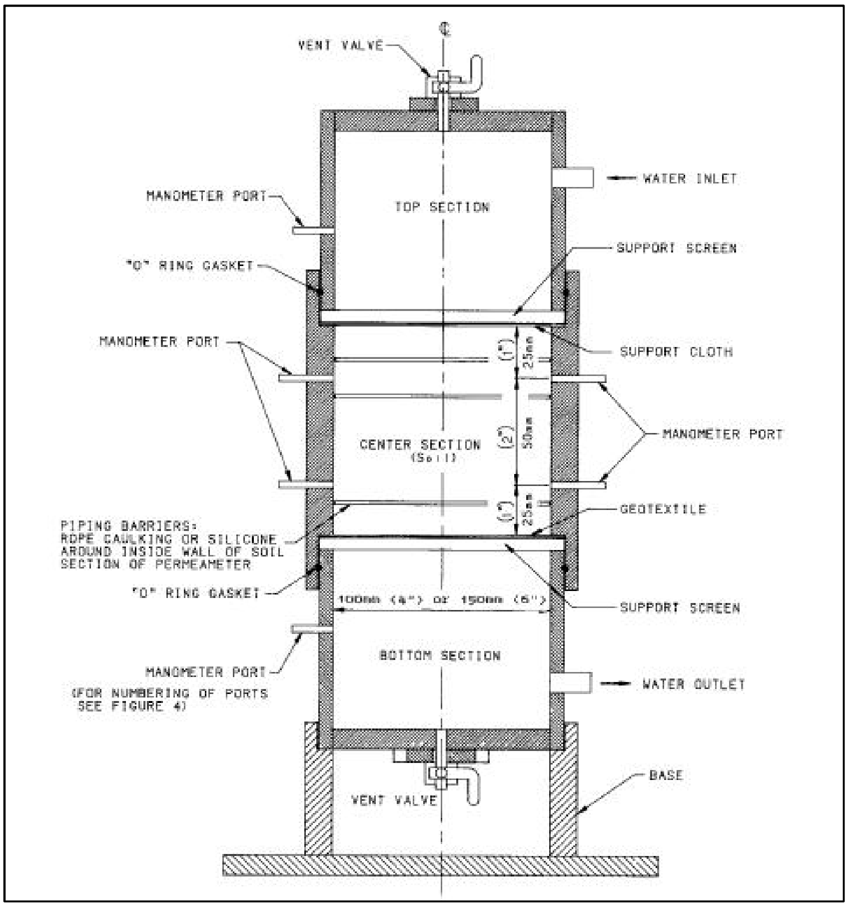



**Figure 3.1: LTGRT apparatus: ASTM D5101-01**

It was noted that there were slight modifications in the apparatus used at Geolaboratory compared to that described in the ASTM D5101 method. The main differences are highlighted as follows:

The positioning of the inlet and outlet of the apparatus at Geolaboratory, which were situated directly at the top and bottom, respectively, of the perspex cylinders, as opposed to positioning them at right angles, as described in ASTM D5101

The Geolaboratory apparatus had seven standpipes of 8 mm diameter connected to one side of the perspex cylinder only. The ASTM method describes the apparatus as having only six standpipes.

The Geolaboratory apparatus can accommodate a soil sample of 200 mm in depth, as opposed to 100 mm depth required by the ASTM D5101 method.
The apparatus in Geolaboratory did not have any vent valves positioned at the top and the bottom of the permeameter.

The differences in the apparatus illustrated above should not have a negative effect on the end results.

**Figure 3.2: Long term gradient ratio test apparatus** (Geolaboratory, Durban)

## 3.4 TEST MATERIAL ENGINEERING PROPERTIES

### 3.4.1 Geotextile properties

For the purpose of this research the names of the manufacturers were omitted and generic nomenclature was used. The geotextiles all weighed between 110 $g/m^2$ and 130 $g/m^2$. The geotextiles used for the research were as follows:

#### *3.4.1.1 Woven slit film polypropylene geotextile (W-SLF-PP)*

These geotextiles are made from long strips of polypropylene film, which are laid flat during a mechanical weaving process. The strips are laid closely together and as a result there are only limited openings in the fabric.

#### *3.4.1.2 Nonwoven continuous filament needle-punched polyester geotextile (NW-N-CF-PET).*

Continuous filament geotextiles are produced from fibres that are drawn from melted polymer that is extruded through spinnerets. The fibres are continually extruded, drawn, cooled and distributed to form a continuous web. The web is bonded by a needle punching process. This type of geotextile is thicker in comparison with a woven or thermally bonded geotextile.

#### *3.4.1.3 Nonwoven needle punched staple fibre polypropylene geotextile with thermal after treatment (NW-N-SF-PP)*

Staple fibre nonwovens are manufactured from fibres of short lengths ranging from approximately 20 mm to 150 mm. The fibres are mixed and laid by carding machines or pneumatic web forming systems. There is a preferential orientation of the fibres, which has an influence on the isotropic strength of the geotextile.

#### *3.4.1.4 Nonwoven and heat bonded continuous filament polypropylene geotextile (NW-HB-SF-PP).*

Thermally bonded non-woven geotextiles are manufactured by spraying polymer filaments onto a moving belt which is then passed through heated rollers. These heated rollers compress the layer of loose filaments and cause partial melting of the polymer filaments, which leads to thermal bonding at the filament cross-over points. The random distribution of the filaments as they are sprayed on to the belt ensures that the geotextile contains a wider range of opening sizes than is found in a woven geotextile. As there is no preferred orientation of the filaments during production, such as the warp and weft directions present in geotextiles, better isotropic strength is achieved. Thermally bonded geotextiles are relatively thin in comparison with geotextiles which do not have thermal treatment. The generic specifications of the specimen geotextiles used in the research are listed in Table 3.1.

| Properties | Notes | Unit | W-SLF-PP | NW-N-CF-PET | NW-N-SF-PP | NW-HB-CF-PP | Test Method |
|---|---|---|---|---|---|---|---|
| **Mass** | Nominal | $g/m^2$ | 120 | 130 | 130 | 110 | ASTM D5261 |
| **Thickness** | Under 2kPa | mm | 0.7 | 1.4 | 0.8 | 0.43 | ASTM D5199 |
| **Throughflow** | @50mm head | $l/s/m^2$ | 20 | 125 | 70 | 30 | ASTM D4491 |
| **Permeability** | @50mm head | m/s ($10^{-4}$) | 2.8 | 4 | 0.07 | 2.9 | ASTM D4491 |
| **Permittivity** | | $s^{-1}$ | N/A | 2.7 | 1.4 | 0.6 | ASTM D5199 |
| **Transmissivity** | | $m^2/s$ | N/A | 8.9E-6 | N/A | | ASTM D4716 |
| **Tensile strength** | Average | kN/m | 19 | 6 | 10 | 7.3 | ASTM D4595 |
| **Elongation at break** | | % | 16 | 60 | >45 | 52 | |
| **Puncture Resistance (CBR)** | Penetration load (50mm) | N | 2600 | 1300 | 1700 | 1100 | ASTM D6241 |
| **Trapezoidal Tear Strength** | Average | N | 300 | 240 | N/A | 290 | ASTM D4533 |
| **Grab Strength** | Average | N | 525 | 500 | N/A | 625 | ASTM D4632 |
| **Burst Strength** | | MPa | N/A | 1.5 | N/A | N/A | ASTM D3786 |
| **Pore Size** | O95 | µm | 670 | 205 | 85 | 140 | ASTM D4751 |
| **UV Resistance** | | | | | | | AS 3706.11 |

**Table 3.1: Geotextile specifications as on manufacturer's data sheets.**

where:

$g/m^2$ = grams per square meter
mm = millimetres
$\ell/s/m^2$ = litres per second per square meter
m/s = meters per second
$s^{-1}$ = seconds to the negative power of one
$m^2/s$ = square meters d per second
kN/m = kilo Newton per meter
N = Newton
MPa = Mega Pascal
µm = Micrometers

### 3.4.2 Soil properties

For the purpose of the investigation, three different soil samples were used that were all classified under the Berea sand formation. The main varying property was the plasticity index (PI), according to which the first and second soil samples were non-plastic (PI=0) and the third soil sample had a PIs of 7%.

Field samples were collected at various locations along the Durban coastline by Geosure (Pty) Ltd, a geotechnical laboratory in Durban. Soil grading tests were compared for control between Geosure geotechnical laboratories and Geosynthetic Laboratory in Pinetown. The grading analysis results are illustrated in Figure 3.3. Full soil indicator laboratory tests were carried out on the three Berea soil samples according to the TMH1 method A1 - A3 and results are shown in Table 3.2.

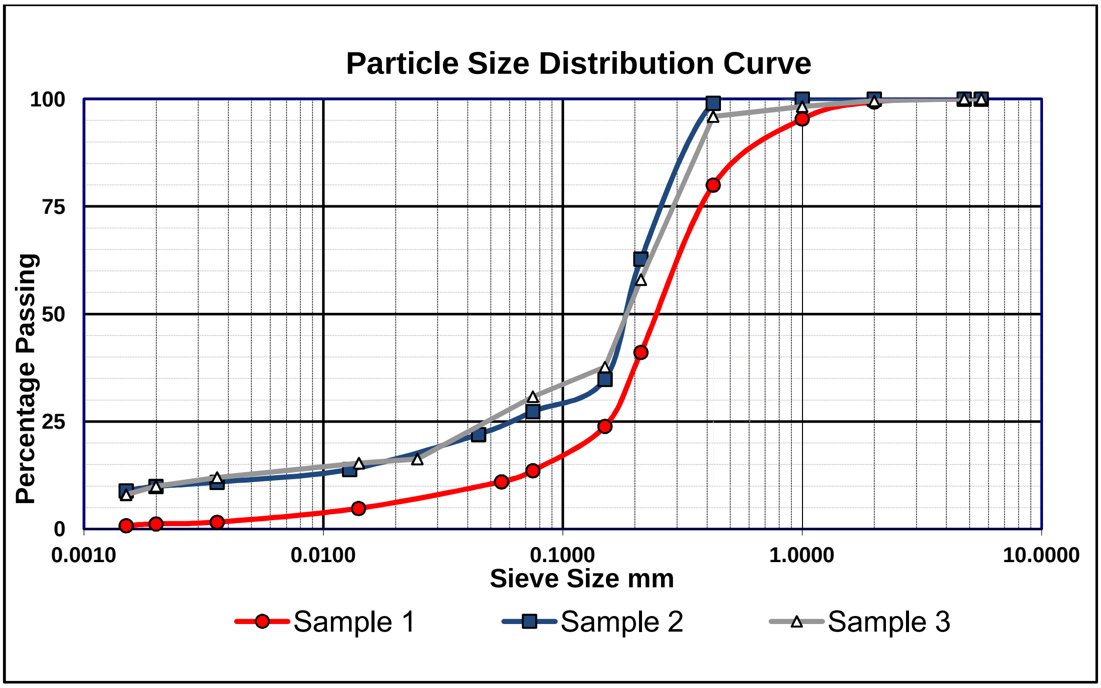



**Figure 3.3 Grading analysis of Berea sands**

| | Soil Sample 1 | Soil Sample 2 | Soil Sample 3 |
|---|---|---|---|
| **Plasticity Index (%)** | Non Plastic (NP) | 7 | Non Plastic (NP 2) |
| **Maximum Dry Density (kg/m$^3$)** | 1861 | 1980 | 2052 |
| **Optimum moisture content (%)** | 8.9 | 13 | 10 |
| **TRH Classification** | G7 | G10 | G10 |
| **Co-efficient of uniformity (Cu)** | 7 | 76 | 112 |

**Table 3.2 Summary of parameters of Berea sands**

| Soil Sample no. | $d_{10}$ (mm) | $d_{20}$ (mm) | $d_{30}$ (mm) | $d_{50}$ (mm) | $d_{60}$ (mm) | $d_{85}$ (mm) |
|---|---|---|---|---|---|---|
| **1** | 0.040 | 0.122 | 0.172 | 0.261 | 0.316 | 0.612 |
| **2** | 0.002 | 0.037 | 0.102 | 0.184 | 0.206 | 0.343 |
| **3** | 0.002 | 0.053 | 0.073 | 0.188 | 0.223 | 0.364 |

**Table 3.3: $d_x$ of all the soil samples**

where $d_x$ = dimension of soil particle size where x per cent is smaller.

All the soil samples' vital parameters are illustrated in Table 3.3 above. These parameters would prove useful when comparing regional filter criteria later in Chapter 5.

## 3.5 TEST DESCRIPTION

The long term gradient ratio test (LTGRT) apparatus has been discussed previously in this chapter. During the test, water flows vertically downward through a permeameter that is filled with a column of soil situated above a candidate geotextile. Standpipes and their positioning are found along one side of the test apparatus. The purpose of the stand-pipes was to monitor the water head at various points along the height of the permeameter.

The change in the flow rate was also monitored over time in order to calculate the entire system's permeability. If three consecutive similar flow readings were recorded at the outlet, the system was classified as stable and the test was stopped. The gradient ratio over time could then be calculated, which would give an idea of the clogging potential of the soil to geotextile system. This is discussed later in this chapter. Furthermore, the permeability of the soil and the soil to geotextile interface could be calculated. If the permeability of the soil to geotextile interface fell below that of the soil, then the geotextile was considered to have failed.

## 3.6 TEST SETUP AND PREPARATION

No scaling was necessary in order to perform the laboratory tests. The geotextile to soil tests were simulated on a one to one scale, as would be encountered in the field.

Approximately 1000 grams of each soil was used for the long term gradient test. The soil was oven dried for 24 hours at 200 °C. The candidate geotextile sample was cut into circular specimens of 132 mm in diameter (Figure 3.4). The geotextile specimen was then placed onto the base plate of the permeameter cylinder.

**Figure 3.4: Berea soil sample with candidate geotextiles cut into discs** (Author)

A steel support screen was placed on the base. The candidate geotextile disc was placed on top of the steel mesh screen. The purpose of the steel mesh screen was to prevent the geotextile from sagging into the base of the apparatus due to the soil load (Figure 3.5 and Figure 3.6).

**Figure 3.5: Steel support screen situated in base piece** (Author)

**Figure 3.6: Geotextile specimen placed on top of steel screen on base** (Author)

After placing the geotextile onto the mesh screen, a perspex cylinder with an internal diameter of 100 mm was placed on top of the permeameter base (Figure 3.7). The candidate soil was placed into the permeameter, directly on top of the candidate geotextile, in layers of approximately 25 mm. This was continued until a total soil thickness of 100 mm had been achieved. A certain degree of compaction was obtained by hand tamping the cylinder ten times on each opposite side whilst filling the cylinder with the candidate soil. Care was taken not too tamp the cylinder too vigorously, as premature piping and loss of fines through the geotextile could occur during this preparation stage. Subsequently, a 50 mm thick layer of clean silica sand was carefully placed directly on top of the candidate soil. This layer of silica sand was also placed in 25 mm layers. The purpose of the silica sand was to prevent the Berea soil sample from being disturbed during the wetting up process, as described below. Another purpose of the silica sand was to confine the Berea soil sample that was being tested. A cover was attached to the top of the permeameter, which also had a water inlet connected to it. Standpipes of 8 mm diameter were connected to outlet ports on one side of the cylinder at pre-determined positions as described later in this section.

**Figure 3.7: Permeameter ready to receive soil sample and top cover**

The inlet pipes that were connected to the top and bottom of the permeameter were 16 mm in diameter. The system underwent a wetting process from the bottom, whereby taps 2 and 3 were opened (Figure 3.8). Taps 1 and 4 remained closed during this period (Figure 3.8). Water was fed from a reservoir situated on top of the apparatus (Figure 3.8). The wetting process, which occurred in 25 mm increments, was slow so as not to disturb the soil to geotextile interface within the permeameter. The purpose was to achieve soil saturation before the start of the test. The wetting process for each set of tests ranged from approximately one to two weeks. During the wetting process the water's meniscus rose slowly through the soils within the permeameter.

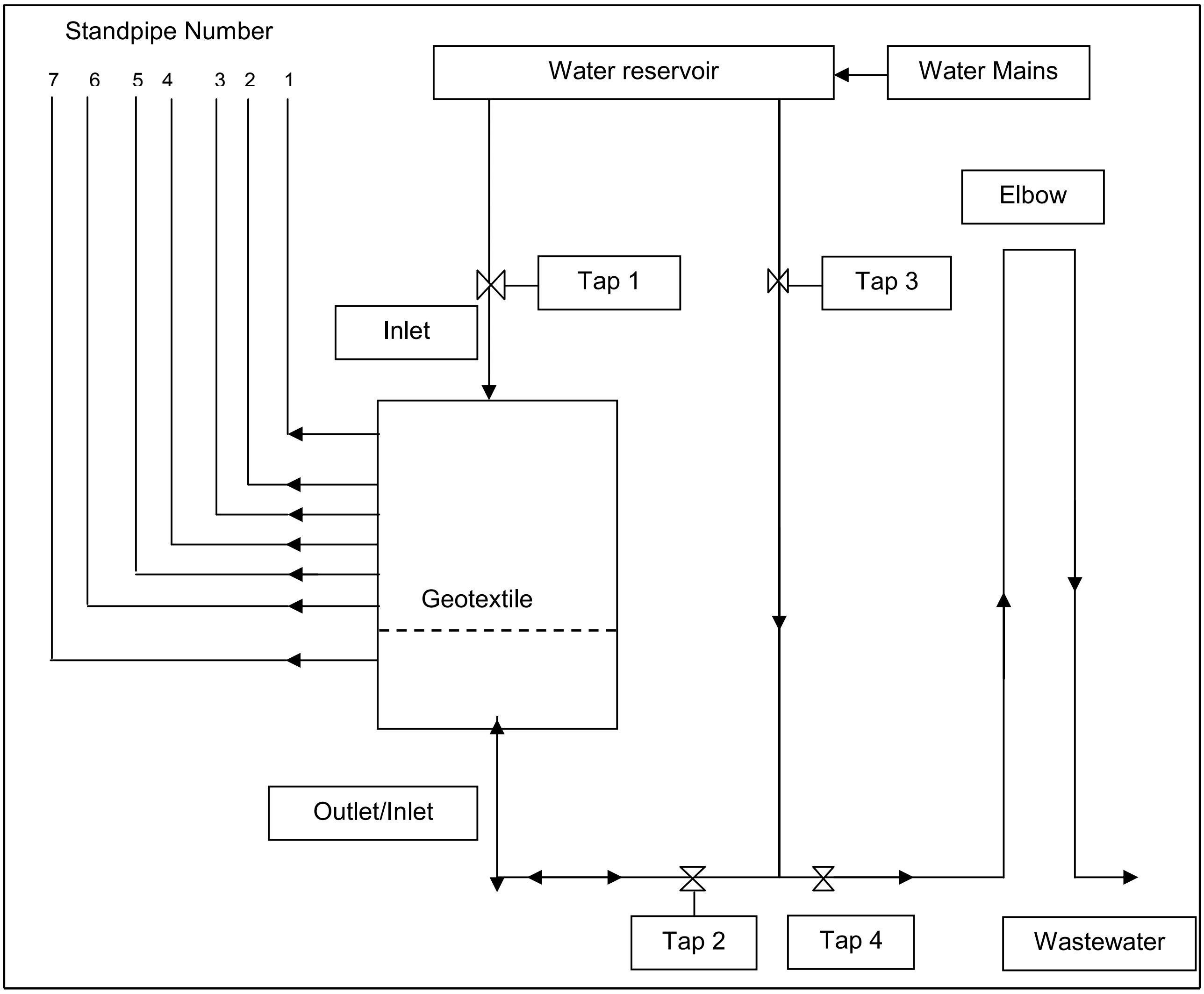



**Figure 3.8: LTGRT flow diagram**

After the water meniscus level had risen approximately 20 mm above the silica sand in the permeameter cylinder, tap 2 was closed. Subsequently, tap 1 was opened and the system was allowed to fill up with water from the top only. This was also done very slowly, in order not to disturb the soil sample within the cylinder. Once this had been achieved, the hydraulic head was pre-determined by adjusting the meniscus level in standpipe 1 in relation to the level of the elbow at the outlet (Figure 3.8). This height difference was set at 100 mm. The height difference was then divided by the soil sample height, also 100 mm, to result in a pre-determined hydraulic gradient of 1. For the purpose of the testing a hydraulic gradient of one (1) was chosen, as referenced in Table 2.1 in Chapter 2 for a typical roadside sub surface drain (Giroud 1996). At this point, readings of the water levels in all the standpipes were taken. At this stage tap 2 was still closed. The system was left to stand for a period of 24 hours. This concluded the sample preparation stage. After the 24 hour period had lapsed, tap 2 and 4 were re-opened and tap 3 was closed and the LTGRT test was started. The aforementioned process was repeated for all 12 LTGRTs.

## 3.7 TESTING

Figure 3.9 below, illustrates an actual LTGRT taking place as part of this research.

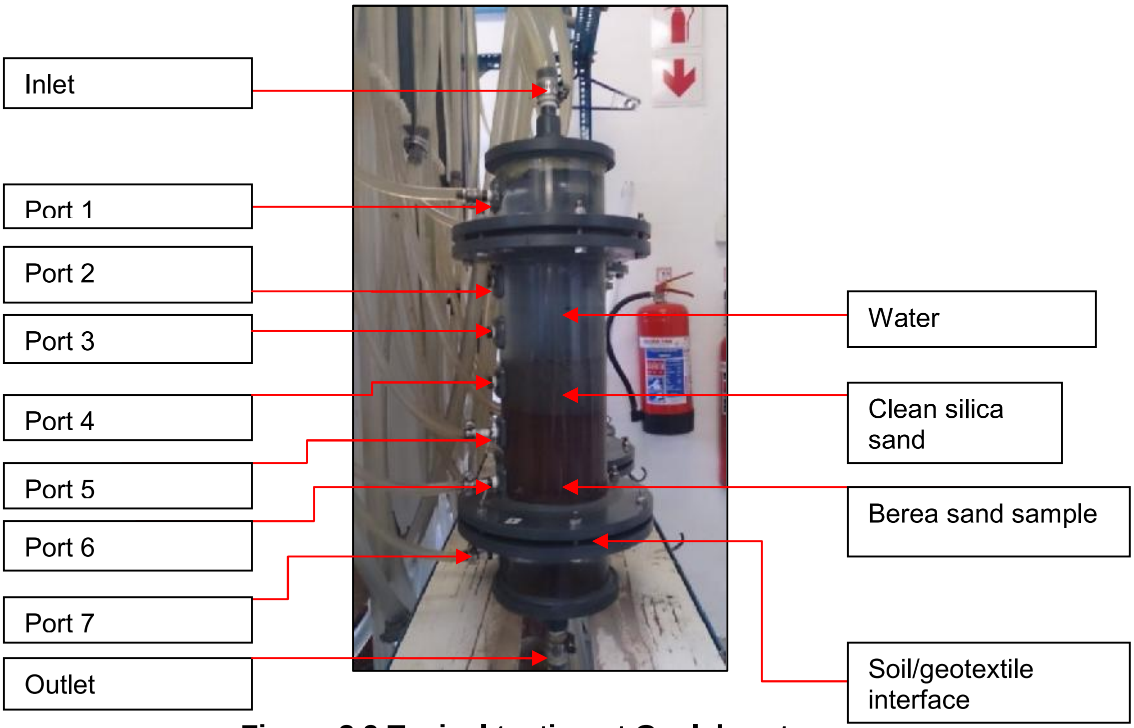



**Figure 3.9 Typical testing at Geolaboratory**

The ports on the apparatus at Geolaboratory were vertically spaced at 25 mm intervals, except for the two bottom most ports, which were spaced at 50 mm from each other. This was to allow the soil to geotextile interface to be situated precisely between the two ports numbered 6 and 7 (Figure 3.9). This meant that the soil to geotextile interface would be 25 mm away from both port 6 and port 7.

As the apparatus could accommodate a soil sample of up to approximately 200 mm in depth, ports number 2, 3 and 4 were redundant, as the hydraulic head readings in these standpipes would be the same, due them all being positioned in water. The LTGRT apparatus was connected to a water reservoir situated directly above the bank of permeameters. One water reservoir fed up to four permeameters simultaneously. This reservoir was in turn connected to another storage tank situated in the laboratory, which was connected to the municipal water mains. After one hour into the test, the flow rate of water passing through the entire system was measured at the outlet. This was achieved by collecting the water passing through the outlet into a measuring beaker over a period of one hour. Flow rate was measured in millilitres per minute (ml/min). Subsequent to this, flow readings were taken on a twenty four hourly basis. Therefore permeability of the entire system could be calculated at regular intervals, using Darcy's equation:

$$\mathbf{Q = kAi}$$

The temperature in the laboratory was kept at approximately 20 degrees Celsius. When the flow rates were found to be the same over three consecutive days, the flow of water through the system was stopped at the inlet. The gradient ratio ($G_R$) over time could be calculated using the following equation (ASTM D5101):

$$\mathbf{G_R = (\Delta h_{sf} / L_{sf}) / ( \Delta h_s / L_s) = L_s \Delta h_{sf} / L_{sf} \Delta h_s}$$

where:

$\Delta h_s$ = $[(S_2 - S_4) + (S_3 - S_5)] / 2$

$\Delta h_{sf}$ = $[(S_4 - S_6) + (S_5 - S_6)] / 2$

Sn = the standpipe reading in cm for the standpipe numbered n.

$L_s$ = 50mm

$L_{sf}$ = 25mm + the geotextile thickness

Refer to Figure 3.10 as a reference diagram.

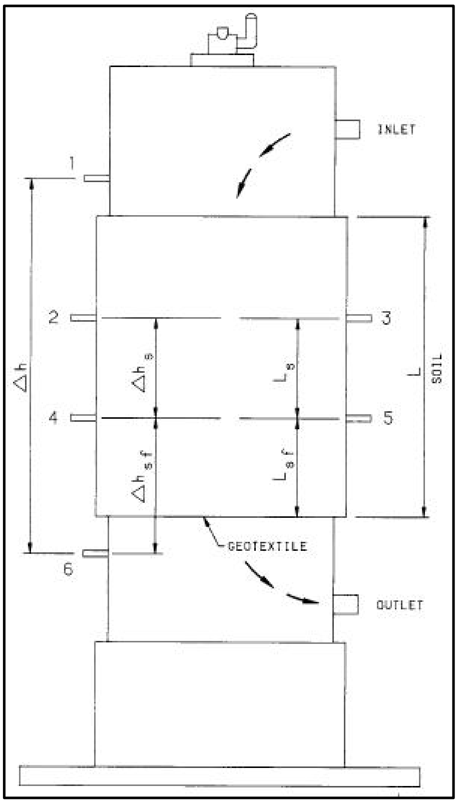



**Figure 3.10: Values for calculating gradient ratios (ASTM D5101)**

Due to the modifications of the apparatus at Geolaboratory, the formula used for the gradient ratio was modified as follows:

where:

$\Delta h_s$ = $(S_2 - S_4)$

$\Delta h_{sf}$ = $(S_4 - S_6)$

$S_n$, $L_s$, $L_{sf}$ remain as defined.

### 3.8 INTERPRETATION OF THE GRADIENT RATIO

According to ASTM D5101, a gradient ratio of 1 is desirable and this renders a soil to geotextile system which has good compatibility. A gradient ratio of more than 1 suggests that the soil to geotextile system is tending towards clogging. Previous research by others has suggested that the upper limit for the gradient ratio should be 3. For the purposes of this research, a gradient ratio of 3 and above would be considered as a soil to geotextile system's tendency towards failure due to clogging.

On the other hand, a gradient ratio of less than 1 suggests that piping of the soil through the geotextile has occurred. A rapid decline in gradient ratio at any point suggests that excessive piping of soil has occurred and this can have the effect of blocking up the drainage system and lead to subsequent failure as a result. The gradient ratio results, analysis and interpretation thereof will be discussed in detail in further chapters of this thesis.

## 3.9 PERMEABILITY

Height readings of the water levels in the standpipes were taken with a tape measure and manually recorded. Initial readings were taken at the start of the flow test. Subsequent readings were taken at the start of the flow tests and then at 24 hourly intervals. The quantity of water flowing through each system was also recorded on a 24 hourly basis. This quantity was measured at the outlet of the permeameter. In this way, the permeability of the system could be calculated using Darcy's equation:

$$\mathbf{Q = k\,(\Delta H / \Delta L)\,A}$$

If the permeability of the soil to geotextile system results in permeability lower than that of the soil, the soil to geotextile system has failed. Luettich *et al*. (1992) had suggested that the permeability of the soil to geotextile system need only be more than that of the soil tested, in order for it to be satisfactory.

## 3.10 CONSTRICTIONS

An evaluation was done of the geotextiles filtration characteristics in relation to its number of constrictions $N_s$. This was achieved by calculating the candidate geotextile's number of constrictions by using the equation below (Giroud 1996):

$$\mathbf{N_s = \sqrt{(1-P)} \times (geotextile\ thickness/fibre\ diameter)}$$

where:

P = Porosity of the geotextile expressed as a percentage

A value $N_s$ of between 25 and 40 is desirable for a geotextile to function optimally as a filter. (Giroud 1996) The geotextile parameters were obtained from the manufacturer's data sheets and, where information was not published, it was requested from the manufacturers.

### 3.11 DESKTOP COMPUTER ANALYSIS

A computer spread sheet program, developed by Kaytech, was used to predict the geotextile filter specification requirements. The predicted computer results would be used to compare with the actual laboratory test results. The computer model was based on the flow chart below (Figure 3.11).

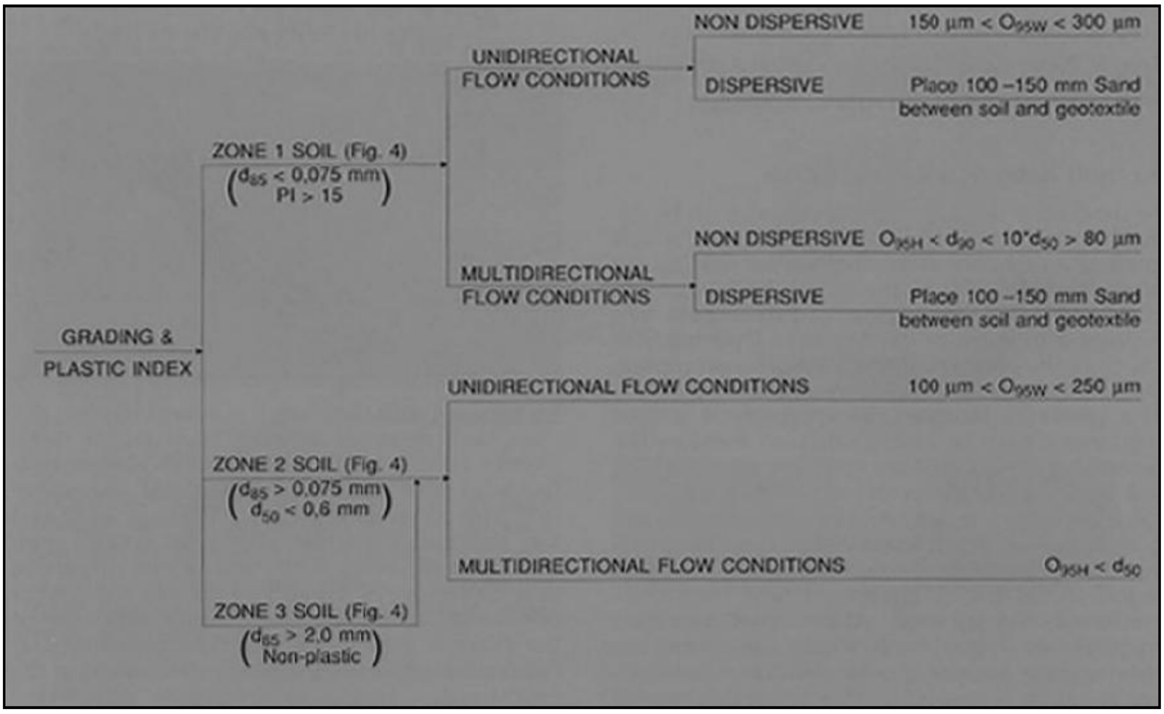



**Figure 3.11: Flow chart for computer spreadsheet** (Kaytech)

The input parameters of the spread sheet were based on:

- Soil zone classification (Figure 3.12)
- Water flow conditions (unidirectional)
- Dispersive, non-dispersive or poor soil conditions
- Relative density ($R_D$) of soil
- Permeability
- Survivability

where:

| | | |
|---|---|---|
| Loose soil | = | $R_D < 35\%$ |
| Medium dense soil | = | $35\% < R_D > 65\%$ |
| Dense soil | = | $R_D > 65\%$ |

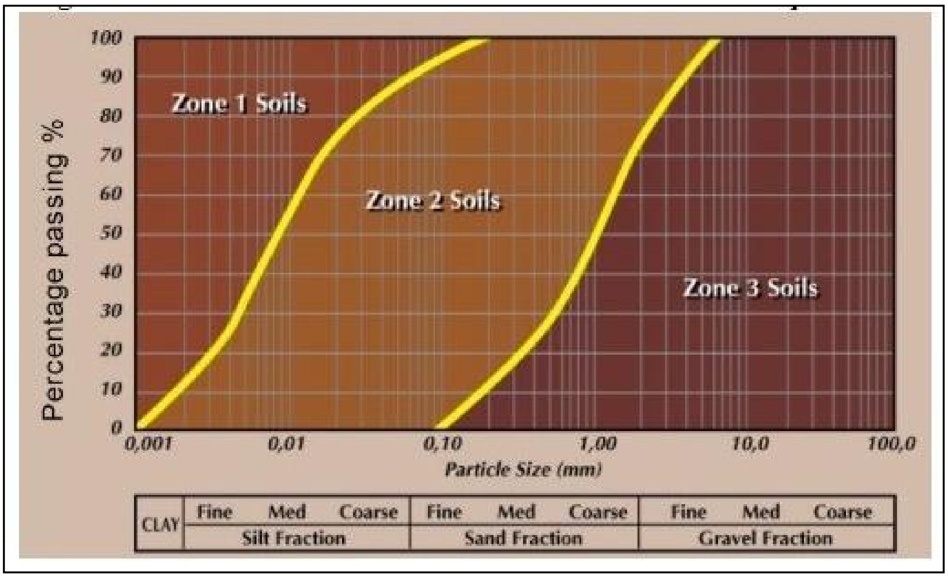



**Figure 3.12: Soil zone classification** (Luettich *et al*. 1992)

Based on the input parameters selected, the program would provide a required generic specification for a geotextile filter for the prevailing conditions. Table 3.3 below highlights the important parameters for the required geotextile filter, as well as the associated test methods. Any geotextile which meets these criteria could potentially be selected as the filter. For critical filter applications where geotextiles are proposed as filters in dams, deep excavations and tailings dam drainage, the program should not be used in isolation.

**Table 3.3: Generic geotextile filter specification** (Kaytech)

| **Geotextile Properties** | | **Test Method** | **Units** |
|---|---|---|---|
| Trapezoidal Tear | Across | ASTM D4533 | N |
| CBR | 50mm probe | SANS 10221-2007 | kN |
| Dart Test | Dia. of hole | EN ISO 13433-2006 | mm |
| Tensile Strength | Across | SANS 10221-2007 | kN/m |
| Permeability | @ 100 mm head | SANS 10221-2008 | m/s |
| Pore size | $O_{95}$ | EN ISO 12956-1999 | µm |

## 3.12 MICROSCOPE IMAGING

Post-test imaging of the contaminated geotextiles was done at the microscopy and microanalysis unit at the University of KwaZulu-Natal, using a scanning electron microscope

(SEM). This was done to assist in visual assessment of the contaminated geotextiles. These results will be presented and discussed in Chapter 4.

## 3.13 LIMITATIONS

Limitations were noted during the experimental phase of the research and are discussed below.

### 3.13.1 Applied vertical stress

No vertical stress was applied to the soil sample, as this is a limitation of the apparatus used. A modified gradient ratio test apparatus has been developed by others, which was not pertinent to the outcomes that this research was trying attempting to achieve.

### 3.13.2 Hydraulic gradient

For the purpose of this research a hydraulic gradient of one (1) was used. Most drainage and filtration applications are situated where low hydraulic gradients are prevalent, for example, sports field drainage, and road and railway drainage. The results for low hydraulic heads would also give us enough understanding of how the geotextiles performed as filters.

### 3.13.3 Algae and iron oxide suppressants

No additives for the prevention of algae and ferric oxide formation were added to the water, as the testing time would not be long enough to warrant the use of these additives.

### 3.13.4 Soil samples

The number of soil to geotextiles permutations was limited to twelve. This equated to three soils and four geotextiles. The main reasons for this were to keep the testing within the time constraints of this study, and to keep it concise. It was also felt that enough data could be extracted from this number of tests in order to deduce some meaningful conclusions.

## 3.14 CONCLUSION

This chapter has summarised the method which was used to try and achieve the desired outcomes of the study. A description of the laboratory tests, as well as the materials used, was given. This chapter further discussed the computer model with its associated parameter predictions. With any testing there are limitations, and these were also alluded to in this chapter. In the following chapter the results obtained from the long term gradient ratio tests and from the computer program will be presented.

# Chapter 4 - Test Results

## 4.1 INTRODUCTION

A total of 12 long term gradient ratio tests (LTGRT) were carried out, which consisted of four variants of geotextiles exposed to three types of Berea sand. The sands were selected on the basis of their plasticity indices, as well as their silt to clay ratios. It was decided to limit the tests to three types of sand as this provided a sufficient base to understand the behaviour of soil to geotextile systems for Berea sand. Each LTGRT was exposed to a hydraulic gradient of 1. A hydraulic gradient ($i_s$) of 1 was used for all test permutations, to enable comparison and mathematical correlation during the analysis. Twelve soil-to-geotextile LTGRT tests were carried out, as illustrated in Table 4.1 below.

| Test Number | Soil Sample No. | Soil PI (%) | Hydraulic Gradient | Geotextile Type |
|---|---|---|---|---|
| 1 | 1 | NP | 1 | NW-N-CF-PET |
| 2 | 1 | NP | 1 | NW-N-SF-PP |
| 3 | 1 | NP | 1 | NW-HB-CF-PP |
| 4 | 1 | NP | 1 | W-SLF-PP |
| 5 | 2 | 7 | 1 | NW-N-CF-PET |
| 6 | 2 | 7 | 1 | NW-N-SF-PP |
| 7 | 2 | 7 | 1 | NW-HB-CF-PP |
| 8 | 2 | 7 | 1 | W-SLF-PP |
| 9 | 3 | NP (2)* | 1 | NW-N-CF-PET |
| 10 | 3 | NP (2)* | 1 | NW-N-SF-PP |
| 11 | 3 | NP (2)* | 1 | NW-HB-CF-PP |
| 12 | 3 | NP (2)* | 1 | W-SLF-PP |

**Table 4.1: Summary of soil to geotextile tests performed**

* where this is the second sample of non-plastic soil.

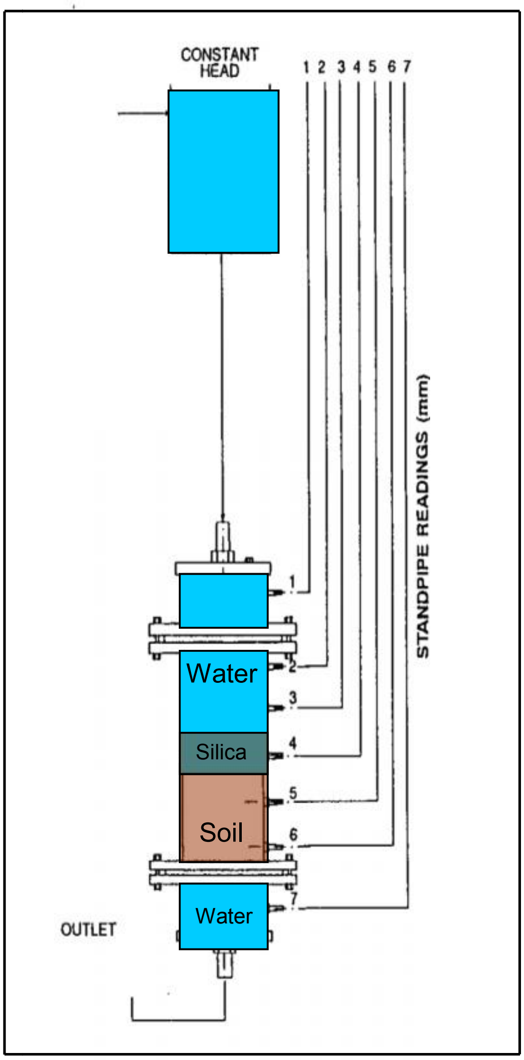



**Figure: 4.1: Reference diagram illustrating standpipe numbers on the LTGRT test apparatus**

For easy reference, the LTGRT diagrammatic layout is illustrated in Figure 4.1 above. The soil to geotextile interface is situated in between standpipes No 6 and 7. Therefore the observations between standpipes 6 and 7 can be regarded as the most important.

## 4.2 NON-PLASTIC SAND VS. NW-N-CF-PET GEOTEXTILE: TEST NO. 1

| Test Cumulative Hours | Quantity ml | Duration min | Permeability k m/s | Sample Height mm | Standpipe Readings - mm Standpipe Number | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| | | | | | 300 | 250 | 200 | 150 | 100 | 50 | 0 |
| | | | | | Inlet | Silica Sand | | | Soil Sample | | Outlet |
| 1 | 150 | 60 | 4.348E-06 | 100 | 1582 | 1582 | 1582 | 1582 | 1576 | 1492 | 1460 |
| 8 | 135 | 60 | 3.914E-06 | 100 | 1582 | 1582 | 1582 | 1582 | 1576 | 1500 | 1460 |
| 72 | 90 | 60 | 2.609E-06 | 100 | 1582 | 1582 | 1582 | 1582 | 1578 | 1492 | 1460 |
| 96 | 75 | 60 | 2.174E-06 | 100 | 1582 | 1582 | 1582 | 1582 | 1575 | 1482 | 1460 |
| 120 | 60 | 60 | 1.739E-06 | 100 | 1582 | 1582 | 1582 | 1582 | 1575 | 1475 | 1460 |
| 144 | 60 | 60 | 1.739E-06 | 100 | 1582 | 1582 | 1582 | 1582 | 1575 | 1495 | 1460 |
| 168 | 60 | 60 | 1.739E-06 | 100 | 1582 | 1582 | 1582 | 1582 | 1575 | 1495 | 1460 |

**Table 4.2: Soil to geotextile permeability and standpipe readings vs. time for test no. 1**

The first test ran for approximately 168 hours before flow equilibrium was reached throughout the entire system (Table 4.2). The sample specimen height for all tests was 100 mm. As in ASTM D5101, all the test readings were stopped when three consecutive unchanged readings in terms of quantity of water, in millilitres, had been observed. Equilibrium flow was achieved from approximately 120 hours onwards. The flow results showed that there was a reduction in the system permeability from the start to the end of the test. Readings in the standpipes numbered 1-4 were constant. The reduction in permeability was observed as illustrated in Figure 4.2. It was also observed that the head difference between standpipes 1 and 7 was 122 mm, as opposed to the desired 100 mm. The difference is attributable to the pressure in the municipal mains and should not have a negative effect on the outcomes of the test results. Head differences closest to 100 mm can therefore be expected for all the testing going forward between standpipes 1 and 7.

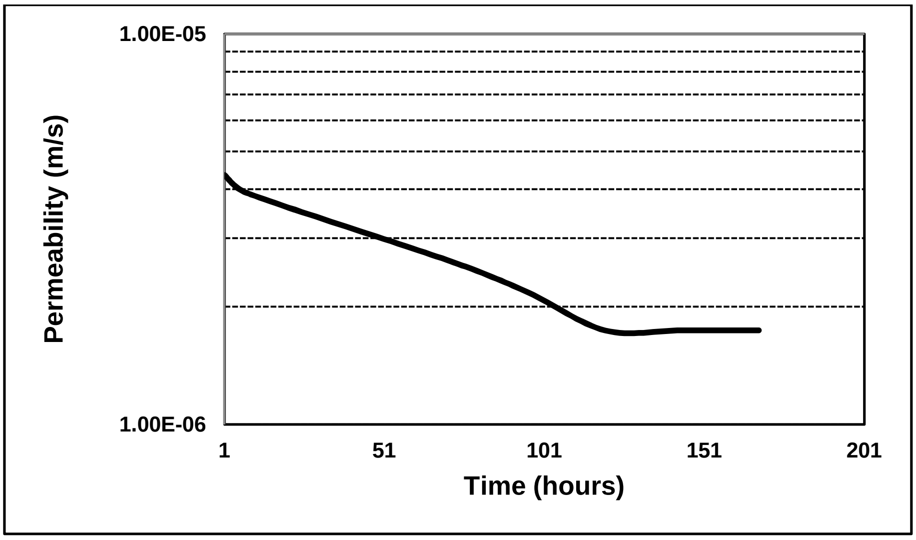



**Figure 4.2: permeability vs. time of entire system: Test no. 1**

In Figure 4.2 it was observed that there was a general drop in permeability between standpipes 4-5 at approximately 75 hours into the test.

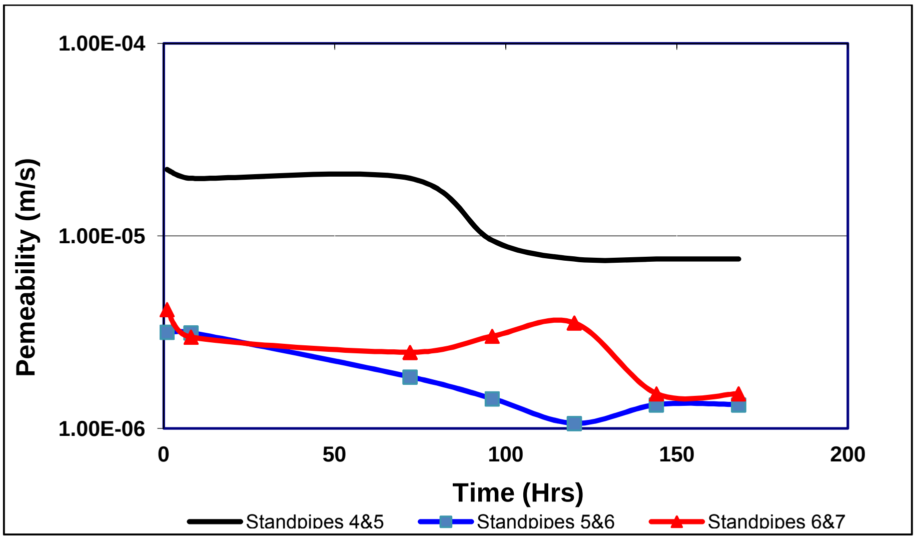



**Figure 4.3: Permeability analyses between standpipes 4-5; 5-6 and 6-7: Test no. 1**

The permeability between standpipes 5 and 6 showed a constant decrease and after approximately 125 hours it started to increase slightly, and this continued to the end of the test (Figure 4.3). It was observed that the permeability between standpipes 6 and 7 was higher than that between standpipes 5 and 6. This means that the soil to geotextile permeability was higher than that of the soil itself, and therefore should be acceptable. Head readings throughout all tests were measured and are obtainable in Appendix A of this thesis.

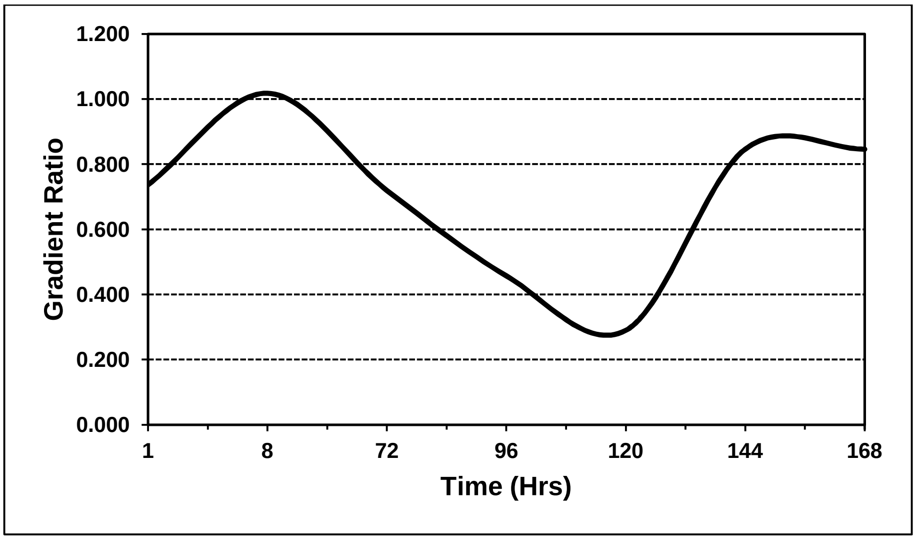



**Figure 4.4: Gradient ratio vs. time: Test no. 1**

Gradient ratios versus time were recorded and are illustrated in Figure 4.4. The gradient ratios were calculated according to the formula discussed in Chapter 3. It was observed that the gradient ratio increased slightly at the beginning of the test and showed a gradual decrease during the middle third of the test. An increase in gradient ratio was observed toward the end of the test. The gradient ratio at the end of the test was 0.846. From the test result, it is evident that the geotextile performed well as a filter with the soil tested.

## 4.3 NON-PLASTIC SAND VS. NW-N-SF-PP GEOTEXTILE: TEST NO. 2

The second LTGRT test lasted 288 hours (Table 4.3). A difference in inlet and outlet readings was observed, when compared with test No.1. The reading in standpipe 5 remained constant from approximately 120 hours until completion of the test. The test was stopped after 288 hours after three consecutive identical readings of 40 ml over a one hour period had been recorded (Table 4.3)

**Table 4.3: Summary of soil to geotextile permeability and standpipe readings vs. time: Test no. 2**

| Test Accumulative Hours | Quantity ml | Duration min | Permeability k m/s | Sample Height mm | Standpipe Readings - mm Standpipe number | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| | | | | | 300 | 250 | 200 | 150 | 100 | 50 | 0 |
| | | | | | Inlet | Silica Sand | | | Soil Sample | | Outlet |
| 1 | 117 | 60 | 4.138E-06 | 100 | 1578 | 1578 | 1578 | 1578 | 1572 | 1490 | 1478 |
| 8 | 115 | 60 | 4.067E-06 | 100 | 1578 | 1578 | 1578 | 1578 | 1572 | 1490 | 1478 |
| 72 | 115 | 60 | 4.067E-06 | 100 | 1578 | 1578 | 1578 | 1578 | 1567 | 1488 | 1478 |
| 96 | 100 | 60 | 3.537E-06 | 100 | 1578 | 1578 | 1578 | 1578 | 1565 | 1488 | 1478 |
| 120 | 70 | 60 | 2.476E-06 | 100 | 1578 | 1578 | 1578 | 1578 | 1560 | 1488 | 1478 |
| 144 | 60 | 60 | 2.122E-06 | 100 | 1578 | 1578 | 1578 | 1578 | 1560 | 1488 | 1478 |
| 168 | 55 | 60 | 1.945E-06 | 100 | 1578 | 1578 | 1578 | 1578 | 1560 | 1485 | 1478 |
| 240 | 40 | 60 | 1.415E-06 | 100 | 1578 | 1578 | 1578 | 1578 | 1560 | 1485 | 1478 |
| 264 | 40 | 60 | 1.415E-06 | 100 | 1578 | 1578 | 1578 | 1578 | 1560 | 1485 | 1478 |
| 288 | 40 | 60 | 1.415E-06 | 100 | 1578 | 1578 | 1578 | 1578 | 1560 | 1485 | 1478 |

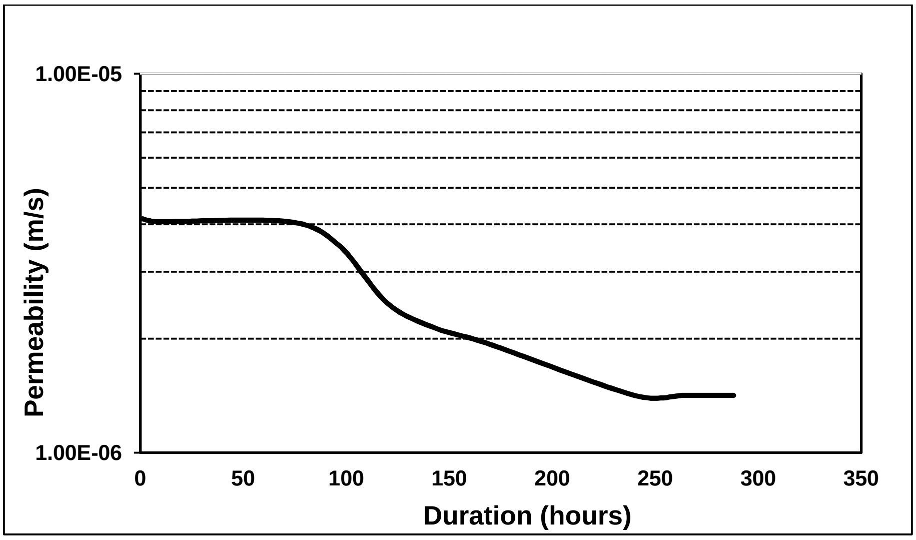



**Figure 4.5 Permeability vs. time of entire system: Test no. 2**

Similar to test number 1, a reduction in permeability was observed over time for the entire system (Figure 4.5). It was observed that the permeability of the system was brought to equilibrium much more quickly than was the case in test No 1.

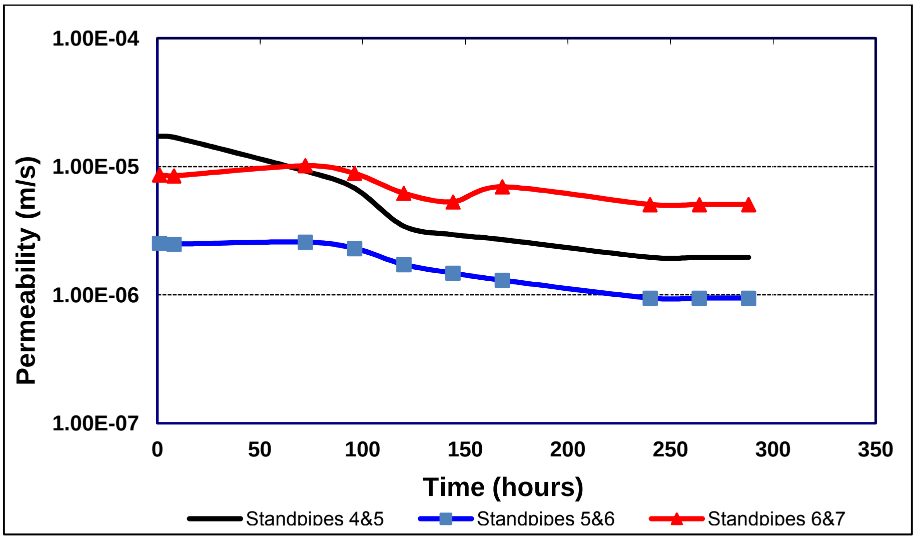



**Figure 4.6 Permeability vs. time across standpipes 4-5; 5-6 and 6-7: Test no. 2**

Figure 4.6 illustrates that the reduction in permeability between standpipes 4 and 5 as well as 5 and 6 followed an even trend downwards. The permeability between standpipes 6 and 7 also showed a reduction in permeability but not at the same rate as across standpipes 4 and 5 and 5and 6. Also interesting to note is that at the beginning of the test, the permeability between standpipes 6 and 7 is higher than that between standpipes 5 and 6. The test also ends with the highest permeability across standpipes 6 and 7. The soil to geotextile permeability was observed to be higher than that of the soil and, therefore, should be acceptable.

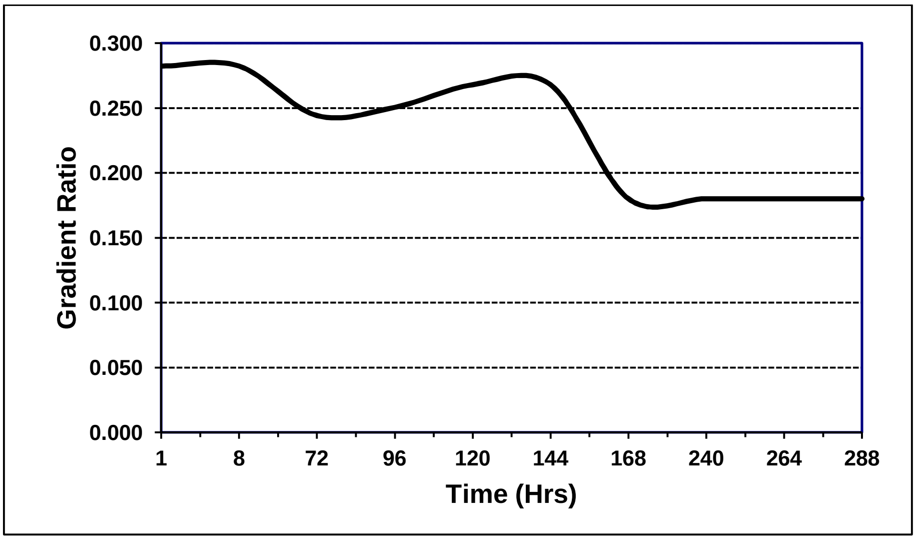



**Figure 4.7 Gradient ratios vs. time: Test no. 2**

The gradient ratio was observed to start at a relatively low level and continued to decrease slightly to stability towards the end of the test (Figure 4.7). A rapid decline in gradient ratio was observed between 144 hours and 168 hours. This was indicative that the fine soil particles had piped through the geotextile. The gradient ratio ended at 0.180. The test result shows that a piping of the finer soil particles had occurred through the geotextile, which is not desirable.

## 4.4 NON- PLASTIC SAND VS. WITH NW-HB-CF-PP GEOTEXTILE: TEST NO. 3

Test number 3 endured for 504 hours (Table 4.4). It was observed that the readings in standpipes 5 and 6 remained constant from approximately 144 hours onwards. Three consecutive readings of 100 ml were recorded at the end of the test.

**Table 4.4: Summary of soil to geotextile permeability and standpipe readings vs. time: Test no. 3**

| Test Accumulative Hours | Quantity ml | Duration min | Permeability k m/s | Sample Height mm | Standpipe Readings - mm | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| | | | | | 300 | 250 | 200 | 150 | 100 | 50 | 0 |
| | | | | | Inlet | Silica Sand | | | Soil Sample | | Outlet |
| 1 | 225 | 60 | 7.958E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1565 | 1520 | 1475 |
| 8 | 200 | 60 | 7.074E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1565 | 1520 | 1475 |
| 72 | 135 | 60 | 4.775E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1562 | 1514 | 1475 |
| 96 | 135 | 60 | 4.775E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1562 | 1514 | 1475 |
| 120 | 100 | 60 | 3.537E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1555 | 1510 | 1475 |
| 144 | 100 | 60 | 3.537E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1550 | 1505 | 1475 |
| 168 | 91 | 60 | 3.218E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1550 | 1505 | 1475 |
| 240 | 80 | 60 | 2.829E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1548 | 1505 | 1475 |
| 264 | 90 | 60 | 3.183E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1548 | 1505 | 1475 |
| 288 | 75 | 60 | 2.653E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1548 | 1505 | 1475 |
| 312 | 90 | 60 | 3.183E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1548 | 1505 | 1475 |
| 336 | 80 | 60 | 2.829E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1548 | 1505 | 1475 |
| 408 | 95 | 60 | 3.360E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1548 | 1505 | 1475 |
| 432 | 115 | 60 | 4.067E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1548 | 1505 | 1475 |
| 456 | 100 | 60 | 3.537E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1548 | 1505 | 1475 |
| 480 | 100 | 60 | 3.537E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1548 | 1505 | 1475 |
| 504 | 100 | 60 | 3.537E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1548 | 1505 | 1475 |

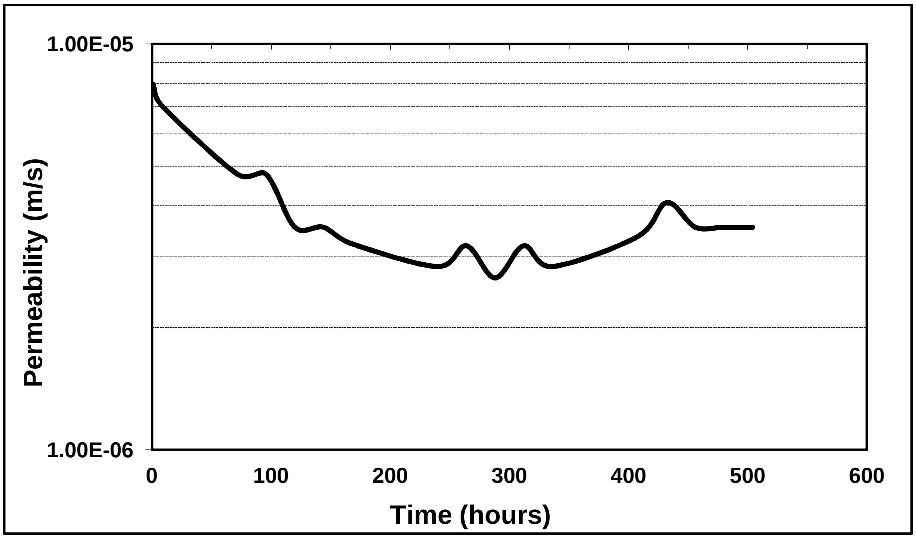



**Figure 4.8: Permeability vs. time of entire system: Test no. 3**

It was observed that the entire system's permeability reduced up to a period of approximately 300 hours, after which it increased until it reached stabilization at 588 hours.

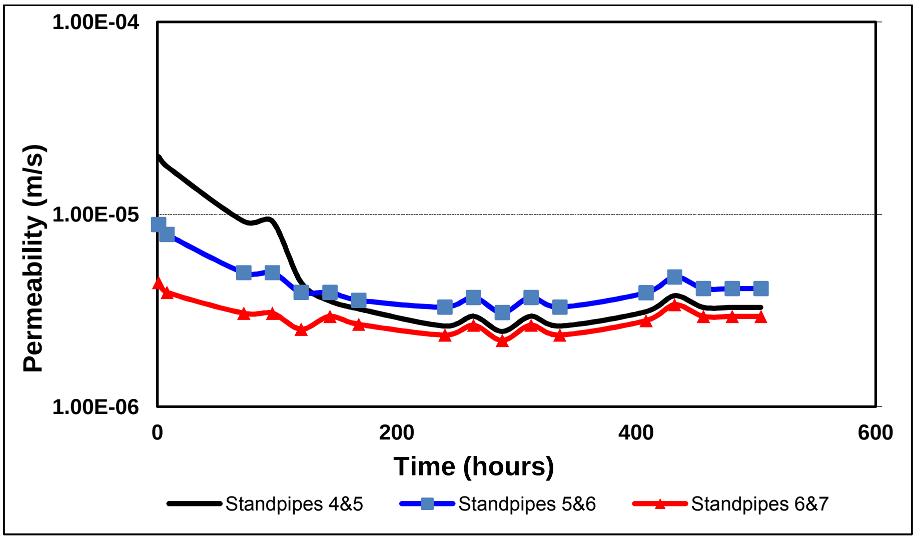



**Figure 4.9: Permeability vs. time across standpipes 4-5; 5-6 and 6-7: Test no. 3**

The permeability between all standpipes dropped relative to each other for the entire duration of the test. The highest residual permeability for this test was observed between standpipes 5 and 6 (Figure 4.9). The permeability of the soil to geotextile was found to be lower than that of the soil, and the combination, therefore, could be problematic.

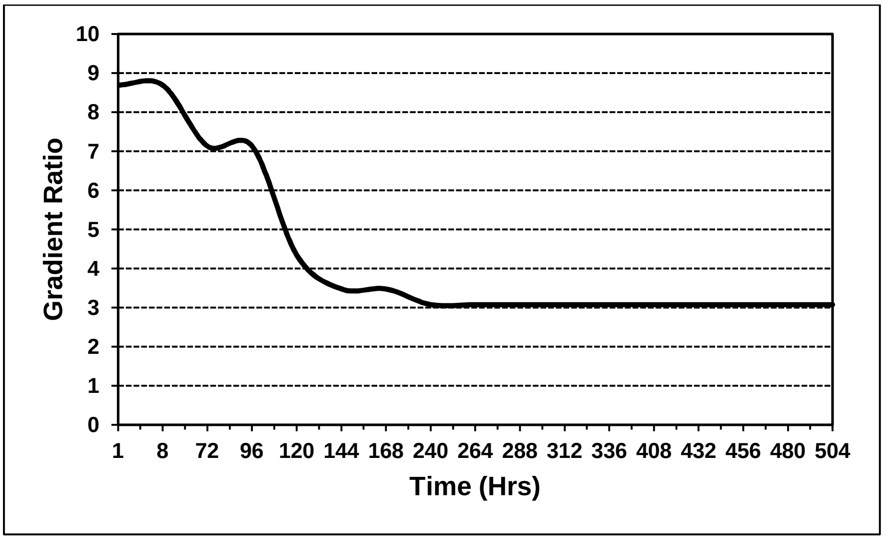



**Figure 4.10: Gradient ratio vs. time: Test no. 3**

It was observed that the gradient ratio reduced and reached stabilisation after approximately 240 hours. Although the gradient ratio remained stable, the test was stopped once the permeability of the system stabilised (Figure 4.10). The gradient ratio obtained at the end of the test was 3.075. The result of this test showed that the geotextile had shown a marginal tendency to clog.

## 4.5 NON-PLASTIC SAND VS. WITH W-SLF-PP GEOTEXTILE: TEST NO. 4

Test no. 4 had a duration of 480 hours (Table 4.5). A rapid decrease in permeability was observed during the early stages of the test and it stabilised during the latter part of the test. The reading in standpipe no. 5 stabilised after 144 hours and that of standpipe no. 6 after 240 hours.

The reduction in permeability of the system was fairly rapid over the early stages of the test (Figure 4.11).

**Table: 4.5 Summary of soil to geotextile permeability and standpipe readings vs. time: Test no. 4**

| Test Accumulative Hours | Quantity ml | Duration min | Permeability k m/s | Sample Height mm | Standpipe Readings - mm 1 300 Inlet | 2 250 Silica Sand | 3 200 | 4 150 | 5 100 Soil Sample | 6 50 | 7 0 Outlet |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 309 | 60 | 1.093E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1520 | 1470 |
| 8 | 270 | 60 | 9.549E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1520 | 1470 |
| 72 | 125 | 60 | 4.421E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1486 | 1470 |
| 96 | 110 | 60 | 3.890E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1486 | 1470 |
| 120 | 95 | 60 | 3.360E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1538 | 1485 | 1470 |
| 144 | 85 | 60 | 3.006E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1530 | 1480 | 1470 |
| 168 | 75 | 60 | 2.653E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1530 | 1480 | 1470 |
| 240 | 60 | 60 | 2.122E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1523 | 1475 | 1470 |
| 264 | 70 | 60 | 2.476E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1528 | 1475 | 1470 |
| 288 | 60 | 60 | 2.122E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1528 | 1475 | 1470 |
| 312 | 70 | 60 | 2.476E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1528 | 1475 | 1470 |
| 336 | 70 | 60 | 2.476E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1528 | 1475 | 1470 |
| 408 | 65 | 60 | 2.299E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1528 | 1475 | 1470 |
| 432 | 80 | 60 | 2.829E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1528 | 1475 | 1470 |
| 456 | 80 | 60 | 2.829E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1528 | 1475 | 1470 |
| 480 | 80 | 60 | 2.829E-06 | 100 | 1570 | 1570 | 1570 | 1570 | 1528 | 1475 | 1470 |

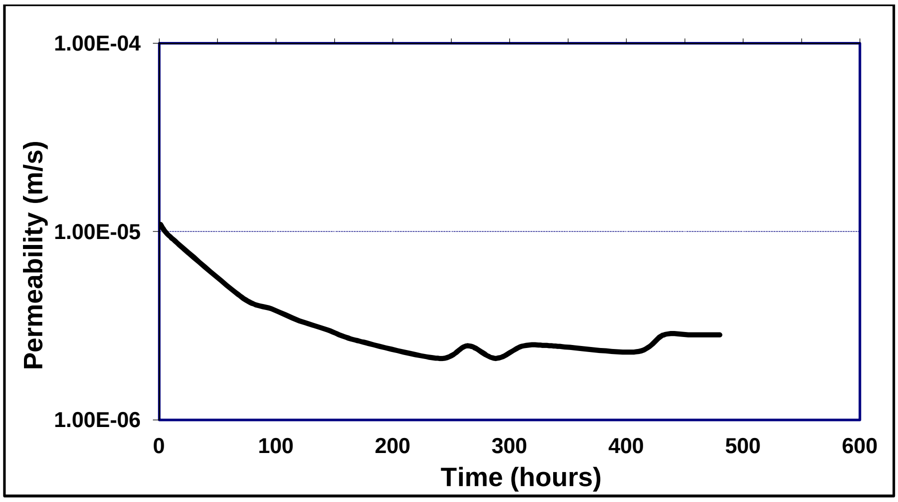



**Figure 4.11: Permeability vs. time of entire system: Test no. 4**

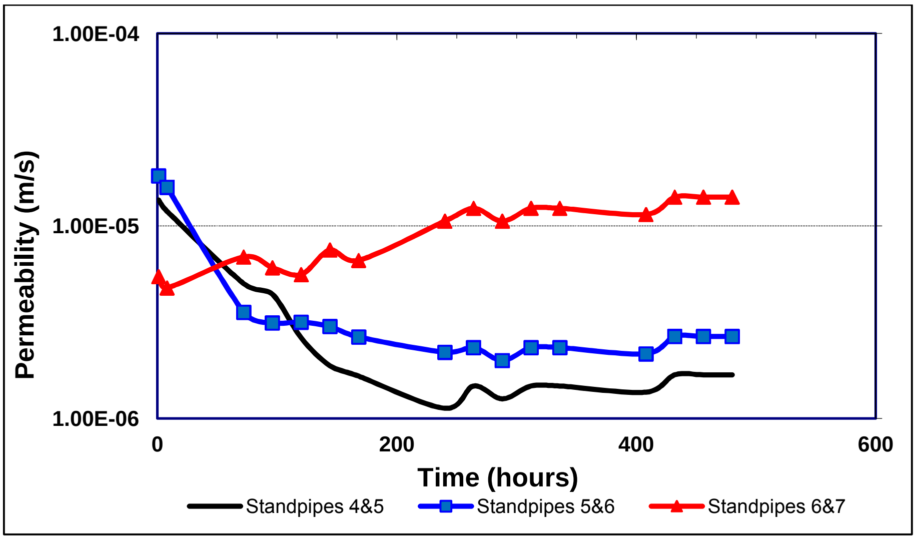



**Figure 4.12: Permeability vs. time across standpipes 4-5; 5-6 and 6-7: Test no. 4**

A reduction of permeability was observed between standpipes 4 and 5 as well as 5 and 6. Between standpipes 6 and 7, an increase in permeability over time was observed (Figure 4.12). The soil to geotextile permeability was observed to be considerably higher than that of the soil and, therefore, should be acceptable.

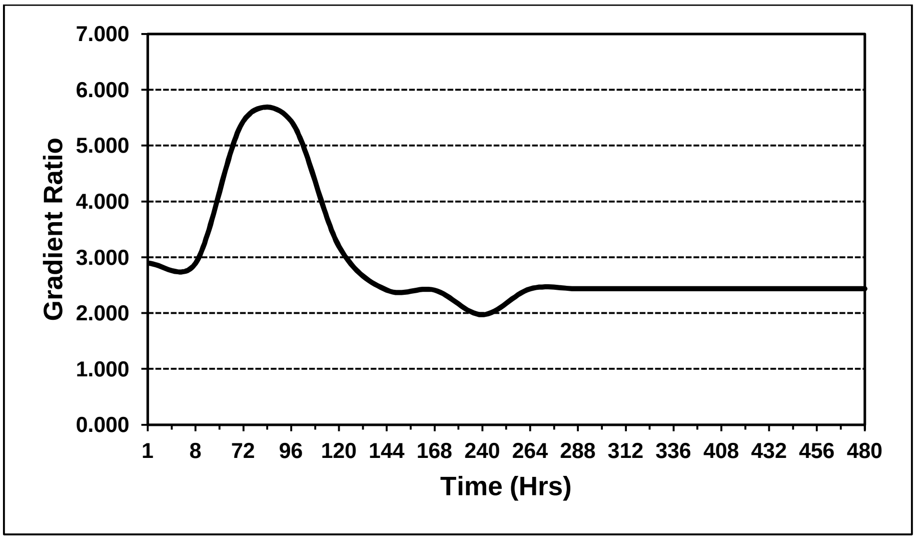



**Figure 4.13: Gradient ratio vs. time: Test no. 4**

It was observed that the gradient ratio increased rapidly during the early parts of the test and reduced to stabilisation after approximately 144 hours (Figure 4.13). Although the gradient ratio stabilised at approximately 264 hours, the test was stopped after the permeability for the entire system stabilised. The gradient ratio obtained at the end of the test was 2.436. From this test result it was evident that the geotextile did not clog and showed relatively good performance as a filter with the soil tested.

## 4.6 SAND WITH PLASTICITY INDEX OF 7 VS. NW-N-CF-PET GEOTEXTILE: TEST NO. 5

The test duration was 860 hours. It was noted that this test took a considerably longer time to stabilise compared to that of tests numbered 1-4. It was also observed that there were fluctuations in standpipe 5 up to approximately 840 hours into the test. Similarly, there were also fluctuations in standpipe 6 up to about 900 hours into the test after it stabilised. The permeability of the system was an order of magnitude of 1 more than that of tests 1-4 (Table 4.6).

## Table: 4.6 Summary of soil to geotextile permeability and standpipe readings vs. time: Test no. 5

| Test | | | Permeability | Sample | Standpipe Readings - mm | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| Accumulative | Quantity | Duration | k | Height | 300 | 250 | 200 | 150 | 100 | 50 | 0 |
| Hours | ml | min | m/s | mm | Inlet | Silica Sand | | | Soil Sample | | Outlet |
| 1 | 1080 | 60 | 3.237E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1565 | 1510 | 1460 |
| 24 | 1125 | 60 | 3.617E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1500 | 1460 |
| 48 | 1260 | 60 | 4.051E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1492 | 1460 |
| 72 | 1250 | 60 | 4.421E-05 | 100 | 1570 | 1570 | 1570 | 1560 | 1540 | 1482 | 1460 |
| 96 | 1570 | 60 | 5.048E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1475 | 1460 |
| 168 | 1400 | 60 | 4.716E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1495 | 1460 |
| 192 | 1350 | 60 | 4.547E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1495 | 1460 |
| 240 | 1275 | 60 | 4.295E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1543 | 1492 | 1460 |
| 264 | 1250 | 60 | 4.210E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1492 | 1460 |
| 336 | 1200 | 60 | 3.858E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1495 | 1460 |
| 360 | 1150 | 60 | 3.874E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1493 | 1460 |
| 384 | 1150 | 60 | 3.874E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1492 | 1460 |
| 408 | 1125 | 60 | 3.789E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1490 | 1460 |
| 432 | 1100 | 60 | 3.705E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1490 | 1460 |
| 504 | 1075 | 60 | 3.621E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1490 | 1460 |
| 528 | 1025 | 60 | 3.453E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1490 | 1460 |
| 552 | 1010 | 60 | 3.402E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1490 | 1460 |
| 576 | 980 | 60 | 3.301E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1490 | 1460 |
| 840 | 910 | 60 | 2.980E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1492 | 1460 |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 864 | 946 | 60 | 3.098E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1492 | 1460 |
| 888 | 940 | 60 | 3.078E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1492 | 1460 |
| 912 | 950 | 60 | 3.111E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1492 | 1460 |
| 936 | 950 | 60 | 3.111E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1492 | 1460 |
| 1008 | 937 | 60 | 2.589E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1492 | 1460 |
| 1032 | 825 | 60 | 2.653E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1490 | 1460 |
| 1056 | 905 | 60 | 2.910E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1490 | 1460 |
| 1080 | 900 | 60 | 2.894E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1490 | 1460 |
| 1104 | 890 | 60 | 2.862E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1490 | 1460 |
| 1176 | 800 | 60 | 2.572E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1490 | 1460 |
| 1200 | 790 | 60 | 2.540E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1490 | 1460 |
| 1224 | 860 | 60 | 2.816E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1480 | 1460 |
| 1248 | 860 | 60 | 2.816E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1480 | 1460 |
| 1272 | 860 | 60 | 2.816E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1480 | 1460 |

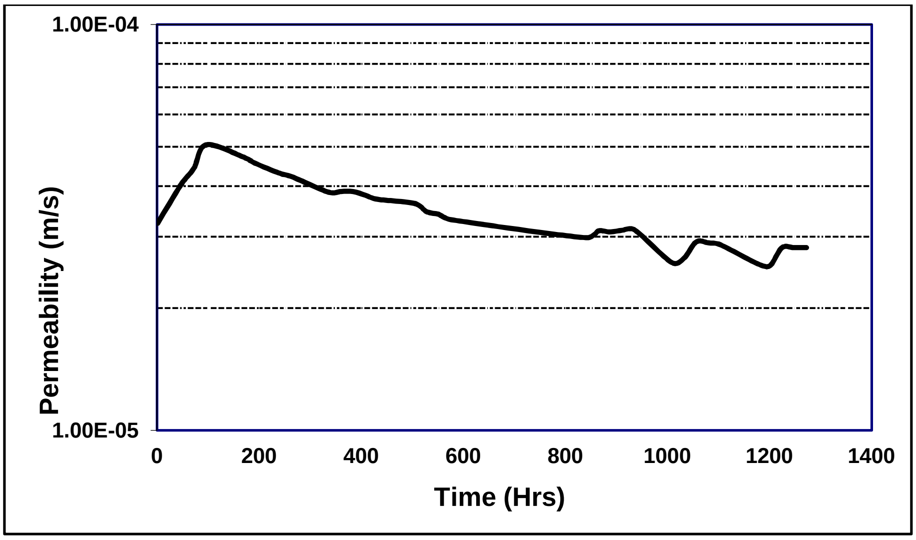



**Figure 4.14: Permeability vs. time of entire system: Test no. 5**

It was observed that after an initial increase in permeability, there was a gradual drop in permeability up to the end of the test (Figure 4.14).

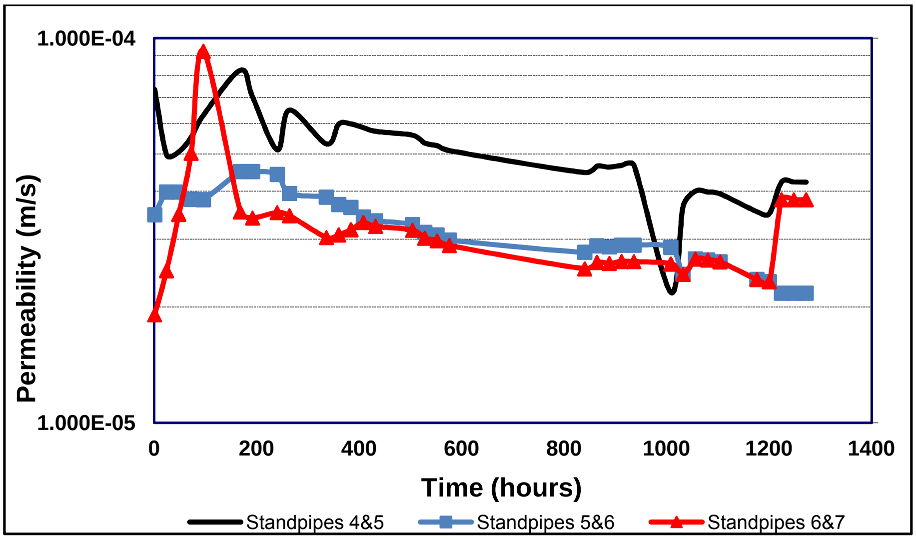



**Figure 4.15: Permeability vs. time across standpipes 4-5; 5-6 and 6-7: Test no. 5**

A reduction in permeability was observed across standpipes 4 and 5 as well as across standpipes 5 and 6 (Figure 4.15). There was also an increase in permeability across standpipes 6 and 7 at the start of the test, after which it dropped quite quickly. There was also a slight increase in permeability towards the end of the test. The soil to geotextile permeability was higher than that of the soil and therefore should be acceptable.

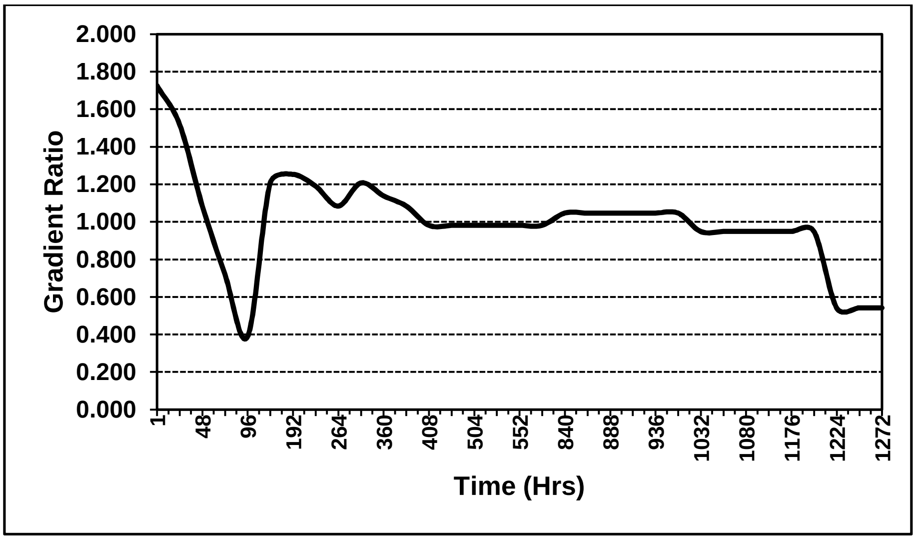



**Figure 4.16: Gradient ratio vs. time: Test no. 5**

The gradient ratio for this test showed a rapid decline up to approximately 96 hours (Figure 4.16). From this it is evident, that some of the fine soil material had passed through the geotextile due to the piping effect. This coincided with the high permeability observed between standpipes 6 and 7 (Figure 4.15). Subsequently, the gradient ratio showed an increase to approximately 1.2, after which it showed some stability up to approximately 1 200 hours into the test. The gradient ratio obtained at the end of the test was 0.542. From this test result, it is evident that the geotextile did not clog and was compatible as a filter with the soil tested.

## 4.7 SAND WITH PLASTICITY INDEX OF 7 VS. NW-N-SF-PP GEOTEXTILE: TEST NO. 6

**Table: 4.7 Summary of soil to geotextile permeability and standpipe readings vs. time: Test no. 6**

| Test Accumulative Hours | Quantity ml | Duration min | Permeability k m/s | Sample Height mm | Standpipe Readings - mm Standpipe Number | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| | | | | | 300 | 250 | 200 | 150 | 100 | 50 | 0 |
| | | | | | Inlet | Silica Sand | | | Soil Sample | | Outlet |
| 1 | 975 | 60 | 2.874E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1555 | 1505 | 1450 |
| 24 | 975 | 60 | 3.079E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1542 | 1495 | 1450 |
| 48 | 1010 | 60 | 3.247E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1542 | 1493 | 1450 |
| 72 | 1000 | 60 | 3.158E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1535 | 1480 | 1450 |
| 96 | 1325 | 60 | 4.184E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1544 | 1480 | 1450 |
| 168 | 1150 | 60 | 3.874E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1542 | 1486 | 1450 |
| 192 | 1100 | 60 | 3.705E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1540 | 1486 | 1450 |
| 240 | 1040 | 60 | 3.503E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1542 | 1486 | 1450 |
| 264 | 1040 | 60 | 3.344E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1540 | 1485 | 1450 |
| 336 | 1000 | 60 | 3.215E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1542 | 1486 | 1450 |
| 360 | 925 | 60 | 2.974E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1542 | 1486 | 1450 |
| 384 | 925 | 60 | 2.974E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1540 | 1485 | 1450 |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 408 | 950 | 60 | 3.054E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1540 | 1485 | 1450 |
| 432 | 900 | 60 | 2.894E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1540 | 1485 | 1450 |
| 504 | 850 | 60 | 2.733E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1540 | 1485 | 1450 |
| 528 | 825 | 60 | 2.653E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1540 | 1485 | 1450 |
| 552 | 800 | 60 | 2.572E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1540 | 1485 | 1450 |
| 576 | 800 | 60 | 2.572E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1540 | 1485 | 1450 |
| 840 | 725 | 60 | 2.331E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1553 | 1495 | 1450 |
| 864 | 775 | 60 | 2.492E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1553 | 1495 | 1450 |
| 888 | 770 | 60 | 2.476E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1553 | 1488 | 1450 |
| 912 | 771 | 60 | 2.479E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1553 | 1488 | 1450 |
| 936 | 780 | 60 | 2.508E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1553 | 1488 | 1450 |
| 1008 | 760 | 60 | 2.444E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1553 | 1488 | 1450 |
| 1032 | 750 | 60 | 2.411E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1490 | 1450 |
| 1056 | 730 | 60 | 2.347E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1490 | 1450 |
| 1080 | 720 | 60 | 2.315E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1490 | 1450 |
| 1104 | 720 | 60 | 2.315E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1490 | 1450 |
| 1176 | 640 | 60 | 2.058E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1490 | 1450 |
| 1200 | 640 | 60 | 2.058E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1490 | 1450 |
| 1224 | 700 | 60 | 2.251E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1488 | 1450 |
| 1248 | 700 | 60 | 2.251E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1488 | 1450 |
| 1272 | 700 | 60 | 2.251E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1488 | 1450 |

Test 6 had a duration of 1 272 hours (Table 4.7). A difference of 110 mm was observed between the inlet and outlet at the end of the test. The level in standpipe 5 only stabilised after 1 032 hours and that of standpipe 6 after 1 224 hours.

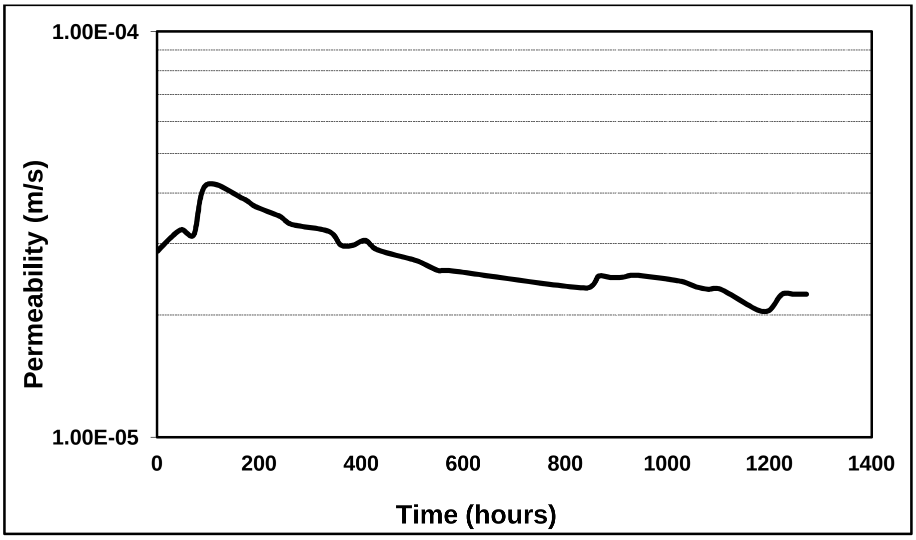



**Figure 4.17: Permeability vs. time of entire system: Test no. 6**

Similar to test no 5, there was an increase in permeability at the start of the test and a gradual decrease over time to the end of the test (Figure 4.17).

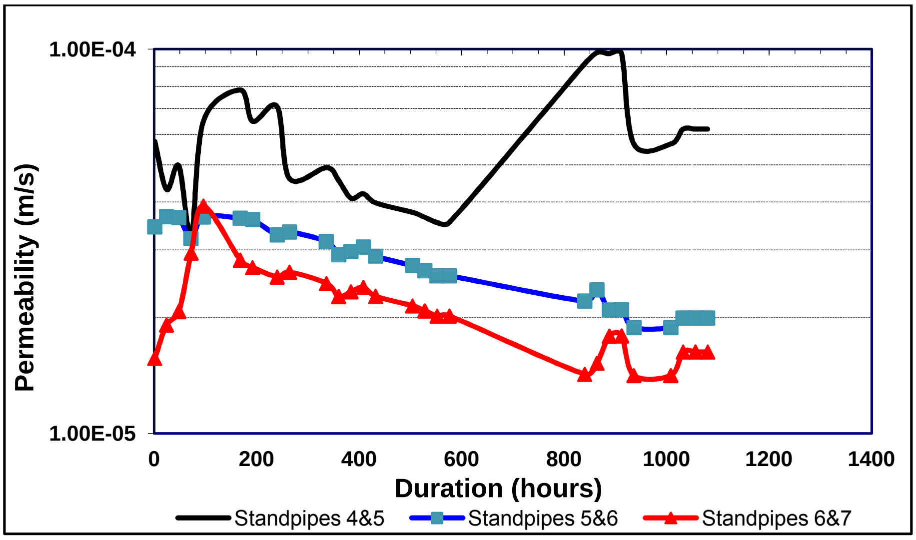



**Figure 4.18: Permeability vs. time across standpipes 4-5; 5-6 and 6-7: Test no. 6**

In Figure 4.18 it was observed that the permeability across standpipes 4 and 5 showed a drop at the start of the test and then showed peaked at approximately 200 hours and 900 hours respectively. The permeability across standpipes 5 and 6 decreased gradually from the start of the test. The permeability across standpipes 6 and 7 showed an increase at the start of the test and a reduction in permeability was observed till the end of the test. The permeability of the soil to geotextile was lower than that of the soil, which could be problematic.

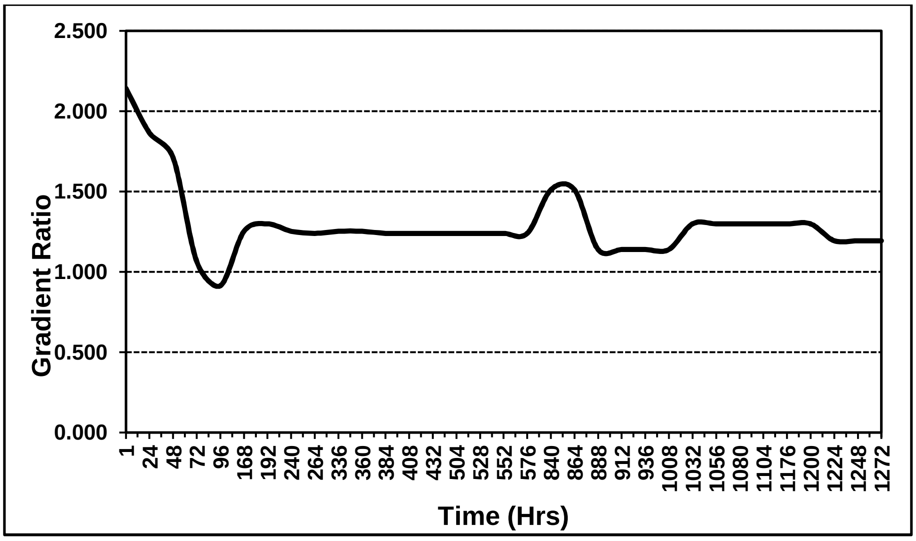



**Figure 4.19: Gradient ratio vs. time: Test no. 6**

At the start of the test, the gradient ratio started above 2 and showed a rapid decrease up to approximately 96 hours (Figure 4.19). It was observed that the gradient ratio of less than 1 was obtained, which coincided with a high permeability measured across standpipes 6-7 (Figure 4.18) over the same time period. The gradient ratio stabilised after approximately 168 hours. The gradient ratio obtained at the end of this test was 1.193. The test result shows that although the geotextile had not clogged, a fair degree of piping of fine soil through the geotextile had occurred at the start of the test, which could potentially be problematic.

## 4.8 SAND WITH PLASTICITY INDEX OF 7 VS. WITH NW-HB-CF-PP GEOTEXTILE: TEST NO. 7

Test 7 endured for 1680 hours (Table 4.8). It was observed that the standpipes 5 and 6 reached stabilisation at 72 hours and 168 hours, respectively. It was observed that the permeability only stabilised after 1 680 hours. A difference of 100 mm was observed between the inlet and outlet at the end of the test.

**Table: 4.8 Summary of soil to geotextile permeability and standpipe readings vs. time: Test no. 7**

| Test Accumulative Hours | Quantity ml | Duration min | Permeability k m/s | Sample Height mm | Standpipe Readings - mm Standpipe Number | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| | | | | | 300 | 250 | 200 | 150 | 100 | 50 | 0 |
| | | | | | Inlet | Silica Sand | | | Soil Sample | | Outlet |
| 1 | 1375 | 60 | 4.421E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1550 | 1525 | 1450 |
| 24 | 1490 | 60 | 5.019E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1542 | 1515 | 1450 |
| 48 | 1520 | 60 | 5.120E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1538 | 1512 | 1450 |
| 72 | 1500 | 60 | 5.584E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1532 | 1502 | 1450 |
| 96 | 1725 | 60 | 5.810E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1540 | 1505 | 1450 |
| 168 | 1800 | 60 | 6.366E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 192 | 1850 | 60 | 6.543E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 240 | 1700 | 60 | 6.013E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 264 | 1740 | 60 | 6.154E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 336 | 1750 | 60 | 6.189E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 360 | 1625 | 60 | 5.747E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 384 | 1690 | 60 | 5.977E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 408 | 1725 | 60 | 6.101E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 432 | 1620 | 60 | 5.730E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 504 | 1590 | 60 | 5.623E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 528 | 1525 | 60 | 5.394E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 552 | 1500 | 60 | 5.305E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 576 | 1450 | 60 | 5.128E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 840 | 1225 | 60 | 4.248E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 864 | 1285 | 60 | 4.456E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 888 | 1285 | 60 | 4.545E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 912 | 1280 | 60 | 4.527E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 936 | 1290 | 60 | 4.562E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1008 | 1275 | 60 | 4.509E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1032 | 1260 | 60 | 4.456E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1056 | 1220 | 60 | 4.315E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1080 | 1205 | 60 | 4.262E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1104 | 1205 | 60 | 4.262E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1176 | 1070 | 60 | 3.784E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1200 | 1060 | 60 | 3.749E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1224 | 1160 | 61 | 4.035E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1248 | 1190 | 62 | 4.073E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1272 | 1190 | 63 | 4.008E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1368 | 1060 | 64 | 3.515E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1392 | 1100 | 65 | 3.591E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1416 | 1150 | 66 | 3.698E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1512 | 1120 | 67 | 3.547E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1526 | 1017 | 68 | 3.174E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1560 | 1010 | 69 | 3.106E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1584 | 1120 | 70 | 3.395E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1628 | 1120 | 71 | 3.347E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |
| 1680 | 1120 | 72 | 3.301E-05 | 100 | 1550 | 1550 | 1550 | 1550 | 1535 | 1505 | 1450 |

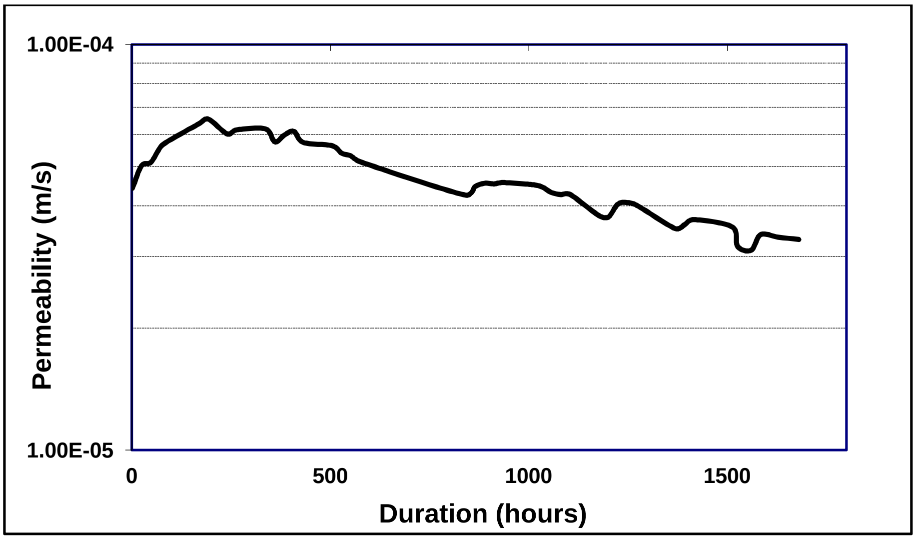



**Figure 4.20: Permeability vs. time of entire system: Test no. 7**

Figure 4.20 shows an initial rise of permeability at the beginning of the test and then a constant drop over the duration of the test.

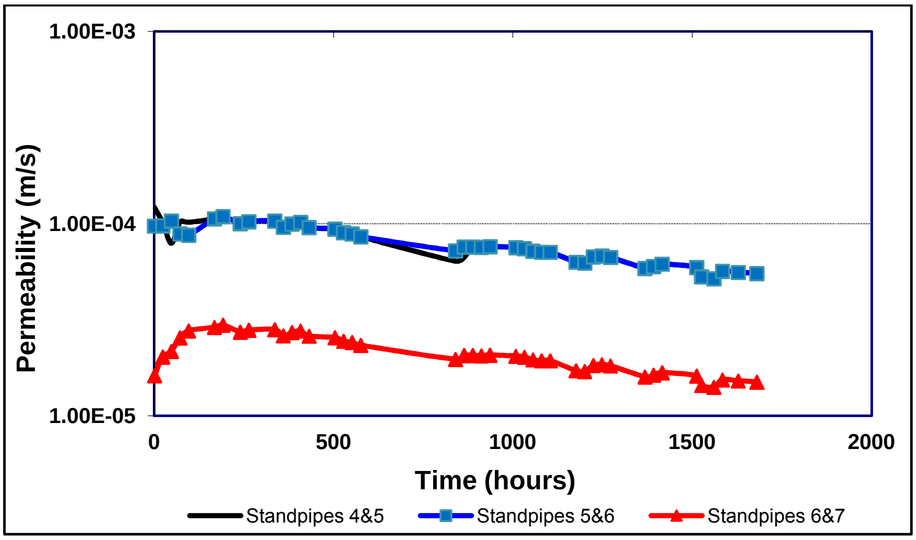



**Figure 4.21: Permeability vs. time across standpipes 4-5; 5-6 and 6-7: Test no. 7**

The permeabilities across standpipes 4 and 5, and 5 and 6 showed consistency relative to each other. A proportional drop in permeability was observed across standpipe 6 and 7 (Figure 4.21). The soil to geotextile permeability was lower than that of the soil, and this could be problematic.

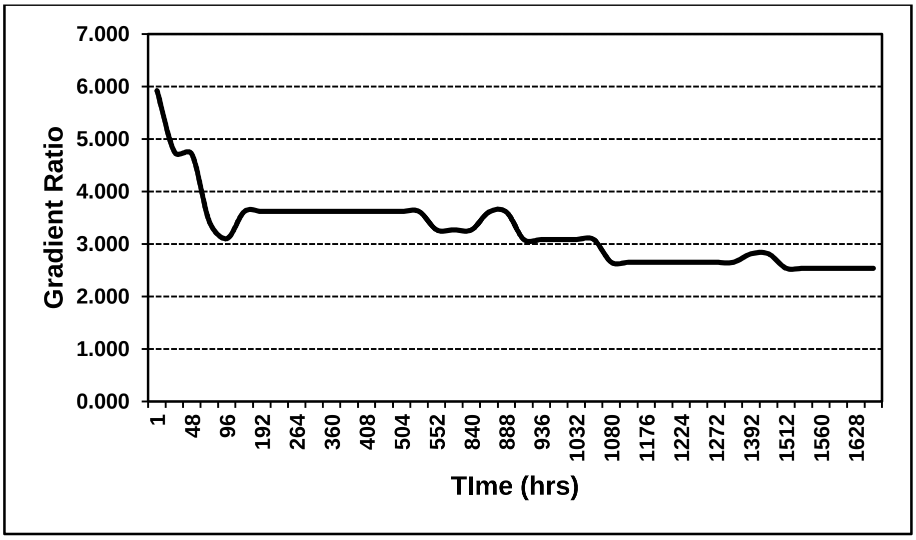



**Figure 4.22: Gradient ratio vs. time: Test no. 7**

The gradient ratio for this test started at approximately 6 (Figure 4.22). After approximately 96 hours, the gradient ratio decreased considerably, by an amount of between 3 and 4. This was indicative of piping of fine soil particles through the geotextile. The gradient ratio obtained at the end of the test was 2.537. This result meant that the geotextile was compatible with this soil in terms of the results of the gradient ratio. However, as alluded to in Figure 4.21, this soil to geotextile permeability was lower than the soil and therefore, in this respect, problematic.

## 4.9 SAND WITH PLASTICITY INDEX OF 7 VS. WITH W-SLF-PP GEOTEXTILE: TEST NO. 8

Test number 8 endured for 1680 hours (Table 4.9). The drop in permeability from the start of the test through to the end of the test was observed to be negligible. The water levels in both standpipes 5 and 6 reach stability after 1 512 hours. Head readings between standpipes 1 and 7 were observed to be 90 mm.

**Table 4.9: Soil to geotextile permeability and standpipe readings vs. time: Test no. 8**

| Test | | | Permeability | Sample | Standpipe Readings - mm | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| Accumulative | Quantity | Duration | k | Height | 300 | 250 | 200 | 150 | 100 | 50 | 0 |
| Hours | ml | min | m/s | mm | Inlet | Silica Sand | | | Soil Sample | | Outlet |
| 1 | 1210 | 60 | 4.755E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1544 | 1502 | 1450 |
| 24 | 1540 | 60 | 6.052E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1532 | 1480 | 1450 |
| 48 | 1540 | 60 | 6.052E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1480 | 1450 |
| 72 | 1490 | 60 | 5.855E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1523 | 1473 | 1450 |
| 96 | 1900 | 60 | 7.467E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1475 | 1450 |
| 168 | 1800 | 60 | 7.074E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1525 | 1473 | 1450 |
| 192 | 1750 | 60 | 6.877E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1523 | 1473 | 1450 |
| 240 | 1650 | 60 | 6.484E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1525 | 1473 | 1450 |
| 264 | 1640 | 60 | 6.445E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1526 | 1470 | 1450 |
| 336 | 1600 | 60 | 6.288E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1526 | 1470 | 1450 |
| 360 | 1500 | 60 | 5.895E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1526 | 1470 | 1450 |
| 384 | 1500 | 60 | 5.895E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1470 | 1450 |
| 408 | 1575 | 60 | 6.189E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1470 | 1450 |
| 432 | 1450 | 60 | 5.698E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1470 | 1450 |
| 504 | 1400 | 60 | 5.502E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1470 | 1450 |
| 528 | 1250 | 60 | 4.912E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1470 | 1450 |
| 552 | 1325 | 60 | 5.207E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1470 | 1450 |
| 576 | 1275 | 60 | 5.010E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1470 | 1450 |
| 840 | 1160 | 60 | 4.559E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1470 | 1450 |

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 864 | 1225 | 60 | 4.814E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1478 | 1450 |
| 888 | 1200 | 60 | 4.716E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1478 | 1450 |
| 912 | 1224 | 60 | 4.810E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1478 | 1450 |
| 936 | 1220 | 60 | 4.794E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1478 | 1450 |
| 1008 | 1210 | 60 | 4.755E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1530 | 1478 | 1450 |
| 1032 | 1206 | 60 | 4.739E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1056 | 1170 | 60 | 4.598E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1080 | 1160 | 60 | 4.559E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1104 | 1155 | 60 | 4.539E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1176 | 1020 | 60 | 4.008E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1200 | 1000 | 60 | 3.930E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1224 | 1100 | 60 | 4.323E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1248 | 1140 | 60 | 4.480E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1272 | 1140 | 60 | 4.480E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1368 | 1020 | 60 | 4.008E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1392 | 1060 | 60 | 4.166E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1416 | 1100 | 60 | 4.323E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1527 | 1475 | 1450 |
| 1512 | 1050 | 60 | 4.126E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1522 | 1470 | 1450 |
| 1526 | 976 | 60 | 3.835E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1522 | 1470 | 1450 |
| 1560 | 1002 | 60 | 3.938E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1522 | 1470 | 1450 |
| 1584 | 1100 | 60 | 4.323E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1522 | 1470 | 1450 |
| 1628 | 1100 | 60 | 4.323E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1522 | 1470 | 1450 |
| 1680 | 1100 | 60 | 4.323E-05 | 100 | 1540 | 1540 | 1540 | 1540 | 1522 | 1470 | 1450 |

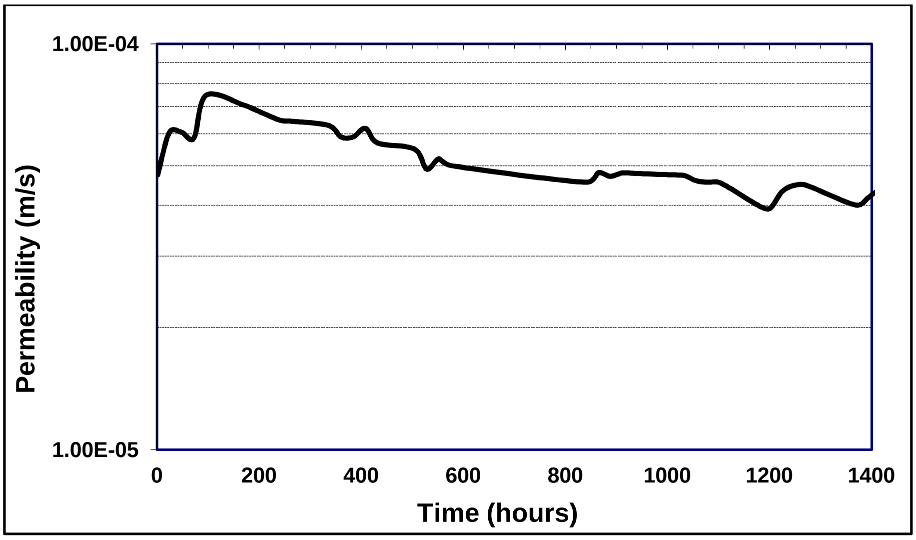



**Figure 4.23: Permeability vs. time of entire system: Test no. 8**

In line with most of the previous tests, there was an initial rise in permeability at the start of the test and a gradual decrease in permeability across the duration of the test (Figure 4.23).

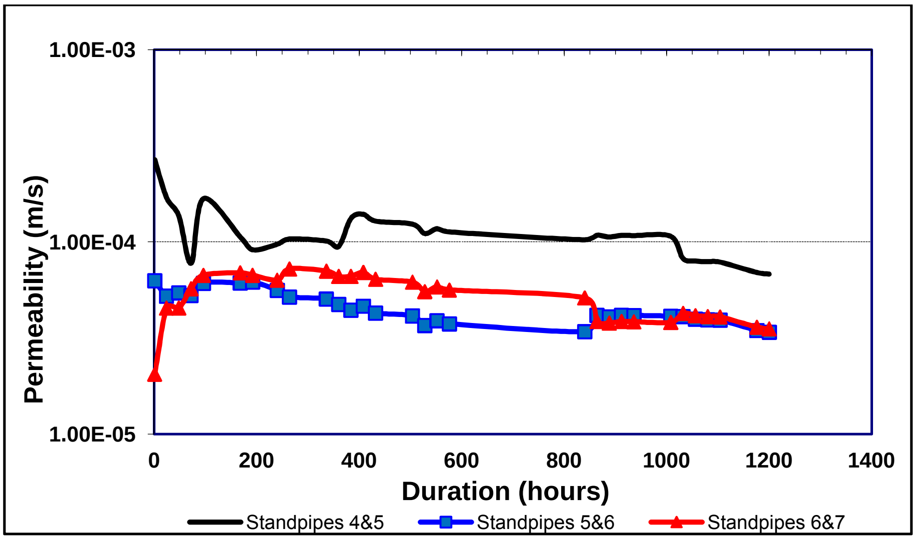



**Figure 4.24: Permeability vs. time across standpipes 4-5; 5-6 and 6-7: Test no. 8**

The permeability across standpipes 6 and 7 was observed to exceed the permeability across standpipes 5 and 6 (Figure 4.24). The permeabilities across these standpipes were approximately the same after 800 hours up to the end of the test. This meant that the soil to geotextile permeability was almost equal to that of the soil, which was not ideal.

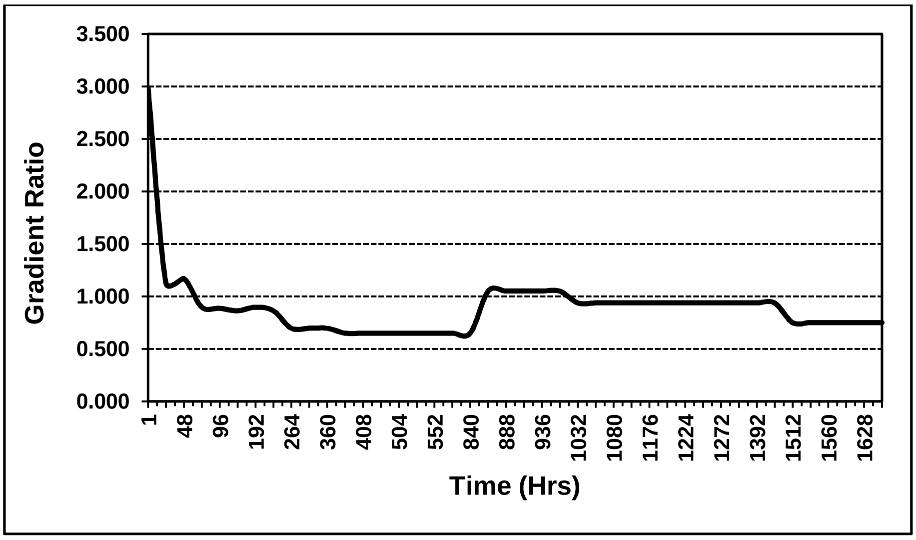



**Figure 4.25: Gradient ratio vs. time: Test no. 8**

During the early parts of the test there was a rapid decline in the gradient from 3.000 to approximately 1.100 (Figure 4.25). This was indicative that initial piping of the soil occurred during this part of the test. The gradient ratio obtained at the end of the test was 0.751. According to the test results obtained, the geotextile was compatible as a filter with the soil tested.

### 4.10 NON PLASTIC SAND (2) VS. WITH NW-N-CF-PET GEOTEXTILE: TEST NO. 9

Test no. 9 had a duration of 648 hours (Table 4.10). The readings in standpipe 5 stabilised after 504 hours and those of standpipe 6 stabilised after 624 hours. The head across the entire system was determined to be 100 mm.

**Table 4.10: Soil to geotextile permeability and standpipe readings vs. time: Test no. 9**

| Test Accumulative Hours | Quantity ml | Duration min | Permeability k m/s | Sample Height mm | Standpipe Readings - mm | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| | | | | | 300 | 250 | 200 | 150 | 100 | 50 | 0 |
| | | | | | Inlet | Silica Sand | | | Soil Sample | | Outlet |
| 1 | 180 | 60 | 6.366E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1570 | 1550 | 1475 |
| 24 | 165 | 60 | 5.836E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1570 | 1550 | 1475 |
| 48 | 165 | 60 | 5.836E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1570 | 1550 | 1475 |
| 120 | 325 | 60 | 1.149E-05 | 100 | 1575 | 1575 | 1575 | 1575 | 1565 | 1550 | 1475 |
| 144 | 350 | 60 | 1.238E-05 | 100 | 1575 | 1575 | 1575 | 1575 | 1565 | 1550 | 1475 |
| 168 | 375 | 60 | 1.326E-05 | 100 | 1575 | 1575 | 1575 | 1575 | 1565 | 1550 | 1475 |
| 192 | 380 | 60 | 1.344E-05 | 100 | 1575 | 1575 | 1575 | 1575 | 1565 | 1550 | 1475 |
| 216 | 345 | 60 | 1.220E-05 | 100 | 1575 | 1575 | 1575 | 1575 | 1565 | 1550 | 1475 |
| 288 | 245 | 60 | 8.665E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1565 | 1555 | 1475 |
| 312 | 225 | 60 | 7.958E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1565 | 1555 | 1475 |
| 336 | 200 | 60 | 7.074E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1565 | 1555 | 1475 |
| 360 | 175 | 60 | 6.189E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1565 | 1555 | 1475 |
| 384 | 155 | 60 | 5.482E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1568 | 1560 | 1475 |
| 456 | 129 | 60 | 4.562E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1568 | 1560 | 1475 |
| 480 | 115 | 60 | 4.519E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1568 | 1560 | 1475 |
| 504 | 110 | 60 | 4.323E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1570 | 1568 | 1475 |
| 528 | 100 | 60 | 3.930E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1570 | 1568 | 1475 |
| 600 | 70 | 60 | 2.751E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1570 | 1567 | 1475 |
| 624 | 70 | 60 | 2.662E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1570 | 1565 | 1475 |
| 648 | 70 | 60 | 2.662E-06 | 100 | 1575 | 1575 | 1575 | 1575 | 1570 | 1565 | 1475 |

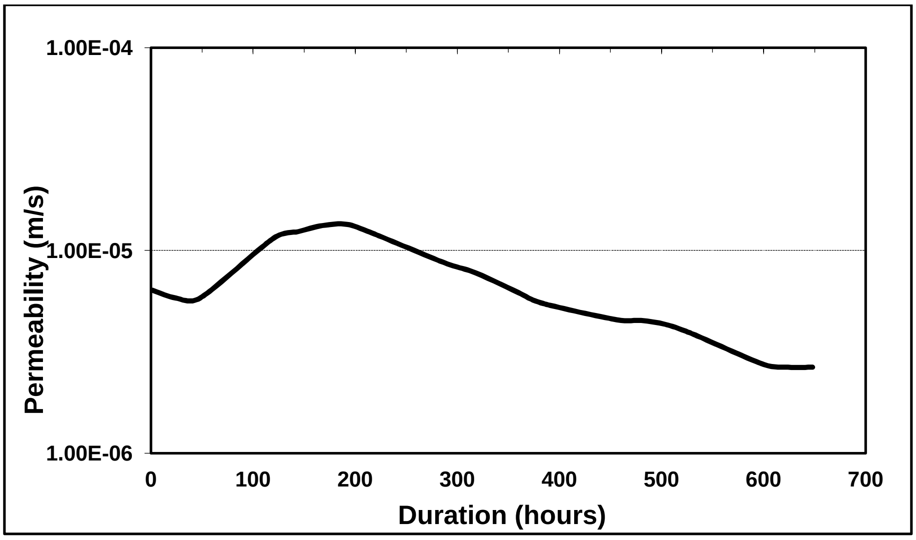



**Figure 4.26: Permeability vs. time of entire system: Test no. 9**

There was a drop in permeability across the entire system (Figure 4.26).

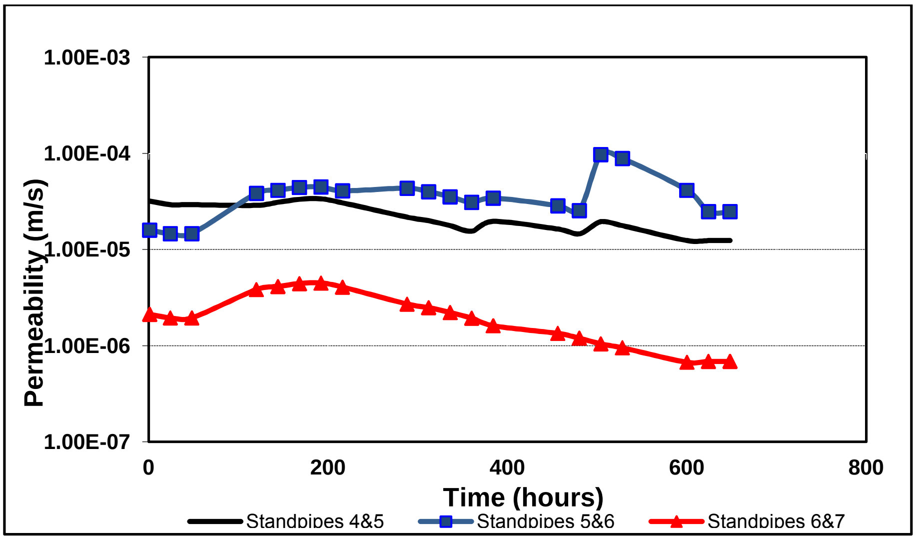



**Figure 4.27: Permeability vs. time across standpipes 4-5; 5-6 and 6-7: Test no. 9**

The permeability measured across standpipes 6 and 7 was found to be at least one order of magnitude lower than that measured across standpipes 4 and 5 and standpipes 5 and 6 (Figure 4.27). The permeability across standpipes 6 and 7 proved to have had a drawdown effect on the permeability of the entire system. This also means that the soil to geotextile permeability was lower than that of the soil and therefore problematic.

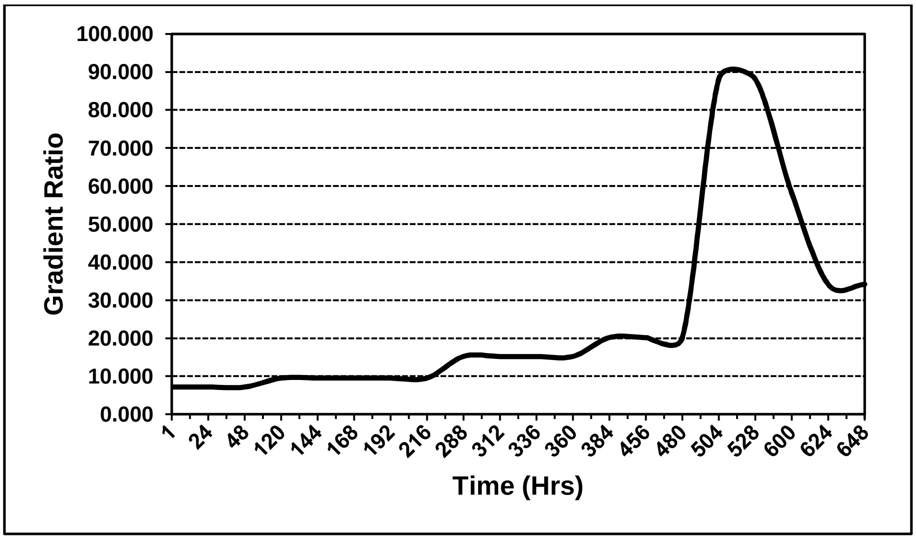



**Figure 4.28: Gradient ratio vs. time: Test no. 9**

The gradient ratio of test 9 was shown to be relatively high from the start of the test (Figure 4.28). This trend continued and the gradient ratio peaked after approximately 505 hours into the test, resulting in a value of 90. The test could have been stopped at this point, but it was decided to rather continue to see what the results of the gradient ratio would yield until the permeability had stabilised. There was a drop in gradient ratio up to the end of the test. The gradient ratio at the end of this test yielded a result of 34.1. The results of this test show that the geotextile experienced a high degree of clogging.

### 4.11 NON PLASTIC SAND (2) VS. WITH NW-N-SF-PP GEOTEXTILE: TEST NO. 10

Test no. 10 had a duration of 528 hours (Table 4.11). There was not much observed movement in the permeability readings over time. Readings in standpipes 5 and 6 stabilised at 504 hours and 384 hours respectively. The head reading across standpipes 1 and 7 was observed to be 100 mm.

**Table 4.11: Soil to geotextile permeability and standpipe readings vs. time: Test no. 10**

| Test Accumulative Hours | Quantity ml | Duration min | Permeability k m/s | Sample Height mm | Standpipe Readings - mm | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| | | | | | 300 | 250 | 200 | 150 | 100 | 50 | 0 |
| | | | | | Inlet | Silica Sand | | | Soil Sample | | Outlet |
| 1 | 305 | 60 | 1.079E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1555 | 1525 | 1470 |
| 24 | 290 | 60 | 1.026E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1555 | 1525 | 1470 |
| 48 | 400 | 60 | 1.415E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1555 | 1525 | 1470 |
| 120 | 800 | 60 | 2.829E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1545 | 1515 | 1470 |
| 144 | 800 | 60 | 2.829E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1545 | 1515 | 1470 |
| 168 | 820 | 60 | 2.900E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1545 | 1515 | 1470 |
| 192 | 845 | 60 | 2.989E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1545 | 1515 | 1470 |
| 216 | 835 | 60 | 2.953E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1545 | 1515 | 1470 |
| 288 | 765 | 60 | 2.706E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1545 | 1515 | 1470 |
| 312 | 770 | 60 | 2.723E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1545 | 1515 | 1470 |
| 336 | 760 | 60 | 2.688E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1545 | 1515 | 1470 |
| 360 | 725 | 60 | 2.564E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1548 | 1515 | 1470 |
| 384 | 700 | 60 | 2.476E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1520 | 1470 |
| 456 | 690 | 60 | 2.440E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1520 | 1470 |
| 480 | 670 | 60 | 2.370E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1550 | 1520 | 1470 |
| 504 | 670 | 60 | 2.370E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1547 | 1520 | 1470 |
| 528 | 670 | 60 | 2.370E-05 | 100 | 1570 | 1570 | 1570 | 1570 | 1547 | 1520 | 1470 |

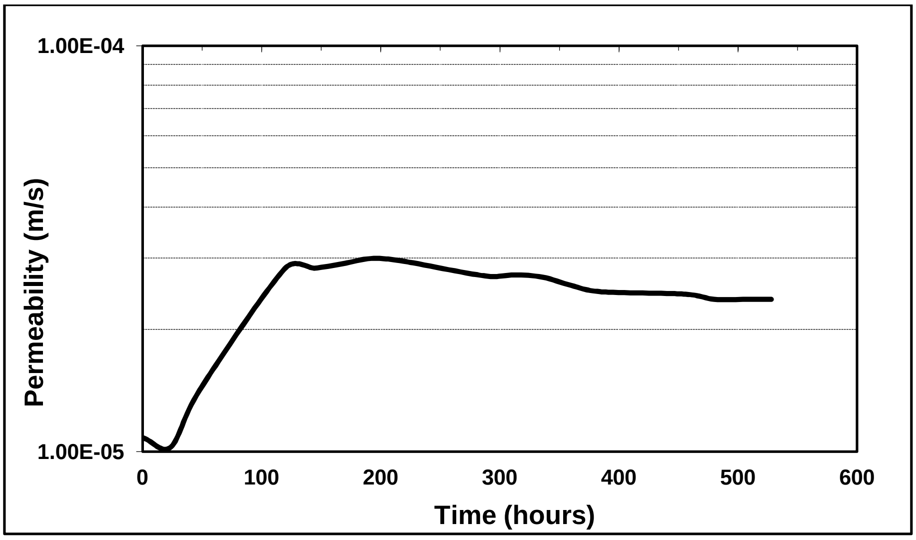



**Figure 4.29: Permeability vs. time of entire system: Test no. 10**

The permeability measured across the system was observed to increase at the beginning of the test and then, approximately 200 hours into the test, the permeability continued to decreaseslightly until the end of the test (Figure 4.29).

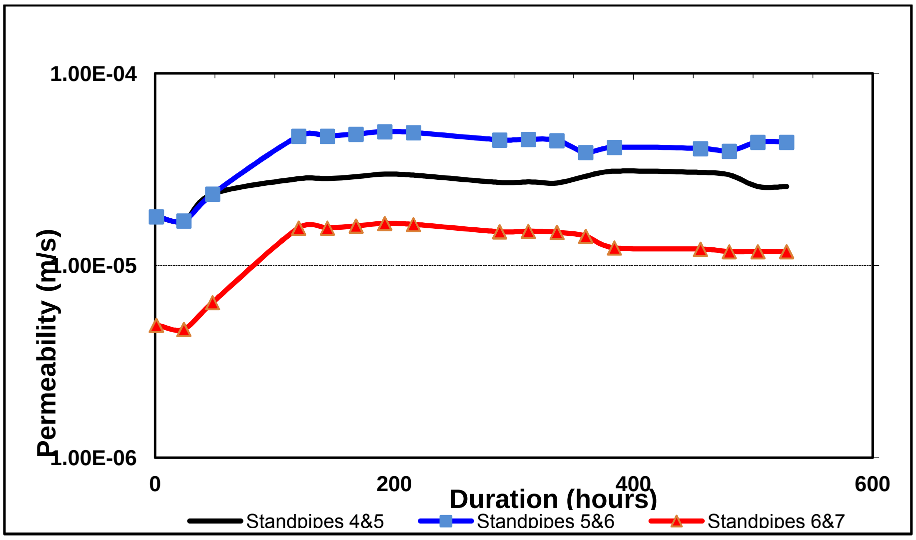



**Figure 4.30: Permeability vs. time across standpipes 4-5; 5-6 and 6-7: Test no. 10**

Similarly to test no. 9, the permeability across standpipes 6 and 7 in test no.10 had the biggest drawdown effect on the overall permeability performance of the entire system (Figure 4.30). Similar to test 9, the soil to geotextile permeability was observed to be much lower than that of the soil itself, which was problematic.

The gradient ratio for test no. 10 started out slightly high (Figure 4.31). The system then showed some recovery, whereby the gradient ratio dropped to below 3, which was within the desirable zone.

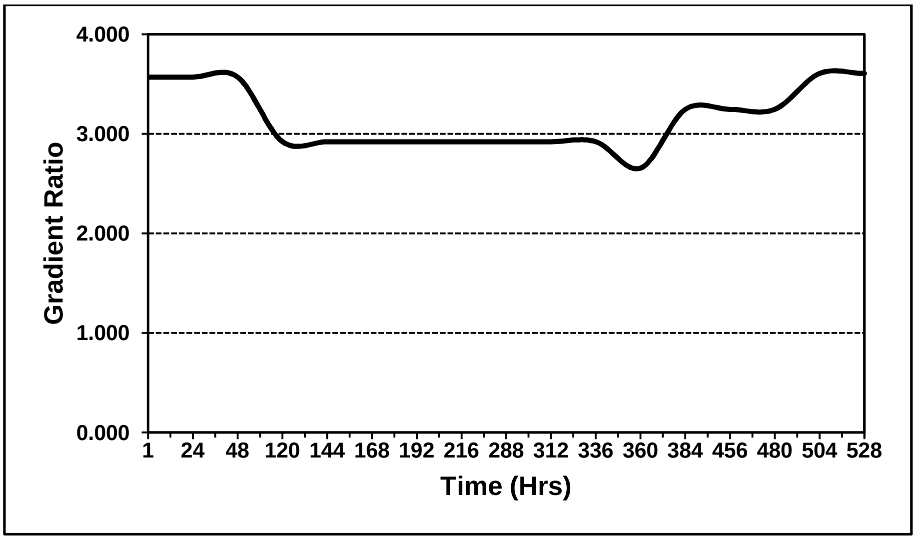



**Figure 4.31: Gradient ratio vs. time: Test no. 10**

After approximately 360 hours into the test, the gradient ratio increased above a level above 3. The gradient ratio ended at 3.604 which is an indication that the geotextile had a tendency to clog.

## 4.12 NON PLASTIC SAND (2) VS. WITH NW-HB-CF-PP GEOTEXTILE: TEST NO. 11

This test, similar to test no 9, lasted for 648 hours (Table 4.12). Standpipes 5 and 6 stabilised at 384 and 570 hours respectively. Except during the first hour, it was noted that there was no substantial difference in the permeability of the entire system at the start and at the end of the test. The head reading across standpipes 1 and 7 was observed to be 100 mm.

**Table 4.12: Soil to geotextile permeability and standpipe readings vs. time: Test no. 11**

| Test | | | Permeability | Sample | Standpipe Readings - mm | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| Accumulative | Quantity | Duration | k | Height | 300 | 250 | 200 | 150 | 100 | 50 | 0 |
| Hours | ml | min | m/s | mm | Inlet | Silica Sand | | | Soil Sample | | Outlet |
| 1 | 270 | 60 | 9.549E-06 | 100 | 1565 | 1565 | 1565 | 1565 | 1555 | 1525 | 1465 |
| 24 | 340 | 60 | 1.203E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1555 | 1525 | 1465 |
| 48 | 590 | 60 | 2.087E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1555 | 1525 | 1465 |
| 120 | 920 | 60 | 3.254E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1550 | 1510 | 1465 |
| 144 | 930 | 60 | 3.289E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1550 | 1510 | 1465 |
| 168 | 950 | 60 | 3.360E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1550 | 1510 | 1465 |
| 192 | 950 | 60 | 3.360E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1550 | 1510 | 1465 |
| 216 | 870 | 60 | 3.077E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1545 | 1510 | 1465 |
| 288 | 830 | 60 | 2.936E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1545 | 1515 | 1465 |
| 312 | 830 | 60 | 2.936E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1545 | 1515 | 1465 |
| 336 | 815 | 60 | 2.882E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1545 | 1515 | 1465 |
| 360 | 770 | 60 | 2.723E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1545 | 1515 | 1465 |
| 384 | 740 | 60 | 2.617E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1550 | 1520 | 1465 |
| 456 | 700 | 60 | 2.476E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1550 | 1520 | 1465 |
| 480 | 680 | 60 | 2.405E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1550 | 1520 | 1465 |
| 504 | 670 | 60 | 2.370E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1550 | 1525 | 1465 |
| 528 | 650 | 60 | 2.299E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1550 | 1525 | 1465 |
| 600 | 570 | 60 | 2.016E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1560 | 1535 | 1465 |
| 624 | 570 | 60 | 2.016E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1560 | 1535 | 1465 |
| 648 | 570 | 60 | 2.016E-05 | 100 | 1565 | 1565 | 1565 | 1565 | 1560 | 1535 | 1465 |

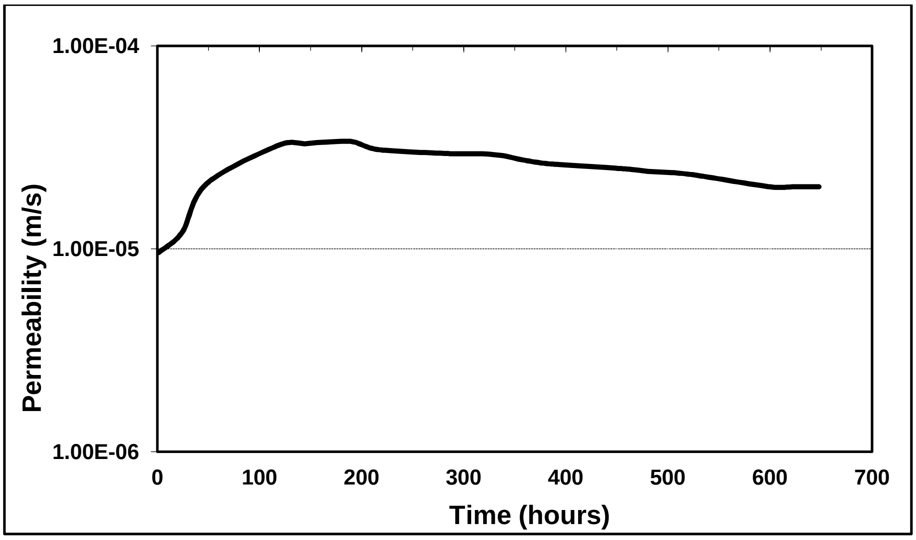



**Figure 4.32: Permeability vs. time of entire system: Test no. 11**

An increase in permeability was observed at the start of the test (Figure 4.32). A slow reduction in permeability was observed to the end of the test.

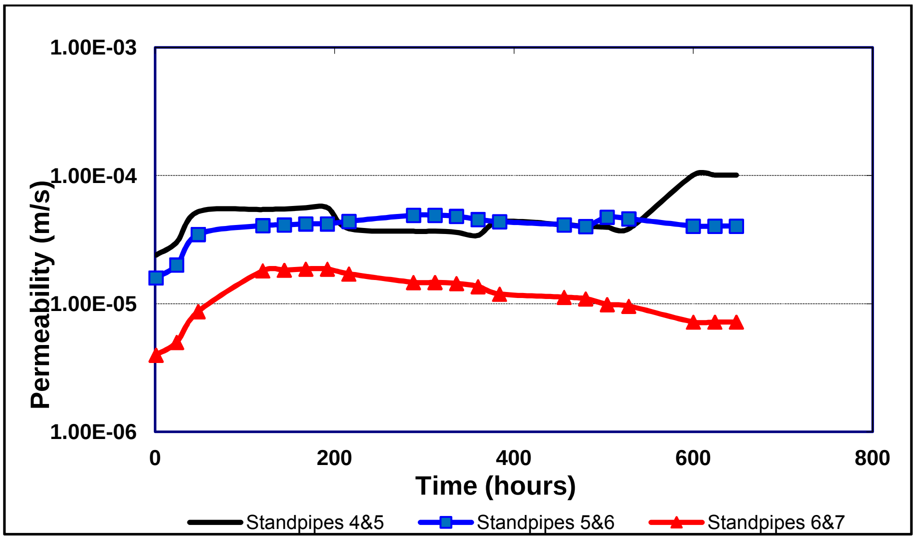



**Figure 4.33: Permeability vs. time across standpipes 4-5; 5-6 and 6-7: Test no. 11**

As with tests 9 and 10, the permeability across standpipes 6 and 7 had the greatest drawdown effect of the system's permeability (Figure 4.33). The permeability across these standpipes was observed to be 1 order of magnitude lower than that measured across standpipes 4 and 5, and 5 and 6. This is indicative that the soil to geotextile permeability was lower than that of the permeating soil and this is classified as a failure of the geotextile. The soil to geotextile permeability was observed to be much lower than that of the soil itself, and this is problematic.

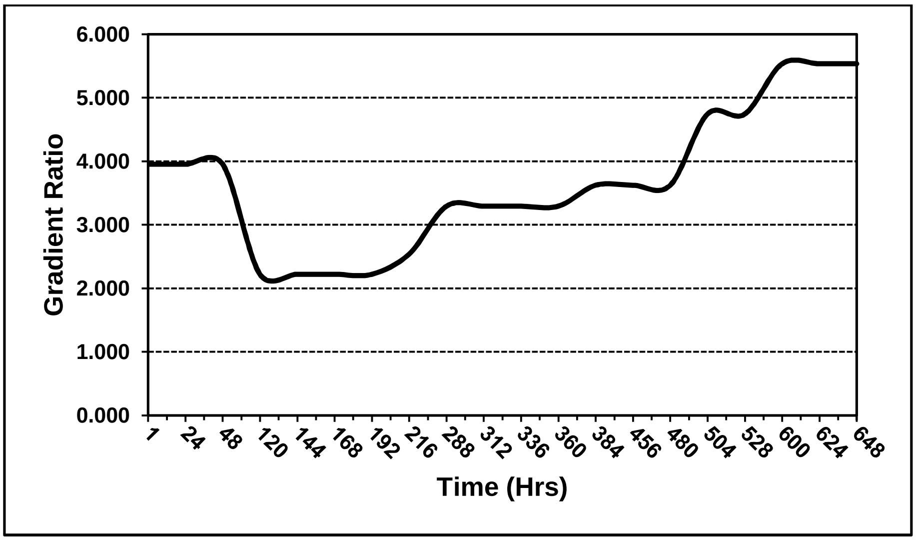



**Figure 4.34: Gradient ratio vs. time: test no 11**

The gradient ratio observed for this test started at approximately 4.000 (Figure 4.34). The system reached a desirable gradient ratio, below 3, after approximately 120 hours. Afterwards it was observed that the gradient ratio increased in stages to reach an undesirable figure of 5.534 at the end of the test. From this test, is evident that the geotextile showed a tendency to clog.

### 4.13 NON PLASTIC SAND (2) VS. WITH W-SLF-PP GEOTEXTILE: TEST NO. 12

The duration of the final test was also 672 hours (Table 4.13). It was observed that the test ended with a similar quantity of water flowing through the system as had at the start. Both head readings in standpipes 5 and 6 stabilised after approximately 600 hours.

**Table 4.13: Soil to geotextile permeability and standpipe readings vs. time: Test no. 12**

| Test Accumulative Hours | Quantity ml | Duration min | Permeability k m/s | Sample Height mm | Standpipe Readings - mm | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| | | | | | 300 | 250 | 200 | 150 | 100 | 50 | 0 |
| | | | | | Inlet | Silica Sand | | | Soil Sample | | Outlet |
| 1 | 250 | 60 | 8.842E-06 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1523 | 1460 |
| 24 | 195 | 60 | 6.897E-06 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1525 | 1460 |
| 48 | 210 | 60 | 7.427E-06 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1525 | 1460 |
| 120 | 510 | 60 | 1.804E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1545 | 1520 | 1460 |
| 144 | 570 | 60 | 2.016E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1545 | 1520 | 1460 |
| 168 | 590 | 60 | 2.087E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1545 | 1520 | 1460 |
| 192 | 590 | 60 | 2.087E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1545 | 1520 | 1460 |
| 216 | 550 | 60 | 1.945E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1545 | 1515 | 1460 |
| 288 | 480 | 60 | 1.698E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1545 | 1520 | 1460 |
| 312 | 480 | 60 | 1.698E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1545 | 1520 | 1460 |
| 336 | 460 | 60 | 1.627E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1545 | 1520 | 1460 |
| 360 | 425 | 60 | 1.503E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1545 | 1520 | 1460 |
| 384 | 400 | 60 | 1.415E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1525 | 1460 |
| 456 | 360 | 60 | 1.273E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1525 | 1460 |
| 480 | 340 | 60 | 1.203E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1535 | 1460 |
| 504 | 330 | 60 | 1.167E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1535 | 1460 |
| 528 | 310 | 60 | 1.096E-05 | 100 | 1560 | 1560 | 1560 | 1560 | 1550 | 1535 | 1460 |
| 600 | 260 | 60 | 9.196E-06 | 100 | 1560 | 1560 | 1560 | 1560 | 1555 | 1540 | 1460 |
| 624 | 245 | 60 | 8.665E-06 | 100 | 1560 | 1560 | 1560 | 1560 | 1555 | 1540 | 1460 |
| 648 | 245 | 60 | 8.665E-06 | 100 | 1560 | 1560 | 1560 | 1560 | 1555 | 1540 | 1460 |
| 672 | 245 | 60 | 8.665E-06 | 100 | 1560 | 1560 | 1560 | 1560 | 1555 | 1540 | 1460 |

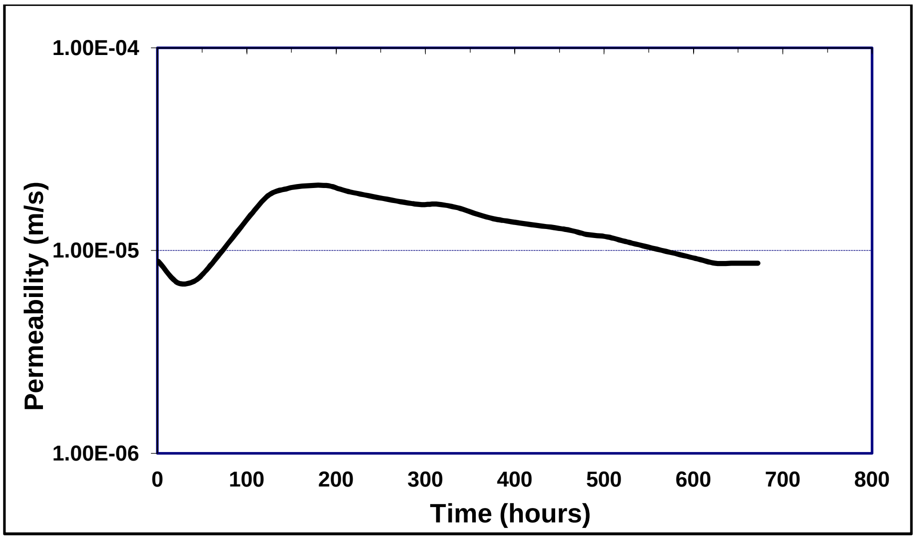



**Figure 4.35: Permeability vs. time of entire system: Test no. 12**

It was observed that that there was an increase in permeability up to approximately 200 hours. Thereafter, up to approximately 650 hours, a slight decrease in permeability was observed.

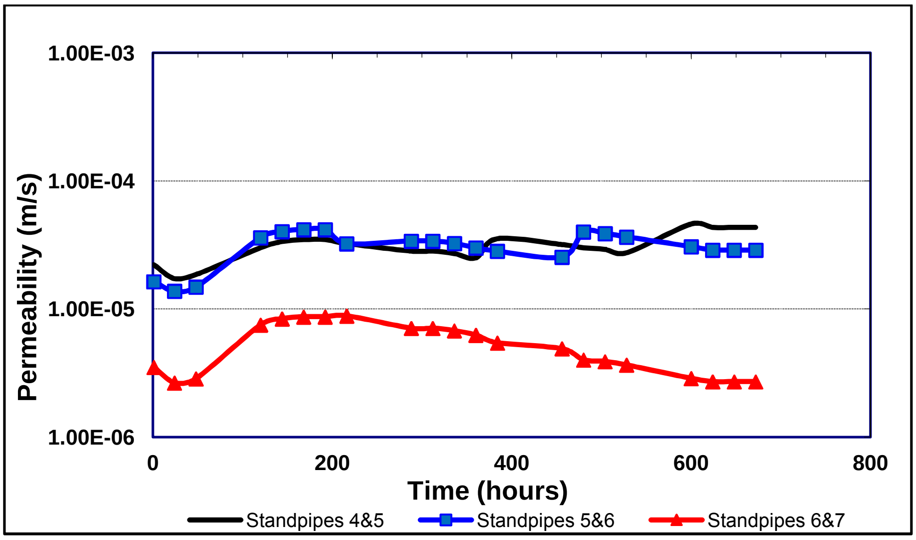



**Figure 4.36: Permeability vs. time across standpipes 4-5; 5-6 and 6-7: Test no. 12**

The measured permeability across standpipes 6 and 7 was approximately one order of magnitude lower than that measured across standpipes 4 and 5, and 6 and 7. The soil to geotextile permeability was observed to be much lower than the soil itself and this is problematic. The same trend was observed for all of the last four tests.

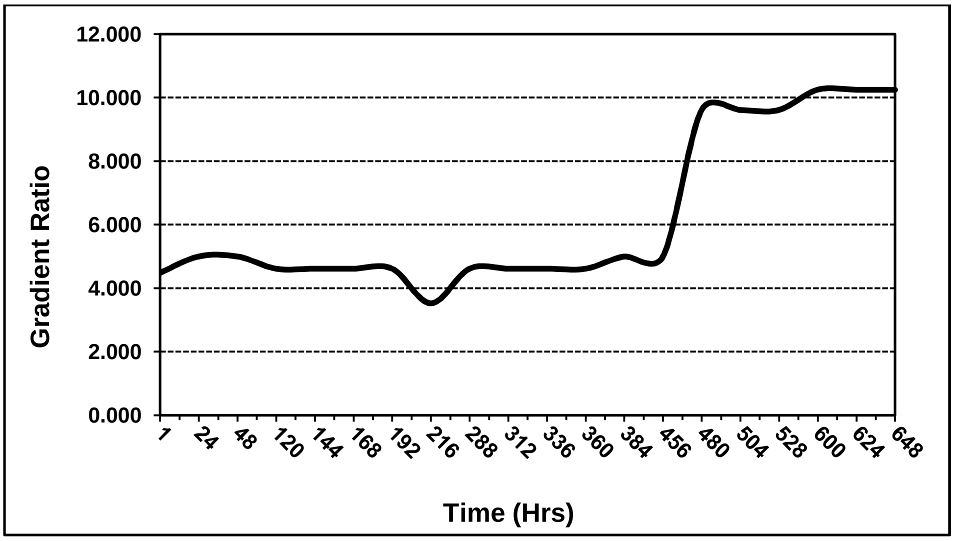
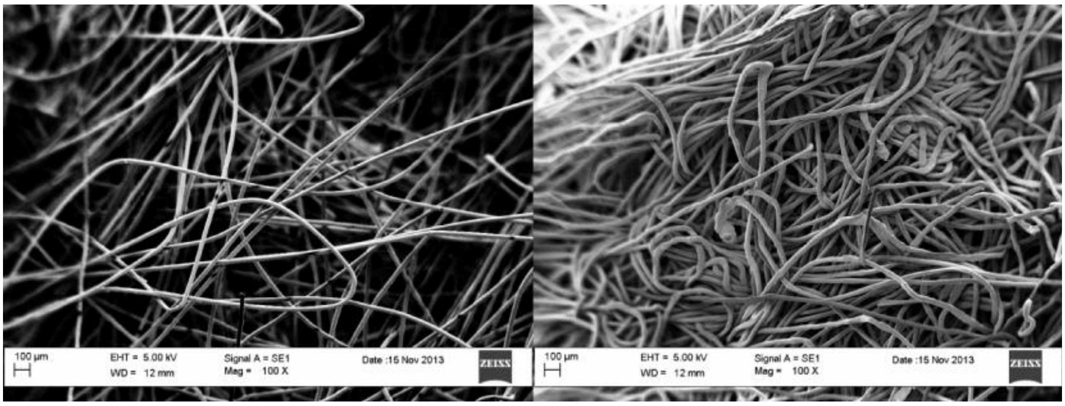



**Figure 4.37: Gradient ratio vs. time: test no 12**

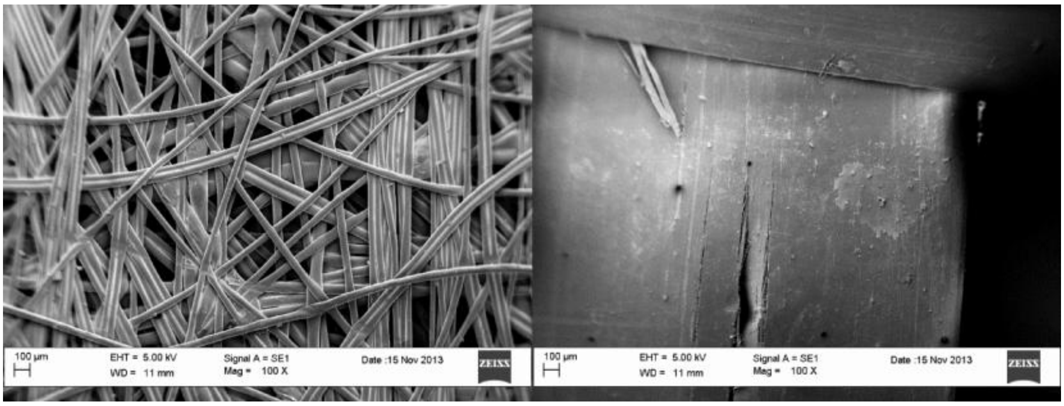
The gradient ratio remained fairly constant for approximately 456 hours (Figure 4.37). It was observed that the gradient ratio increased at the same time at which a higher permeability across the entire system was encountered. The gradient ratio at the end of this test was observed to be 10.250. This is evidence of the geotextile showing signs of a high degree of clogging.

## 4.16 MICRO ELECTROSCOPIC IMAGES

After all permutations of the permeameter tests had been completed, the soil contaminated candidate geotextiles were sent for micro imaging. A set of micro images was also obtained of the uncontaminated geotextile samples. These visuals are presented below.



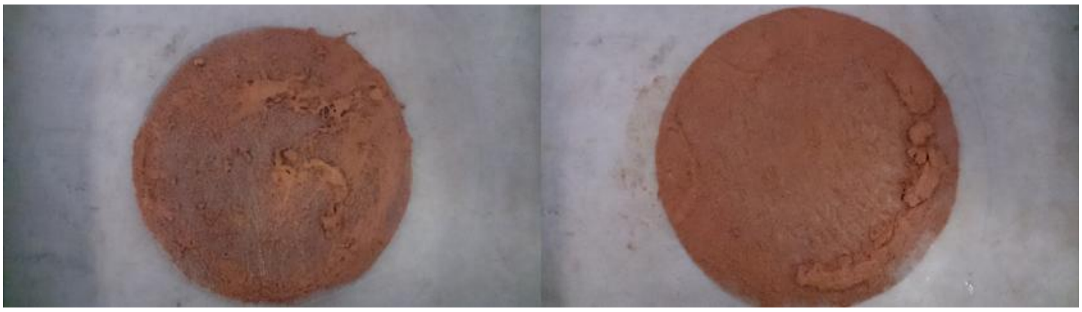

**A**=NW-N-CF-PET **B**=NW-N-SF-PP



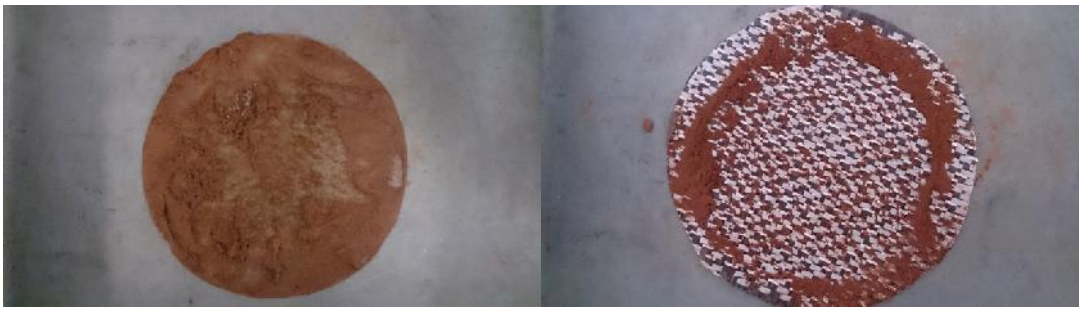

**C**= NW-HB-CF-PP **D**= W-SLF-PP

**Figure 4.38: Electro microscopic images of the geotextiles before tests:**

Figure 4.38 illustrates all the geotextile in its uncontaminated form. It can be seen that the fibre diameter for the geotextile in image A is much finer than that in image B. The geotextile in image A also has the highest porosity and pore opening size of all the candidate geotextiles. The pore opening size of the geotextile in image C was the second largest. The geotextile in image C has a flatter fibre, because of the heat bonding process. The woven slit film geotextile can be clearly seen in image D. It has a closed weave structure. Water and particles would be able to pass through the openings only at the weave overlaps.



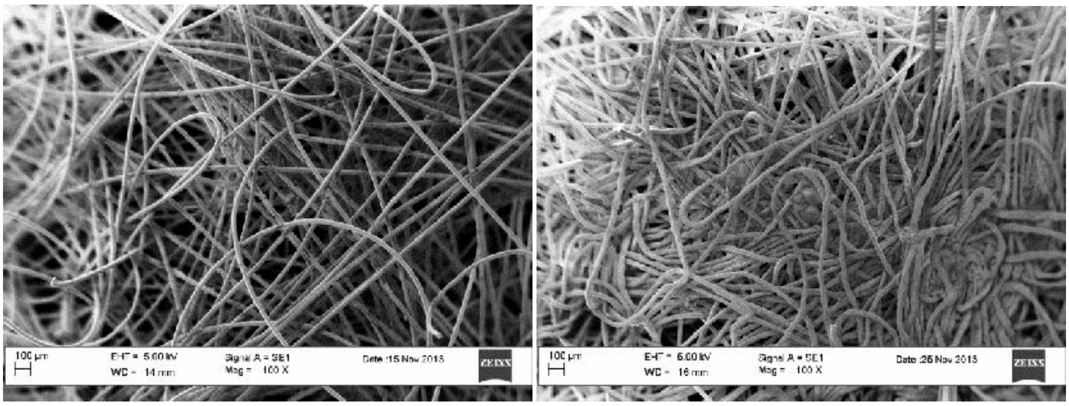
**E-1: NW-N-CF-PET** **F-1: NW-N-SF-PP**



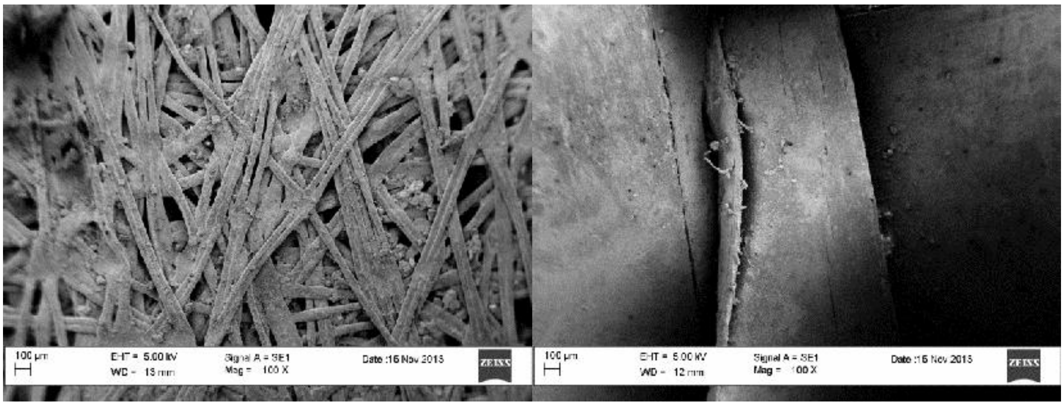
**G-1: NW-HB-CF-PP** **H-1: W-SLF-PP**

**Figure 4.39: Post-test photographic images of geotextiles: tests 1-4**

Figure 4.39 shows photographic images of the geotextiles just after the tests had finished. Some soil contamination was observed for the NW-N-CF-PET geotextile. It was observed that the heavier soil contamination occurred on the NW-N-SF-PP as well as the NW-HB-CF-PP geotextiles. It was also evident that the soil particles migrated towards the gaps at the slit film overlaps for the W-SLF-PP geotextile. A ring formation or the remnants of an outer soil ring were visible on the geotextiles. This should be ignored, as the ring is formed at the permeameter clamped area, which falls outside the affected area of the tested geotextile .



**E: NW-N-CF-PET** **F: NW-N-SF-PP**

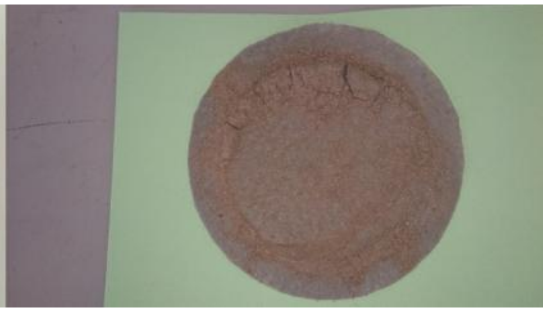


**G: NW-HB-CF-PP** **H: W-SLF-PP**

**Figure 4.40: Post-test microscopic images of geotextiles at 100 x magnification: tests 1-4**

Figure 4.40 shows the visible levels of contamination for the geotextiles in image F and G.

The geotextiles in image E and H show little or no contamination. The particles in image H can be seen conglomerating in the openings at the weave overlaps.

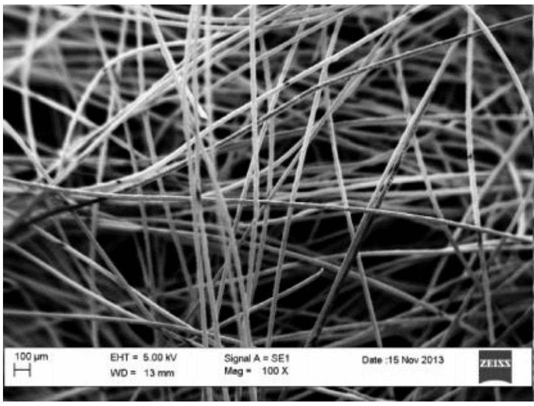


**I-1: NW-N-CF-PET** **J-1: NW-N-SF-PP**

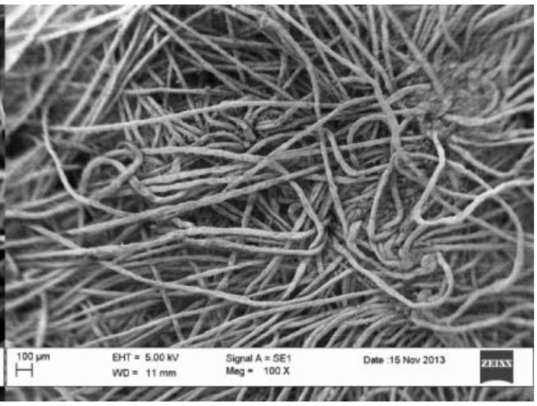
**K-1: NW-HB-CF-PP** **L-1: W-SLF-PP**

**Figure 4.41: Post-test photographic images of geotextiles: tests 5-8**

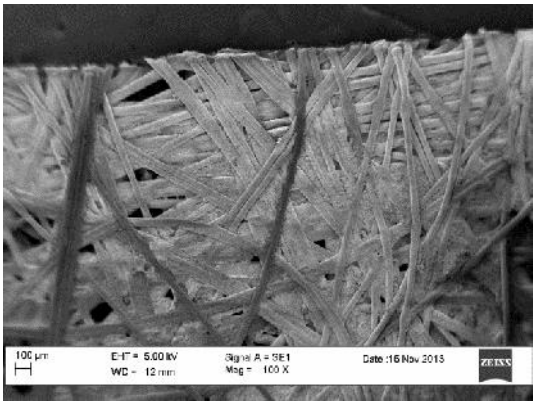
Photographs I-1 and L-1 show partial soil contamination (Figure 4.41. Photographs J-1 and K-1 shows a heavier degree of contamination. Photo J-1 was taken after oven drying.



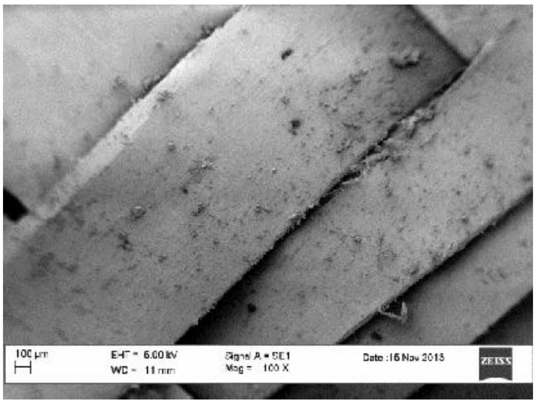

I: NW-N-CF-PET




J: NW-N-SF-PP



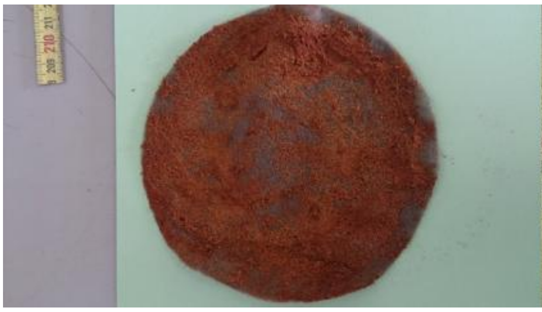

K: NW-HB-CF-PP

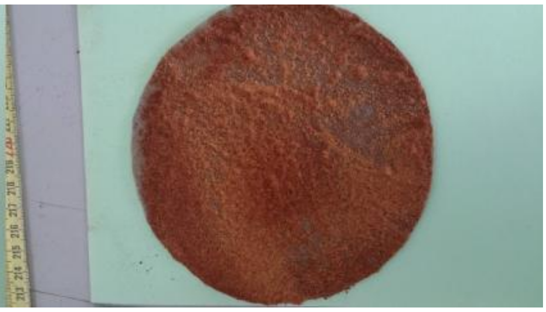



L: W-SLF-PP

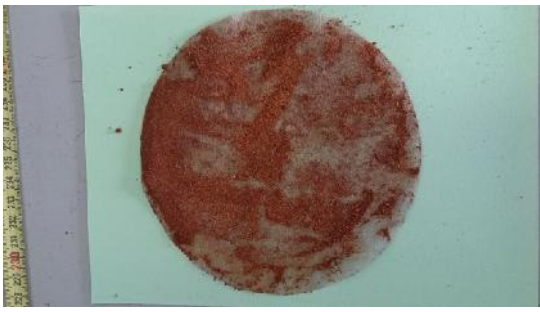
**Figure 4.42: Post-test microscopic images of geotextiles at 100x magnification: tests 5-8**

Images J and K confirm that a higher number of soil particles has been trapped within the geotextile fabric. As a result, the geotextiles in image J and K had a contaminated residual permeability that was less than that of the soil. The geotextiles in images I and L showed less contamination and as a result were acceptable as geotextile filters.





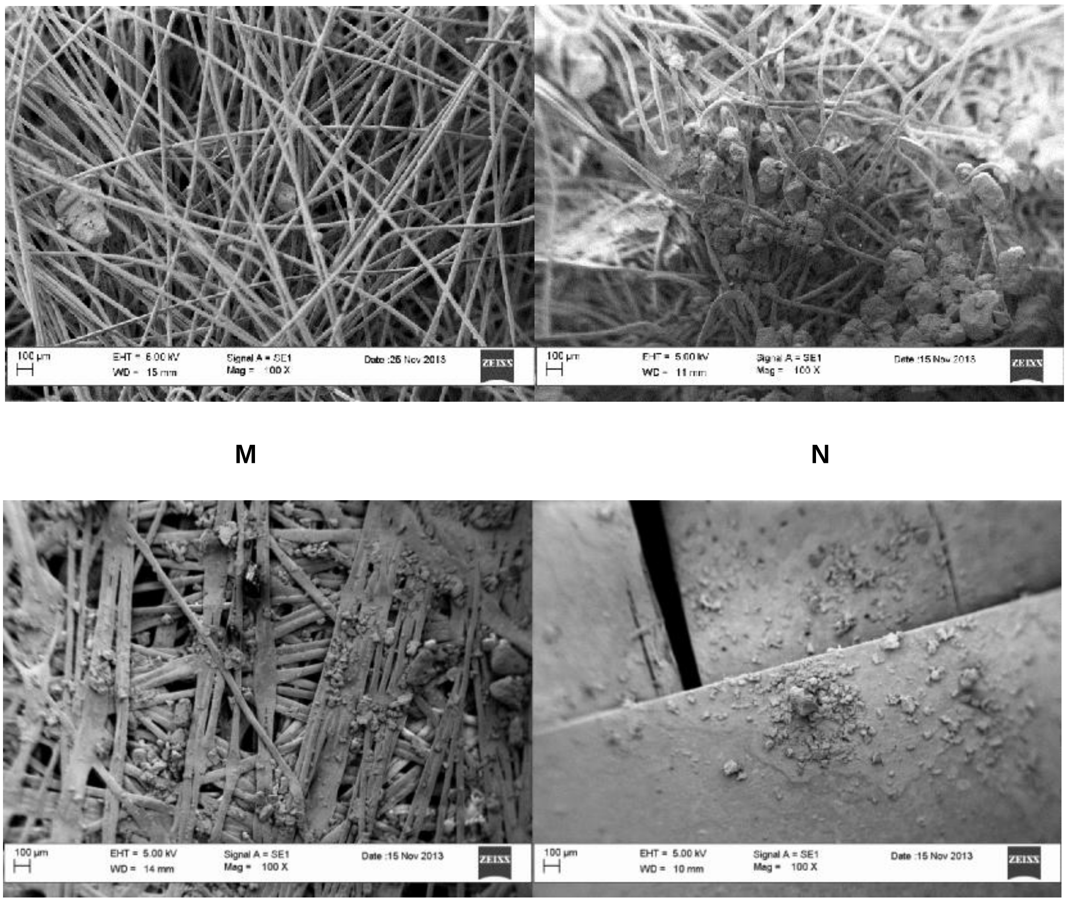
**M-1** **N-1**



**O-1** **P-1**

**Figure 4.43: Post-test photographic images of geotextiles: tests 9-12**

It can be seen in the photographic images that all the geotextiles suffered high degrees of contamination. These geotextiles experienced a high degree of clogging and are therefore not acceptable as filters in conjunction with the soil they were tested with.



M N

O P

**Figure 4.44: Post-test microscopic images of geotextiles at 100x magnification: tests 9-12**

From the images in Figure 4.44, it can be deduced that the pore openings of all these geotextiles were clogged. This was also evident in the high gradient ratios obtained for this set of tests, as well as the low soil to geotextile permeability. More soil residue is also evident on the woven geotextile, near the overlap openings in image P.

## 4.17 COMPUTER SPREADSHEET

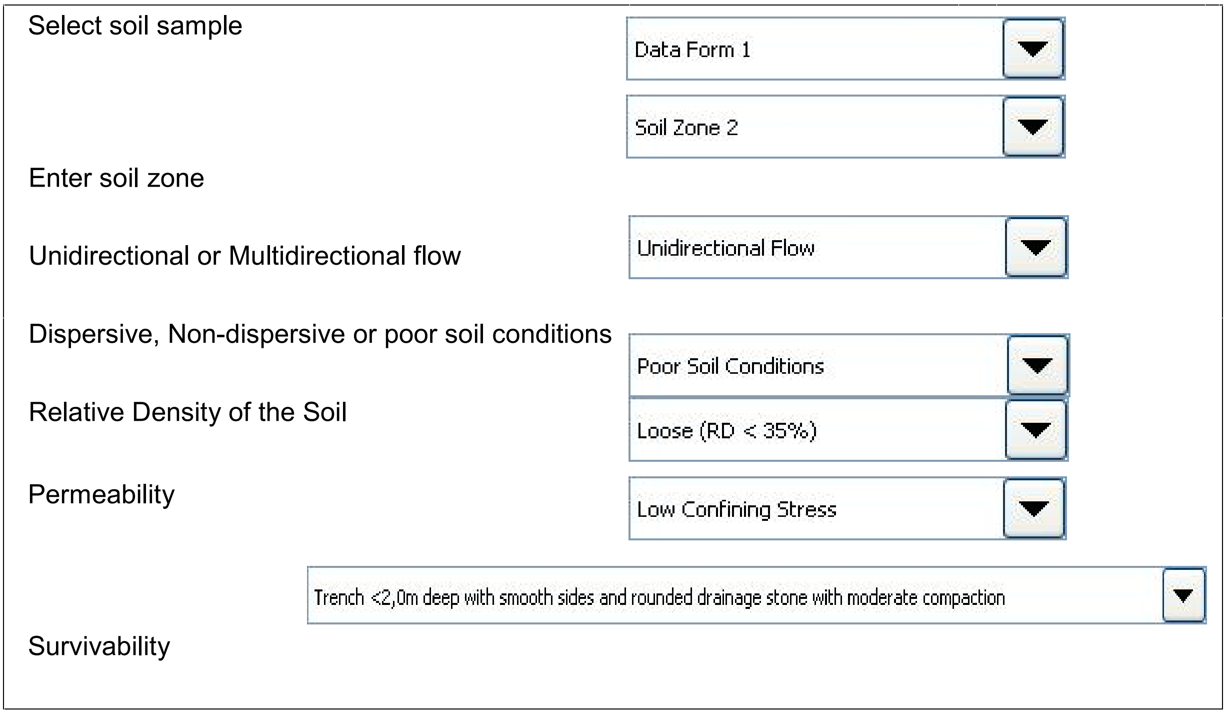



**Figure 4.45: Input parameters for computer spreadsheet**

The input parameters that were used for the computer program are shown in figure 4.45. The input parameters remained the same for all three soil types as these were very similar. The soil was classified as Zone 2 (Luettich *et al*., 1992) and problematic as far as geotextiles are concerned. The computer spread sheet was adapted from Luettich *et al.* (1992); however, it does not take the soil's co-efficient of uniformity into account.

**Table 4.14: Output with recommended geotextile specifications**

| Recommended Geotextile | | Test Method | Units | Recommended Specification |
|---|---|---|---|---|
| Trapezoidal Tear | Across | ASTM D4533 | N | 175 |
| CBR | 50 mm probe | SANS 10221-2007 | kN | 1 |
| Dart Test | Dia. of hole | EN ISO 13433-2006 | mm | 32 |
| Tensile Strength | Across | SANS 10221-2007 | kN/m | 7 |
| Permeability | @ 100 mm head | SANS 10221-2008 | m/s | $1 \times 10^{-4}$ |
| Pore size | $O_{95}$ | EN ISO 12956-1999 | µm | 100µm<$O_{95}$>250µm |

After the input parameters have been inserted as shown in Figure 4.45, the spreadsheet program recommends 6 parameters (Table 4.14). These parameters are further explained as follows:

### 4.17.1 Trapezoidal Tear

This is the resistanceto tear, measured in Newtons (N), when a geotextile is clamped at diagonally opposite corners and a strain applied to it. It is a parameter which is linked to the survivability of the geotextile during installation.

### 4.17.2 CBR

This is the resistance, measured in kilo Newtons (kN), when a probe of a certain diameter is pushed perpendicularly through the plane of the geotextile. This parameter is linked to the survivability requirement of the geotextile.

### 4.17.3 Dart test

A probe of a certain diameter and weight is dropped from a certain height onto the geotextile. The diameter of the hole is measured in mm. This test is also linked to survivability.

### 4.17.4 Tensile strength

A strip of geotextile is clamped on opposite sides and strained at a certain rate. The resistance to rupture is measured in kilo Newtons per meter (kN/m). This is a common test done on geotextiles and is related to survivability.

### 4.17.5 Permeability

This parameter is the measure of the quantity of water, measured in m/s, which flows perpendicularly through the plane of the geotextile. This is an important parameter, as it must always be higher than that of the adjacent soil.

### 4.17.6 Pore size

This is the size of the openings (pore size) of the geotextile. This parameter is important and linked to the retention criteria. The pore size must be small enough to retain the sand particles and large enough to let water flow through it.

**Table 4.15: Summary of geotextile to soil performance**

| Test Number | Criteria met in terms of clogging | Criteria met in terms of permeability | Comments |
|---|---|---|---|
| 1 | Y | Y | Pass |
| 2 | Y | Y | Pass |
| 3 | N | N | Fail |
| 4 | Y | Y | Pass |
| 5 | Y | Y | Pass |
| 6 | Y | N | Fail |
| 7 | Y | N | Fail |
| 8 | Y | Y | Pass |
| 9 | N | N | Fail |
| 10 | N | N | Fail |
| 11 | N | N | Fail |
| 12 | N | N | Fail |

From all the test permutations performed it can be seen that the results of only four of the twelve tests have been acceptable (Table 4.15). This proves the Berea sand to be very challenging, as it is a highly variable soil in terms of its engineering properties.

## 4.18 CONCLUSION

The outcomes of the 12 tests performed in the laboratory have provided a greater understanding of the soil to geotextile filtration behaviour when geotextiles commercially available in South Africa are used in conjunction with some of the Berea sands encountered along the KwaZulu-Natal coastline. The results of these tests are analysed and discussed in further detail in the next chapter. Laboratory observations and outcomes allow for the following preliminary conclusions:

i) No two of the geotextiles tested behaved identically when exposed to the same soil type.
ii) The non-plastic Berea sands used in tests nos. 1-4 proved to be problematic when used with some of the geotextiles.
iii) Berea sands with a relatively high plasticity index, used in tests nos. 5-8, did not pose a threat in terms of clogging to the geotextiles tested.
iv) The non-plastic Berea sands used in tests nos. 9-12 proved to be problematic in terms of clogging with all of the geotextiles tested.
v) None of the geotextiles met the criteria regarding the number of constrictions being $20>N_s<40$, as proposed by Giroud (1996). However, at most times, the geotextiles behaved satisfactorily.
vi) The coefficients of uniformity (Cu) of all the soils tested showed too much variation and it was therefore inconclusive what effect the Cu had on all the soil to geotextile test outcomes.

In the following chapter, the results obtained will be analysed in more detail and discussed.

# Chapter 5 - Analysis

## 5.1 INTRODUCTION

This chapter will attempt to analyse the laboratory results obtained to add more meaning. In addition to interpreting the results of the laboratory tests, this chapter also aims at the following:

i) Meeting the primary objectives as set out in Chapter 1 of this thesis.
ii) Evaluating the currently available geotextile filter design criteria in the South African context and adding meaningful contributions towards the way forward.

## 5.2 SOIL INTERNAL STABILITY AND CO-EFFICIENT OF UNIFORMITY.

Terzaghi's criterion was used to assess the soil's inherent internal stability by using the following formula:

$$\mathbf{d_{85} / d_{15} > 5} \text{ (soil is internally stable)}$$

The co-efficient of uniformity ($C_u$) was calculated for all soils using the formula:

$$\mathbf{C_u = d_{60}/d_{10}}$$

Similarly, the co-efficient of curvature ($C_c$) was calculated for all soils using the formula:

$$\mathbf{C_c = d_{30}^2 / (d_{30}.d_{60})}$$

The results which were obtained are illustrated in Table 5.1 below.

**Table 5.1: Inherent stability of soil**

| **Soil Sample No** | **$d_{85}/d_{15}$** | **$C_u$** | **$C_c$** | **Plasticity Index** |
|---|---|---|---|---|
| 1 | 7.17 | 8 | 2.341 | Non Plastic |
| 2 | 20.62 | 95 | 25.25 | 7% |
| 3 | 32.69 | 111 | 11.95 | Non Plastic |

The soils all seemed to have inherent stability. There was a big variance in the results obtained for the co-efficient of uniformity ($C_u$).

## 5.3 NUMBER OF CONSTRICTIONS

The number of constrictions for the geotextiles was calculated after Giroud (1996):

$$N_s = \sqrt{(1-P)} \times (\text{geotextile thickness/fibre diameter})$$

The results are tabulated in Table 5.2 below.

**Table 5.2: Number of constrictions (Ns) for the candidate geotextiles**

| Geotextile Type | Porosity (%) | Thickness (mm) (under 2kPa) | Fibre Diameter (μm) | $N_s$ | Notes |
|---|---|---|---|---|---|
| **NW-N-CF-PET** | 93 | 1.20 | 32 | 10 | $N_s$ <20 |
| **NW-N-SF-PP** | 65 | 0.80 | 60 | 8 | $N_s$ <20 |
| **NW-HB-CF-PP** | Not Applicable | 0.43 | 45 | 5 | $N_s$ <20 |
| **W-SLF-PP** | Not Applicable | Not applicable | Not Applicable | Not applicable | - |

It was noted that none of the geotextiles used in this experiments had met Giroud's criteria, according to which the number of constrictions (Ns) should preferably be between 20 and 40 if the geotextile is used as a filter.

## 5.4 SYSTEM PERMEABILITY VS. SOIL TO GEOTEXTILE PERMEABILITY

At the end of each of the tests, the soil's permeability was determined by the permeability calculated across standpipes 5 and 6. This was compared to that of the soil to geotextile system, which was measured across standpipes 6 and 7 in all the tests. The permeability across standpipes 6 and 7 was indicative of the soil to geotextile permeability. If this permeability is lower than that of the soil, then the soil to geotextile system has failed.

The soil to geotextile permeability should always be more than that of the soil in order not to have potential pore water pressure building up adjacent to the geotextile. Therefore the ratio of the permeabilities $k_{6\text{-}7}$ / $k_{5\text{-}6}$ should always be a minimum of 1. The value of the gradient ratio should be between 1 and 3 in order for it to be acceptable against clogging. A value of less than 1 suggests that piping has occurred. A gradient ratio value of higher than 3 indicates that clogging of the geotextile has occurred, which is unacceptable.

### 5.4.1 Tests 1-4

**Table 5.3: Soil to geotextile permeability vs. sand permeability with gradient ratios at end of tests**

| Test no. | Soil sample no. | $k_{5\text{-}6}$: Soil permeability (m/s) | $k_{6\text{-}7}$: Across standpipes 6-7 (m/s) | $k_{6\text{-}7}$ / $k_{5\text{-}6}$ | Gradient Ratio |
|---|---|---|---|---|---|
| **1** | 1 | 1.326E-06 | 1.516E-06 | 1.143 | 0.846 |
| **2** | 1 | 9.431E-07 | 5.053E-06 | 5.358 | 0.180 |
| **3** | 1 | 4.113E-06 | 2.947E-06 | 0.717 | 3.075 |
| **4** | 1 | 2.669E-06 | 1.415E-05 | 5.300 | 2.436 |

The soil to geotextile permeabilities for tests 2 and 4 were noticeably higher than those of the soil itself (Table 5.3). This is not necessarily the case for the gradient ratios. It was also observed that the soil to geotextile permeability for test no.1 was marginally higher than that of the soil, given that this test yielded satisfactory results in terms of its gradient ratio. Test no 3 shows that the soil to geotextile permeability was lower than that of the soil itself, therefore making it undesirable.

From this set of tests it was evident that one cannot look at the gradient ratio in isolation but need also to determine the permeability at the soil to geotextile interface. Testnumber 3 had an acceptable gradient ratio, but fell short on the soil to geotextile permeability, which was less than 1 and therefore a failure as a result. The geotextile used in test 2 had the smallest opening size of all the geotextiles, but showed the highest degree of piping, as well as soil to geotextile permeability. This could possibly have been caused by concentrated flow at one point through the geotextile. This set of tests yielded only one geotextile failure.

### 5.4.2 Tests 5 – 8

**Table 5.4: geotextile permeability vs. sand permeability with gradient ratios at end of tests**

| Test no. | Soil sample no. | $k_{5\text{-}6}$: Soil permeability (m/s) | $k_{6\text{-}7}$: Across standpipes 6-7 (m/s) | $k_{6\text{-}7}$ / $k_{5\text{-}6}$ | Gradient Ratio |
|---|---|---|---|---|---|
| **5** | 2 | 2.173E-05 | 3.802E-05 | 1.750 | 0.542 |
| **6** | 2 | 1.997E-05 | 1.629E-05 | 0.816 | 1.193 |
| **7** | 2 | 5.659E-05 | 2.201E-05 | 0.389 | 2.537 |
| **8** | 2 | 3.741E-05 | 4.863E-05 | 1.300 | 0.751 |

From these tests, it can be seen that all geotextiles behaved satisfactorily in terms of the gradient ratios obtained (Table 5.4). Tests 6 and 7 showed permeabilities across standpipes 6 and 7 that were lower than those across standpipes 5 and 6.

Once again, although the gradient ratios observed for this set of tests were all acceptable, the soil to geotextile permeability was the deciding factor in determining whether the geotextile was suitable or not. In the case of test 6, although the soil to geotextile permeability was slightly below 1 it is left to the engineer's discretion to decide whether the application would be critical or not. The geotextiles used in conjunction with the slightly plastic Berea sand thus yielded unacceptable results related to the soil to geotextile permeability criteria, rather than to clogging.

### 5.4.3 Tests 9 – 12

**Table 5.5: geotextile permeability vs. sand permeability with gradient ratios at end of tests**

| Test no. | Soil sample no. | $k_{5\text{-}6}$: Soil permeability (m/s) | $k_{6\text{-}7}$: Across standpipes 6-7 (m/s) | $k_{6\text{-}7}$ / $k_{5\text{-}6}$ | Gradient Ratio |
|---|---|---|---|---|---|
| **9** | 3 | 2.476E-05 | 6.877E-07 | 0.028 | 34.100 |
| **10** | 3 | 4.388E-05 | 1.185E-05 | 0.270 | 3.604 |
| **11** | 3 | 4.032E-05 | 7.200E-06 | 0.179 | 5.534 |
| **12** | 3 | 2.888E-05 | 2.708E-06 | 0.094 | 10.250 |

From this set of tests it was noticed that all the geotextiles had relatively high gradient ratios (Table 5.5). The permeabilities across standpipes 6 and 7 were considerably lower than those measured across standpipes 5 and 6. This means that all the geotextiles used with this soil were inadequate to be used as filters.

The high gradient ratios are indicative of a high degree of clogging of the geotextile. The results of the soil to geotextile permeabilities were all well below those of the soil, making all these geotextile unacceptable. The sand tested was cohesionless and had a high $C_u$. Migration of fines through the soil body had occurred. A sufficient number of fines became entrapped in the geotextile to render it clogged. In the case of the woven geotextile, it was considered blocked rather than clogged.

## 5.5 SUMMARY OF GRADIENT RATIOS VS. TIME

### 5.5.1 Test numbers 1 - 4

#### *5.5.1.1 Test No.1: Nonwoven, needle-punched continuous filament polyester geotextile (NW-N-CF-PET) vs. non plastic Berea sand*

During the test this geotextile performed relatively well, as the system did not show excessive build-up of gradient ratio nor a significant drop in gradient ratio (Table 5.6). At the start of the test there was a slight rise in gradient ratio, which suggests a tendency for the geotextile to clog.

**Table 5.6: Summary of gradient ratios vs. time: test nos. 1-4**

| Time (hours) | Test No 1 Gradient ratios | Test No 2 Gradient ratios | Test No 3 Gradient ratios | Test No 4 Gradient ratios |
|---|---|---|---|---|
| 1 | 0.737 | 0.283 | 8.687 | 2.896 |
| 8 | 1.018 | 0.283 | 8.687 | 2.896 |
| 72 | 0.719 | 0.244 | 7.128 | 5.441 |
| 96 | 0.457 | 0.251 | 7.128 | 5.441 |
| 120 | 0.290 | 0.268 | 4.344 | 3.197 |
| 144 | 0.846 | 0.268 | 3.475 | 2.413 |
| 168 | 0.846 | 0.180 | 3.475 | 2.413 |
| 240 | | 0.180 | 3.075 | 1.972 |
| 264 | | 0.180 | 3.075 | 2.436 |
| 288 | | 0.180 | 3.075 | 2.436 |
| 312 | | | 3.075 | 2.436 |
| 336 | | | 3.075 | 2.436 |
| 408 | | | 3.075 | 2.436 |
| 432 | | | 3.075 | 2.436 |
| 456 | | | 3.075 | 2.436 |
| 480 | | | 3.075 | 2.436 |
| 504 | | | 3.075 | |

The system showed some quick recovery by the gradient ratio dropping slightly below 1. This suggests that some of the finer soil particles which were either lodged in the geotextile or at the soil to geotextile interface were pushed through or out of the geotextile as water continued to flow through the soil to geotextile interface. The test also equalised most quickly, t which was indicative that a natural soil filter had formed most quickly on the upstream side of the geotextile. According to the gradient ratio results obtained, no tendency towards clogging and piping could be established. As ascertained in the previous chapter, the soil to geotextile permeability was higher than that of the soil, which satisfied the permeability criterion. The geotextile to soil compatibility for this test was therefore found to be acceptable.

#### *5.5.1.2 Test No.2: Nonwoven, needle-punched staple fibre polypropylene polyester geotextile (NW-N-SF-PP) – with single side thermally treated * vs. non plastic Berea sand*

The geotextile in test no 2 showed that excessive piping had occurred at the start of the test, which was indicative of a gradient ratio which was recorded as well below 1 (Table 5.1). Although this geotextile had smaller pore openings than the geotextile tested in test number 1, it was thinner and did not exhibit the thickness within which soil particles could lodge. Therefore, instead of the finer soil particles soil being lodged in the geotextile, they had been washed through it. This was also evident when comparing the number of constrictions of this geotextile to that of the geotextile used in test number 1.

Although the gradient ratio for this test was found to be below 1, which meant that there was a considerable amount of initial piping, it remained relatively constant for the remainder of the test, which suggested that not much further piping occurred. This was due to a natural reverse filter establishing itself on the adjacent upstream side of the geotextile. Although the gradient ratio indicated that no clogging was evident, piping was evident, which could also potentially be problematic. It should therefore be established by the design engineer whether any piping of the soil through the geotextile is to be permitted for the design criteria, as silting up of the drainage medium and pipe is possible, which could cause failure. It had already been established in the previous chapter that the soil to geotextile permeability was adequate. In conclusion, the geotextile to soil compatibility for this test was found to be satisfactory.

#### *5.5.1.3 Test No.3: Nonwoven, needle-punched staple fibre polypropylene polyester geotextile (NW-HB-CF-PP) vs. non plastic Berea sand*

The nonwoven needle-punched heat bonded continuous filament geotextile (NW-HB-CF-PP) exhibited the highest gradient ratio for this test set (Table 5.1). This geotextile was also found to have the lowest number of constrictions, which did not assist the efficiency of its filtration characteristics in terms of encouraging the establishment of natural filter adjacently upstream of the geotextile. Furthermore, the geotextile also exhibited a lack of thickness as required by the criteria proposed by Giroud (1996), given in Chapter 2. During manufacture, this type of geotextile is heat bonded which melts the filaments and reduces the pore opening sizes considerably. This resulted in this geotextile being relatively thin compared to the other geotextiles tested. The thickness of the NW-HB-CF-PP geotextile was, on average, 0.3 mm. This test also took the longest to stabilise of any in this test series. The gradient ratio was on the upper limit of acceptability and, as established in the previous chapter, the permeability of the soil to geotextile interface was lower than that of the soil. This rendered the geotextile unsuitable to be used as a filter with this soil.

#### *5.5.1.4 Test No.4: Woven, slit film polypropylene geotextile (W-SLF-PP) vs. non plastic Berea sand*

The gradient ratio for this test showed strong signs of blocking (Table 5.1). Blocking is referred to when using woven geotextiles. This has been discussed in Chapter 2. The slit film woven geotextile did not exhibit functional thickness when compared to the mechanically bonded, 3 dimensional, fibrous nonwoven geotextiles. These flat slit film woven geotextiles act as two dimensional filters, rather than as three dimensional filters as do nonwoven geotextiles. From the test results it can be deduced that piping of the finer soil particles had occurred through the openings of the geotextile after approximately 96 hours into the test. This test also took the second longest of this test series to stabilise. The gradient ratio stabilised at 2.436, which is close to the upper limit. The gradient ratio still suggested blocking, but at an acceptable limit. Therefore this geotextile could to be used as a filter with the soil tested.

#### *5.5.1.5 Summary of tests numbers 1-4*

The gradient ratios obtained from tests 1 to 4 provided the following conclusions about the soil to geotextile compatibility:

1. The NW-N-CF-PET geotextile performed the best of all geotextiles tested with this soil type. This test took the shortest time to stabilise. The soil to geotextile performance was acceptable in terms of both gradient ratio and permeability.
2. The NW-N-SF-PP geotextile showed signs of excessive piping. This test was the second quickest to stabilise. The soil to geotextile performance was adequate in terms of both gradient ratio and permeability. The author suspected that concentrated water throughflow was experienced at the soil to geotextile interface, which resulted in a high amount of piping and low gradient ratio values.
3. The NW-HB-CF-PP geotextile started off with high gradient ratios and showed recovery to the upper limit of around 3. This test took the longest to stabilise. This geotextile posed a higher risk of failure due to clogging than the other geotextiles tested in this series. The soil to geotextile performance was not good enough, as high levels of clogging were apparent, as well as low soil to geotextile permeability.

4. The W-SLF-PP also exhibited signs of a high risk of failure due to blocking. This test took third longest to stabilise.

### 5.5.2 Tests numbers 5 – 8

#### *5.5.2.1 Test No.5: Nonwoven, needle-punched continuous filament polyester geotextile (NW-N-CF-PET) vs. Berea sand (PI = 7)*

This test exhibited a relatively high permeability. As observed in the previous chapter (Figure 4.15), the soil to geotextile permeability only showed recovery close to the end of the test. It is uncertain whether the permeability across the soil to geotextile would have further reduced in the future. The test should have been run for a longer period to determine what the long term soil to geotextile permeability would have been. For the purpose of this test experiment, the result of the soil to geotextile permeability was considered to be acceptable. The gradient ratio at the start of the test was 1.724 and reduced to 0.39 after approximately 96 hours. This was evidence that a high degree of piping had occurred. Subsequent to this, it was anticipated that a reverse natural filter started forming adjacently upstream of the geotextile to raise the gradient ratio up to approximately 1.200. The gradient ratio remained relatively uniform up to approximately 1 200 hours. After this, a second stage of piping of fine material through the geotextile occurred, which was in line with the increased permeability across standpipes 6 and 7 at this time. The system's gradient ratio stabilised at 0.542, which was evidence that the geotextile was compatible as a filter with this soil (Table 5.7).

**Table 5.7: Summary of gradient ratios vs. time: test nos. 5-8**

| Time (Hours) | Test No 5 Gradient ratios | Test No 6 Gradient ratios | Test No 7 Gradient ratios | Test No 8 Gradient ratios |
|---|---|---|---|---|
| 1 | 1.724 | 2.142 | 5.919 | 2.985 |
| 24 | 1.517 | 1.864 | 4.750 | 1.126 |
| 48 | 1.084 | 1.709 | 4.705 | 1.171 |
| 72 | 0.719 | 1.062 | 3.420 | 0.898 |
| 96 | 0.390 | 0.913 | 3.101 | 0.887 |
| 168 | 1.207 | 1.252 | 3.617 | 0.863 |
| 192 | 1.253 | 1.298 | 3.617 | 0.898 |
| 240 | 1.190 | 1.252 | 3.617 | 0.863 |
| 264 | 1.084 | 1.239 | 3.617 | 0.697 |
| 336 | 1.207 | 1.252 | 3.617 | 0.697 |
| 360 | 1.138 | 1.252 | 3.617 | 0.697 |
| 384 | 1.084 | 1.239 | 3.617 | 0.651 |
| 408 | 0.981 | 1.239 | 3.617 | 0.651 |
| 432 | 0.981 | 1.239 | 3.617 | 0.651 |
| 504 | 0.981 | 1.239 | 3.617 | 0.651 |
| 528 | 0.981 | 1.239 | 3.617 | 0.651 |
| 552 | 0.981 | 1.239 | 3.268 | 0.651 |
| 576 | 0.981 | 1.239 | 3.268 | 0.651 |
| 840 | 1.047 | 1.511 | 3.268 | 0.651 |
| 864 | 1.047 | 1.511 | 3.606 | 1.051 |
| 888 | 1.047 | 1.138 | 3.606 | 1.051 |
| 912 | 1.047 | 1.138 | 3.083 | 1.051 |
| 936 | 1.047 | 1.138 | 3.083 | 1.051 |
| 1008 | 1.047 | 1.138 | 3.083 | 1.051 |
| 1032 | 0.948 | 1.298 | 3.083 | 0.938 |
| 1056 | 0.948 | 1.298 | 3.083 | 0.938 |
| 1080 | 0.948 | 1.298 | 2.650 | 0.938 |
| 1104 | 0.948 | 1.298 | 2.650 | 0.938 |
| 1176 | 0.948 | 1.298 | 2.650 | 0.938 |
| 1200 | 0.948 | 1.298 | 2.650 | 0.938 |
| 1224 | 0.542 | 1.193 | 2.650 | 0.938 |
| 1248 | 0.542 | 1.193 | 2.650 | 0.938 |
| 1272 | 0.542 | 1.193 | 2.650 | 0.938 |

| | | | | |
|---|---|---|---|---|
| 1368 | | | 2.650 | 0.938 |
| 1392 | | | 2.810 | 0.938 |
| 1416 | | | 2.810 | 0.938 |
| 1512 | | | 2.537 | 0.751 |
| 1526 | | | 2.537 | 0.751 |
| 1560 | | | 2.537 | 0.751 |
| 1584 | | | 2.537 | 0.751 |
| 1628 | | | 2.537 | 0.751 |
| 1680 | | | 2.537 | 0.751 |

#### *5.5.2.2 Test No.6: Nonwoven, needle-punched staple fibre polypropylene polyester geotextile (NW-N-SF-PP) – with single side thermally treated * vs. Berea sand (PI=7)*

The gradient ratios obtained from this test are all below 3 (Table 5.7). There was initial piping at the start of the test. After approximately 198 hours into the test, the gradient ratio showed some recovery as it was anticipated that the natural reverse filter had started forming on the upstream side of the geotextile. However, the soil to geotextile permeability was found to be lower than that of the soil and this was problematic (Figure 4.18). This could be a function of the manufacturing technique, whereby one side of the geotextile is thermally treated. Quite often geotextiles undergo thermal treatments to increase their tensile strength and doing so compromises their filtration attributes. Therefore this geotextile did not perform satisfactorily as a filter with this soil.

#### *5.5.2.3 Test No.7: Nonwoven, needle-punched staple fibre polypropylene polyester geotextile (NW-HB-CF-PP) vs. Berea sand (PI = 7)*

The gradient ratios obtained for this test showed that from the onset the geotextile experienced a high degree of clogging (Table 5.7). The gradient ratio recovered to a reading of 2.537 at the end of the test. In terms of the requirements of ASTM D5101, the gradient ratio was acceptable. However, the soil to geotextile permeability was much lower than that of the soil and therefore not acceptable in terms of the permeability requirements. This geotextile underwent a heat bonding process during manufacture. This had a significant influence on its thickness, as it was reduced. The pore size of the geotextile was also reduced, as the fibres were melted after the needle punching process. This is a probable reason for this geotextile failing in terms of the required permeability criteria. In conclusion, this geotextile was not suitable to be used as a filter with this soil.

#### *5.5.2.4 Test No.8: Woven, slit film polypropylene geotextile (W-SLF-PP) vs. Berea sand (PI = 7)*

From this test it was evident that a considerable amount of piping of the fine soil particles had occurred through openings of the geotextile (Table 5.7). This geotextile had the biggest opening size of all the geotextiles tested. The amount of piping could be problematic, as the finer soil material could potentially block the drain system to the point where it became dysfunctional. The soil to geotextile permeability, as shown in Table 5.4, was higher than that of the soil. It was concluded that the geotextile was acceptable to be used as a filter with the tested soil, although caution should be exercised with regard to potential piping.

#### *5.5.2.5 Summary of tests numbers 5-8*

The gradient ratios and soil to geotextile permeabilities obtained from tests 5 to 8 provided the following conclusions about the soil to geotextile compatibility:

1. The NW-N-CF-PET geotextile again performed most optimally of all the geotextiles tested with this soil type. This test had one of the shortest times to stabilise. The soil to geotextile performance was acceptable in terms of both gradient ratio and permeability (test 5).
2. The NW-N-SF-PP geotextile never showed signs of clogging and ended with an acceptable gradient ratio which showed no signs of clogging. However, the soil to geotextile interface had a permeability rating less than that of the soil. Therefore this geotextile was not suited to be used as a filter with this soil. (test 6).
3. Similarly to test number 7, the test on the NW-HB-CF-PP geotextile resulted in a gradient ratio which suggested that, although there was a tendency to clogging, it was acceptable in terms of its gradient ratio. The soil to geotextile permeability, however, was found to be much lower than that of the soil and therefore rendered this geotextile unacceptable for use as a filter with the soil tested (test 7).
4. The W-SLF-PP was not observed to have any sign of blocking. The soil to geotextile permeability was also found to be higher than that of the soil, which made this geotextile suitable as a filter for this soil (test 8).

### 5.5.3 Test numbers 9-12

#### *5.5.3.1 Test No.9: Nonwoven, needle-punched continuous filament polyester geotextile (NW-N-CF-PET) vs. non plastic Berea sand (2)*

From this test result it was evident that the gradient ratios obtained for this test were relatively high (Table 5.8). This meant that the geotextile had experienced a high degree of clogging. The finer fraction of the soil was trapped in the matrix of the fibre of the geotextile. The geotextile showed signs of clogging very early in the test and showed no signs of recovery until 624 hours into the test. The corresponding permeability, as is illustrated in Table 5.5, also proved that the permeability of the soil to geotextile interface was way below the soil's permeability. This was due the fact that, according to the soil grading curve illustrated in Chapter 3, it was a semi-gap graded soil. Together with the soil being non plastic, it had been anticipated that the soil itself would not have enough bigger particles to hold back the fine particles. This resulted in difficulty being experienced in forming a natural filter adjacent to the geotextile, and the finer particles then clogged up the pores of the geotextile. This was also unacceptable in terms of the soil to geotextile permeability criterion. It was therefore concluded that the geotextile was not acceptable as a filter with this soil.

#### *5.5.3.2 Test No.10: Nonwoven, needle-punched staple fibre polypropylene polyester geotextile (NW-N-SF-PP) – with single side thermally treated * vs. non plastic Berea sand (2)*

The gradient ratios observed for this test were just above 3, which was evidence that the geotextile had experienced clogging (Table 5.8). The gradient ratio showed some recovery to below 3 during the middle part of the test, but eventually ended up at 3.604. The soil to geotextile permeability test results, as in Table 5.5, showed that it was much lower than that of the soil. Therefore the geotextile was unfit to be used as a filter with the soil.

**Table 5.8: Summary of gradient ratios vs. time: test nos. 9-12**

| Time (Hours) | Test No 9 Gradient ratios | Test No 10 Gradient ratios | Test No 11 Gradient ratios | Test No 12 Gradient ratios |
|---|---|---|---|---|
| 1 | 7.105 | 3.568 | 3.953 | 4.485 |
| 24 | 7.105 | 3.568 | 3.953 | 4.998 |
| 48 | 7.105 | 3.568 | 3.953 | 4.998 |
| 120 | 9.473 | 2.919 | 2.223 | 4.614 |
| 144 | 9.473 | 2.919 | 2.223 | 4.614 |
| 168 | 9.473 | 2.919 | 2.223 | 4.614 |
| 192 | 9.473 | 2.919 | 2.223 | 4.614 |
| 216 | 9.473 | 2.919 | 2.541 | 3.524 |
| 288 | 15.16 | 2.919 | 3.294 | 4.614 |
| 312 | 15.16 | 2.919 | 3.294 | 4.614 |
| 336 | 15.16 | 2.919 | 3.294 | 4.614 |
| 360 | 15.16 | 2.654 | 3.294 | 4.614 |
| 384 | 20.13 | 3.244 | 3.623 | 4.998 |
| 456 | 20.13 | 3.244 | 3.623 | 4.998 |
| 480 | 20.13 | 3.244 | 3.623 | 9.612 |
| 504 | 88.1 | 3.604 | 4.743 | 9.612 |
| 528 | 88.1 | 3.604 | 4.743 | 9.612 |
| 600 | 58.1 | | 5.534 | 10.25 |
| 624 | 34.1 | | 5.534 | 10.25 |
| 648 | 34.1 | | 5.534 | 10.25 |
| 672 | | | | 10.25 |

#### *5.5.3.3 Test No. 11: Nonwoven, needle-punched staple fibre polypropylene polyester geotextile (NW-HB-CF-PP) vs. non plastic Berea sand (2)*

As in tests 1 and 2, this test produced gradient ratios above 3, which were indicative that the geotextile had experienced a degree of clogging (Table 5.8). In terms of the geotextile to soil permeability test results in Table 5.5, it can be seen that this is lower than that of the soil. From the test outcomes it was concluded that the geotextile was not compatible as a filter with the soil.

#### *5.5.3.4 Test No. 12: Woven, slit film polypropylene geotextile (W-SLF-PP) vs. non plastic Berea sand (2)*

The gradient ratios observed for this test showed that an excessive degree of blocking of the geotextile had occurred from the start of the test (Table 5.8). The gradient ratios never showed recovery and ended at an unacceptable gradient ratio of 10.25. The soil to geotextile permeability was also much lower than that of the soil. It is concluded that the geotextile was unacceptable to be used as a filter with this soil.

#### *5.5.3.5 Summary of test numbers 9-12*

All these geotextiles showed signs of high gradient ratios, which are indicative of excessive clogging. This was also proven by the results of the soil to geotextile permeabilities for all the tests. Judging from the grading curve of soil sample 3, compared to that of sample 2, one would have expected the soils to behave similarly when in contact with the same geotextiles. However, soil sample 2 was found to have some cohesion, which assisted with holding back the finer particles within the soil so that it did not clog the geotextile or pipe. It can be seen that a slight variation in the soil parameters had a huge effect on its compatibility with the geotextiles. The challenge of Berea sands is their high degree of variability over a small area.

## 5.6 EVALUATION OF GEOTEXTILE FILTER DESIGN CRITERIA

In order to evaluate the available filter design criteria, a comparative analysis was carried out using the results of the laboratory tests. This was a way to validate the laboratory results, provided that there was correlation between the two sets of results.

### 5.6.1 SABS 1200

SABS 1200 defines the geotextiles in grades from 1-10, Grade 1 being the lowest in mass (100 g/m2) and grade 10 (300 g/m2) being the highest. It is the author’s opinion that most specifications that are based on the SABS specifications would call for filtration grade geotextile of grades between 1 and 6. All the geotextiles tested in this study would meet the criteria set out by SABS 1200. Thus, if an engineer were to use only the SABS specifications to justify its use in the Berea sands tested in this study, the probability of failure of a geotextile would be high. It is therefore suggested that the SABS specifications not be considered in isolation, as they have proved to be inadequate.

### 5.6.2 COLTO: 1988

COLTO (1988) tabulates drainage and filtration grade geotextiles in terms of penetration load, puncture resistance and water percolation. Similar to the case of the SABS, it will be risky to use the COLTO required specifications in isolation. Most of the geotextiles presented in this thesis would meet the criteria of a grade 3 geotextile, according to COLTO. If an engineer were to use grade 3 geotextile as his criteria in the Berea sand context, one can easily see that the recipe is set up for probable failure.

### 5.6.3 AASHTO M288

An analysis was performed according to AASHTO M288, in order to find what the geotextile criteria should be with the three Berea sands. This was done with reference to Tables 2.10 and 2.11 in Chapter 2 of this study and as outlined below. This method has already been discussed in Chapter 2. The tables are included again below for easy reference.

**Table 5.9: Geotextile strength requirements (AASHTO M288)**

| | | | Geotextile Class | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Class 1 | | Class 2 | | Class 3 | |
| | | | Elongation | | Elongation | | Elongation | |
| | **Test Methods** | **Units** | **<50%** | **≥50%** | **<50%** | **≥50%** | **<50%** | **≥50%** |
| Grab Strength | ASTM D 4632 | N | 1400 | 900 | 1100 | 700 | 800 | 500 |
| Sewn Seam Strength | ASTM D 4632 | N | 1260 | 810 | 990 | 630 | 720 | 450 |
| Tear Strength | ASTM D 4533 | N | 500 | 350 | 400 | 250 | 300 | 180 |
| Puncture Strength | ASTM D 4833 | N | 500 | 350 | 400 | 250 | 300 | 180 |
| Burst Strength | ASTM D 3786 | kPa | 3500 | 1700 | 2700 | 1300 | 2100 | 950 |

**Table 5.10: Subsurface drainage geotextile requirement (AASHTO M288)**

| | Test Methods | Units | Requirements<br>Percent silt and clay (<0.075 mm)<br><15 | <br><br>15 to 50 | <br><br>>50 |
|---|---|---|---|---|---|
| Geotextile Class | | | | Class 2 | |
| Permittivity | ASTM D 4491 | $sec^{-1}$ | 0.5 | 0.2 | 0.1 |
| Apparent Opening Size | ASTM D 4751 | mm | 0.43<br>max. avg. roll value | 0.25<br>max. avg. roll value | 0.22<br>max. avg. roll value |
| Ultraviolet Stability (Retained Strength) | ASTM D 4355 | % | 50% after 500 hrs of exposure | | |

### *5.6.3.1 Soil sample 1 – Berea sand non-plastic*

The percentage of particles of a size below 0.075mm < 15% (Table 5.10). For soil sample 1, the required geotextile specifications according to AASHTO M288 are defined in Table 5.11.

**Table 5.11: Soil sample 1: AASHTO geotextile specification requirements vs. tested geotextiles**

| | | | **AASHTO M288 Required Specification** | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Geotextile Property** | **Unit** | **Test Method** | **<50% Elongation** | **>50% elongation** | **NW-N-CF-PET** | **NW-N-SF-PP** | **NW-HB-CF-PP** | **W-SLF-PP** |
| **Grab Strength** | **N** | **ASTM D 4632** | *1100* | *700* | 500 | N/A | 625 | 525 |
| **Tear strength** | **N** | **ASTM D 4533** | *400* | *350* | 240 | N/A | 290 | 300 |
| **Puncture Strength** | **N** | **ASTM D4833** | *400* | *350* | 1300 | 1700 | 1100 | 2600 |
| **Burst Strength** | **kPa** | **ASTM 3786** | *2700* | *1700* | 1500 | N/A | N/A | N/A |
| **Permittivity** | **$Sec^{-1}$** | **ASTM D4491** | *0.5* | *0.5* | 2.7 | 1.4 | 0.6 | N/A |
| **Apparent Opening size** | **mm** | **ASTM D4751** | *0.43* | *0.43* | 0.205 | 0.085 | 0.140 | N/A |
| **Ultraviolet Stability** | **%** | **ASTM D 4355** | *50% after 500 hrs exposure* | *50% after 500 hrs exposure* | | | | N/A |

It is evident from Table 5.11 that woven geotextiles are not recommended as filters (Table 5.11). It can also be established that all the geotextiles tested had smaller openings than those suggested by AASHTO M288. The strength characteristics were not as important for this study, besides their consideration regarding possible installation damage.

#### *5.6.3.2 Soil samples 2 and 3 – Berea sand (PI = 7) and Berea sand non plastic (2)*

**Table 5.12: Soil samples 2 and 3: AASHTO geotextile specification requirements vs. tested geotextiles**

| | | | AASHTO M288 Required Specification | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Geotextile Property** | **Unit** | **Test Method** | **<50% Elongation** | **>50% Elongation** | **NW-N-CF-PET** | **NW-N-SF-PP** | **NW-HB-CF-PP** | **W-SLF-PP** |
| **Grab Strength** | **N** | **ASTM D 4632** | *1100* | *700* | 500 | N/A | 625 | 525 |
| **Tear strength** | **N** | **ASTM D 4533** | *400* | *350* | 240 | N/A | 290 | 300 |
| **Puncture Strength** | **N** | **ASTM D4833** | *400* | *350* | 1300 | 1700 | 1100 | 2600 |
| **Burst Strength** | **kPa** | **ASTM 3786** | *2700* | *1700* | 1500 | N/A | N/A | N/A |
| **Permittivity** | **$Sec^{-1}$** | **ASTM D4491** | *0.200* | *0.200* | 2.7 | 1.4 | 0.6 | N/A |
| **Apparent Opening size** | **mm** | **ASTM D4751** | *0.250* | *0.250* | 0.205 | 0.085 | 0.140 | N/A |
| **Ultraviolet Stability** | **%** | **ASTM D 4355** | *50% after 500 hrs exposure* | *50% after 500 hrs exposure* | | | | N/A |

When comparing the AASHTO requirements in terms of geotextile opening sizes, it was evident that all the geotextiles' pore opening sizes were smaller than the recommended pore opening sizes (Table 5.12). It is uncertain whether using geotextiles with larger pore opening sizes would be compatible with the Berea sand. The theory of the number of constrictions ($N_s$) is also ignored by AASHTO M288. Whilst the table is a guideline, it should be established whether the application of the geotextile filter is critical or non-critical. Critical applications would imply that the consequence of failure of the geotextile performance could lead to a loss of human life; for example, a dam embankment failure due to the failed performance of a geotextile. If the application of the geotextile filter is for a critical application, AASHTO suggest that long term filtration tests, such as the long term gradient ratio testing according to ASTM D5101, be carried out.

### 5.6.4 Dutch practice

The geotextile filter criteria based on the $d_{90}$ of the soil is as follows:

$O_{95} < d_{90}$ for wovens,

$O_{90} < 1.8d_{90}$ for nonwovens

**Soil sample 1:** $d_{90} = 0.799$ mm

For woven geotextile $0_{95} < 0.799$ mm

For a nonwoven geotextile $0_{90} < 1.43$ mm

A summary of the actual geotextile pore opening size and the recommended geotextile pore opening size for soil samples 1, 2 and 3 according to Dutch practice is illustrated in Tables 5.13-5.15 below.

**Table 5.13: Geotextile pore opening vs. Dutch criteria for soil sample 1**

| Geotextile | Pore opening size (mm) | Recommended pore opening size (mm) | Geotextile compatibility |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | <1.430 | Adequate |
| **NW-N-SF-PP** | 0.085 | <1.430 | Adequate |
| **NW-HB-CF-PP** | 0.140 | <1.430 | Unacceptable |
| **W-SLF-PP** | 0.670 | <0.799 | Adequate |

**Soil sample 2:** **$d_{90}$ = 0.372 mm**

For a woven geotextile: $0_{95} < 0.372$ mm

For nonwoven geotextile: $0_{90} < 0.670$ mm

**Table 5.14: Geotextile pore opening vs. Dutch criteria for soil sample 2**

| Geotextile | Pore opening size (mm) | Recommended pore opening size (mm) | Geotextile compatibility |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | <0.670 | Adequate |
| **NW-N-SF-PP** | 0.085 | <0.670 | Unacceptable |
| **NW-HB-CF-PP** | 0.140 | <0.670 | Unacceptable |
| **W-SLF-PP** | 0.670 | <0.372 | Adequate |

**For soil sample 3:** **$d_{90}$ = 0.392 mm**

For a woven geotextile: $0_{95}$ < 0.392 mm

For a nonwoven geotextile: $0_{90}$ < 0.710 mm

**Table 5.15: Geotextile pore opening vs. Dutch criteria for soil sample 3**

| **Geotextile** | **Pore opening size (mm)** | **Recommended pore opening size (mm)** | **Geotextile compatibility** |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | <0.710 | Unacceptable |
| **NW-N-SF-PP** | 0.085 | <0.710 | Unacceptable |
| **NW-HB-CF-PP** | 0.140 | <0.710 | Unacceptable |
| **W-SLF-PP** | 0.670 | <0.392 | Unacceptable |

**Table 5.16: Summary of Dutch filter criteria and geotextile performance**

| **Test No** | **Soil Sample** | **Gtx Pore opening criteria met** | **Overall Gtx performance in laboratory** |
|---|---|---|---|
| 1 | 1 | Yes | Pass |
| 2 | 1 | Yes | Pass |
| 3 | 1 | Yes | Fail |
| 4 | 1 | Yes | Pass |
| 5 | 2 | Yes | Pass |
| 6 | 2 | Yes | Fail |
| 7 | 2 | Yes | Fail |
| 8 | 2 | No | Pass |
| 9 | 3 | Yes | Fail |
| 10 | 3 | Yes | Fail |
| 11 | 3 | Yes | Fail |
| 12 | 3 | No | Fail |

It can be seen from Table 5.16 that although the geotextiles in most cases met the pore opening criteria of the Dutch method, there were still a fair number of soil to geotextile failures. So there was a poor correlation between the geotextile opening size criteria versus the soil to geotextile systems that had actually passed. From the results t of a total of twelve tests, only five of the recommended criteria resulted in adequate soil to geotextile performance. Using the Dutch practice with the Berea sand would not have been ideal.

### 5.6.5 German practice

**Soil Sample 1:** $d_{40}$ = 0.208 mm > 0.06 mm (stable problematic soil)

$d_{10}$ = 0.040 mm

$d_{90}$ = 0.799 mm

Cu = 8

**Soil sample 2:** $d_{40}$ = 0.161 mm > 0.06 mm (stable problematic soil)

$d_{10}$ = 0.002 mm

$d_{90}$ = 0.372 mm

Cu = 95.2

**Soil sample 3:** $d_{40}$ = 0.157 mm > 0.06mm (stable problematic soil)

$d_{10}$ = 0.002 mm

$d_{90}$ = 0.392 mm

Cu = 111.6

Therefore recommended geotextile opening size is:

$$\mathbf{O_{95} < 5d_{10}.Cu^{0.5} \text{ and } O_{95} < d_{90}}$$

**Soil sample 1:** **$O_{95}$ < 0.566 mm and $O_{95}$ < 0.799 mm**

A summary of the actual geotextile pore opening size and the recommended geotextile pore opening size for soil samples 1, 2 and 3 according to German practice is illustrated in Tables 5.17- 5.19 below.

**Table 5.17: Geotextile pore opening vs. German criteria for soil sample 1**

| Geotextile | Pore opening size (mm) | Recommended pore opening size (mm) | Geotextile compatibility |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | <0.566 and <0.799 | Adequate |
| **NW-N-SF-PP** | 0.085 | <0.566 and <0.799 | Adequate |
| **NW-HB-CF-PP** | 0.140 | <0.566 and <0.799 | Unacceptable |
| **W-SLF-PP** | 0.670 | <0.566 and <0.799 | Adequate |

**Soil sample 2:** **$O_{95}$ < 0.097 mm and $O_{95}$ < 0.372 mm**

**Table 5.18: Geotextile pore opening vs. German criteria for soil sample 2**

| Geotextile | Pore opening size (mm) | Recommended pore opening size (mm) | Geotextile compatibility |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | <0.097 and <0.372 | Adequate |
| **NW-N-SF-PP** | 0.085 | <0.097 and <0.372 | Unacceptable |
| **NW-HB-CF-PP** | 0.140 | <0.097 and <0.372 | Unacceptable |
| **W-SLF-PP** | 0.670 | <0.097 and <0.372 | Adequate |

**Soil sample 3:** **$O_{95}$ < 0.106 mm and $O_{95}$ < 0.392 mm**

**Table 5.19: Geotextile pore opening vs. German criteria for soil sample 3**

| Geotextile | Pore opening size (mm) | Recommended pore opening size (mm) | Geotextile compatibility |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | <0.106 and <0.392 | Unacceptable |
| **NW-N-SF-PP** | 0.085 | <0.106 and <0.392 | Unacceptable |
| **NW-HB-CF-PP** | 0.140 | <0.106 and <0.392 | Unacceptable |
| **W-SLF-PP** | 0.670 | <0.106 and <0.392 | Unacceptable |

**Table 5.20: Summary of German filter criteria and geotextile performance**

| Test no | Soil Sample no | Gtx Pore opening criteria met | Overall Gtx performance in laboratory |
|---|---|---|---|
| 1 | 1 | Yes | Pass |
| 2 | 1 | Yes | Pass |
| 3 | 1 | Yes | Fail |
| 4 | 1 | No | Pass |
| 5 | 2 | No | Pass |
| 6 | 2 | Yes | Fail |
| 7 | 2 | No | Fail |
| 8 | 2 | No | Pass |
| 9 | 3 | No | Fail |
| 10 | 3 | Yes | Fail |
| 11 | 3 | No | Fail |
| 12 | 3 | No | Fail |

As can be seen in Table 5.20, there is a poor correlation between the geotextile pore opening criteria versus the soil to geotextile system performance. The German practice also does not really seem to work well for the Berea sands tested.

### 5.6.6 American practice

Soil sample 1: $d_{50}$ = 0.261 mm > 0.075 mm

$d_{85}$ = 0.612 mm

Soil sample 2: $d_{50}$ = 0.184 mm > 0.075 mm

$d_{85}$ = 0.343 mm

Soil Sample 3: $d_{50}$ = 0.188 mm > 0.075 mm

$d_{85}$ = 0.364 mm

Geotextile filter criteria for nonwovens: $\mathbf{0.297\ mm \leq O_{95} \leq 1.8d_{85}}$

Geotextile filter criteria for woven geotextiles: $\mathbf{0.297\ mm \leq O_{95} \leq d_{85}}$

**Soil Sample 1:**

Nonwoven geotextiles: $0.297mm \leq O_{95} \leq 1.102$ mm

Woven geotextiles: $0.297mm \leq O_{95} \leq 0.612$ mm

A summary of the actual geotextile pore opening size and the recommended geotextile pore opening size for soil samples 1, 2 and 3 according to American practice is illustrated in Tables 5.21- 5.23 below.

**Table 5.21: Geotextile pore opening vs. American criteria for soil sample 1**

| **Geotextile** | **Pore opening size (mm)** | **Recommended pore opening size (mm)** | **Geotextile compatibility** |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | >0.297 and <1.102 | Adequate |
| **NW-N-SF-PP** | 0.085 | >0.297 and <1.102 | Adequate |
| **NW-HB-CF-PP** | 0.140 | >0.297 and <1.102 | Unacceptable |
| **W-SLF-PP** | 0.670 | >0.297 and <0.612 | Adequate |

**Soil Sample 2**

Nonwoven geotextiles: $0.297 mm \leq O_{95} \leq 0.617$ mm

Woven geotextiles: $0.297 mm \leq O_{95} \leq 0.343$ mm

**Table 5.22: Geotextile pore opening vs. American criteria for soil sample 2**

| **Geotextile** | **Pore opening size (mm)** | **Recommended pore opening size (mm)** | **Geotextile compatibility** |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | >0.297 and <1.102 | Adequate |
| **NW-N-SF-PP** | 0.085 | >0.297 and <1.102 | Unacceptable |
| **NW-HB-CF-PP** | 0.140 | >0.297 and <1.102 | Unacceptable |
| **W-SLF-PP** | 0.670 | >0.297 and <0.343 | Adequate |

**Soil Sample 3**

Nonwoven geotextiles: $0.297mm \leq O_{95} \leq 0.655$ mm

Woven geotextiles: $0.297mm \leq O_{95} \leq 0.364$ mm

**Table 5.23: Geotextile pore opening vs. American criteria for soil sample 3**

| Geotextile | Pore opening size (mm) | Recommended pore opening size (mm) | Geotextile compatibility |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | >0.297 and <1.102 | Adequate |
| **NW-N-SF-PP** | 0.085 | >0.297 and <1.102 | Adequate |
| **NW-HB-CF-PP** | 0.140 | >0.297 and <1.102 | Unacceptable |
| **W-SLF-PP** | 0.670 | >0.297 and <0.364 | Adequate |

**Table 5.24: Summary of American filter criteria and geotextile performance**

| Test no | Soil Sample no | Gtx Pore opening criteria met | Overall Gtx performance in laboratory |
|---|---|---|---|
| 1 | 1 | No | Pass |
| 2 | 1 | No | Pass |
| 3 | 1 | No | Fail |
| 4 | 1 | No | Pass |
| 5 | 2 | No | Pass |
| 6 | 2 | No | Fail |
| 7 | 2 | No | Fail |
| 8 | 2 | No | Pass |
| 9 | 3 | No | Fail |
| 10 | 3 | No | Fail |
| 11 | 3 | No | Fail |
| 12 | 3 | No | Fail |

The results observed from Table 5.24 showed a stronger correlation between the failed systems and those where the required geotextile pore opening size was not met. However, there still remains a fair degree of uncertainty and room for error. The American practice would not be ideal to follow for the Berea sand tested.

### 5.6.7 French practice

Soil sample 1: Cu = 8 > 4

$d_{15}$ = 0.085 mm

$d_{85}$ = 0.612 mm

Soil Sample 2: Cu = 95.2 > 4

$d_{15}$ = 0.017 mm

$d_{85}$ = 0.343 mm

Soil Sample 3: Cu = 111.6 >4

$d_{15}$ = 0.011 mm

$d_{85}$ = 0.364 mm

The soils are all considered as loose and therefore the geotextile characteristic pore opening size is recommended to be as follows:

$$\mathbf{4d_{15} \leq O_{95} \leq d_{85}}$$

**Soil sample 1:** **0.340 mm ≤ O95 ≤ 0.612 mm**

A summary of the actual geotextile pore opening size and the recommended geotextile pore opening size for soil samples 1, 2 and 3 according to French practice is illustrated in Tables 5.25- 5.27.

**Table 5.25: Geotextile pore opening vs. French criteria for soil sample 1**

| Geotextile | Pore opening size (mm) | Recommended pore opening size (mm) | Geotextile compatibility |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | >0.340 and <0.612 | Adequate |
| **NW-N-SF-PP** | 0.085 | >0.340 and <0.612 | Adequate |
| **NW-HB-CF-PP** | 0.140 | >0.340 and <0.612 | Unacceptable |
| **W-SLF-PP** | 0.670 | >0.340 and <0.612 | Adequate |

**Soil Sample 2:** **0.068 mm ≤ O95 ≤ 0.343 mm**

**Table 5.26: Geotextile pore opening vs. French criteria for soil sample 2**

| **Geotextile** | **Pore opening size (mm)** | **Recommended pore opening size (mm)** | **Geotextile compatibility** |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | >0.068 and <0.343 | Adequate |
| **NW-N-SF-PP** | 0.085 | >0.068 and <0.343 | Unacceptable |
| **NW-HB-CF-PP** | 0.140 | >0.068 and <0.343 | Unacceptable |
| **W-SLF-PP** | 0.670 | >0.068 and <0.343 | Adequate |

**Soil Sample 3:** **0.044mm ≤ $O_{95}$ ≤ 0.364 mm**

**Table 5.27: Geotextile pore opening vs. French criteria for soil sample**

| **Geotextile** | **Pore opening size (mm)** | **Recommended pore opening size (mm)** | **Geotextile compatibility** |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | >0.044 and <0.364 | Unacceptable |
| **NW-N-SF-PP** | 0.085 | >0.044 and <0.364 | Unacceptable |
| **NW-HB-CF-PP** | 0.140 | >0.044 and <0.364 | Unacceptable |
| **W-SLF-PP** | 0.670 | >0.044 and <0.364 | Unacceptable |

3

**Table 5.26: Summary of French filter criteria and geotextile performance**

| Test no | Soil Sample no | Gtx Pore opening criteria met | Overall Gtx performance in laboratory |
|---|---|---|---|
| 1 | 1 | No | Pass |
| 2 | 1 | No | Pass |
| 3 | 1 | No | Fail |
| 4 | 1 | No | Pass |
| 5 | 2 | Yes | Pass |
| 6 | 2 | Yes | Fail |
| 7 | 2 | Yes | Fail |
| 8 | 2 | No | Pass |
| 9 | 3 | Yes | Fail |
| 10 | 3 | Yes | Fail |
| 11 | 3 | Yes | Fail |
| 12 | 3 | No | Fail |

Once again, it could be observed in Table 5.26 that there was a poor correlation between the geotextile pore opening criteria versus the overall soil to geotextile performance. The French criteria would therefore not be ideal to have adopted for the Berea sand.

### 5.6.8 Luettich *et al*. design chart

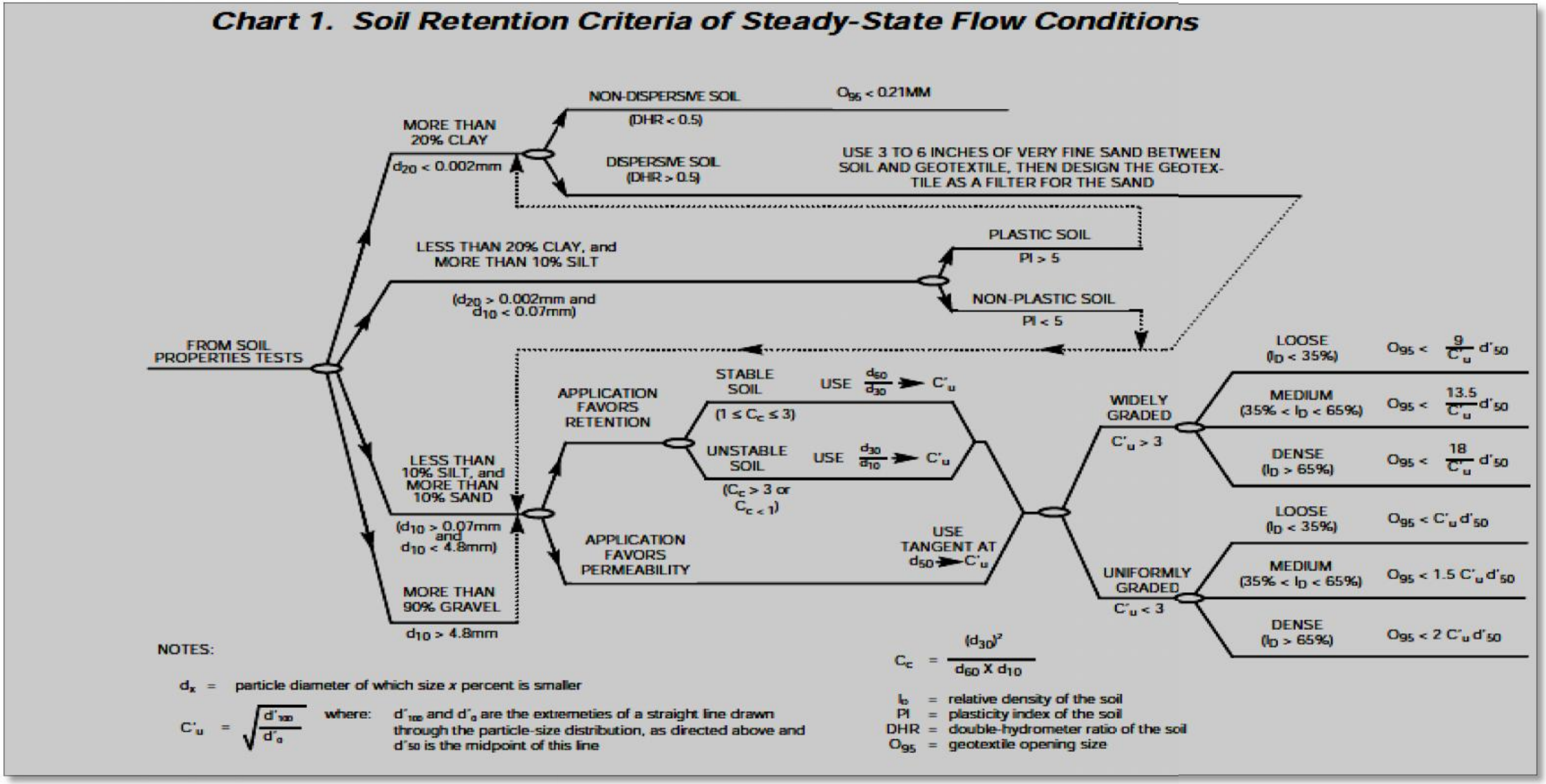



**Figure 5.1: Geotextile filter design chart** (after Luettich *et al*. 1992)

**Soil sample 1:**

$d_{10}$ = 0.040 mm

$d_{20}$ = 0.122 mm

$C'_u$ = $d_{60}/d_{10}$ = 1.837 < 3

**$0_{95}$ < $C'u.\ d'_{50}$ = 0.813 mm**

A summary of the actual geotextile pore opening size and the recommended geotextile pore opening size for soil samples 1, 2 and 3 according to the Leuttich *et al*. practice is illustrated in Tables 5.27-5.29 below.

**Table 5.27: Geotextile pore opening vs. Leuttich et al. criteria for soil sample 1**

| **Geotextile** | **Pore opening size (mm)** | **Recommended pore opening size (mm)** | **Geotextile compatibility** |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | 0.813 | Adequate |
| **NW-N-SF-PP** | 0.085 | 0.813 | Adequate |
| **NW-HB-CF-PP** | 0.140 | 0.813 | Unacceptable |
| **W-SLF-PP** | 0.670 | 0.813 | Adequate |

**Soil sample 2**

$d_{10}$ = 0.002 mm

PI = 7 > 5

Soil dispersivity is unknown, as it was not tested. If a soil were dispersive, then all geotextile filter opening sizes are recommended to be **< 0.210 mm.** If the soil were dispersive it is recommended that a 75 mm- 150 mm layer of clean filter sand be placed between the geotextile and the in-situ soil. The geotextile should then be designed for the clean filter sand layer. For the purpose of this study this would be the better option in this case, as it eliminates the uncertainties in variation and behaviour of the Berea sand.

**Table 5.28: Geotextile pore opening vs. Leuttich *et al*. criteria for soil sample 2**

| Geotextile | Pore opening size (mm) | Recommended pore opening size (mm) | Geotextile compatibility |
|---|---|---|---|
| NW-N-CF-PET | 0.205 | <0.210 | Adequate |
| NW-N-SF-PP | 0.085 | <0.210 | Unacceptable |
| NW-HB-CF-PP | 0.140 | <0.210 | Unacceptable |
| W-SLF-PP | 0.670 | <0.210 | Adequate |

**Soil sample 3**

$d_{10}$ = 0.002 mm

PI = Non plastic

$C_c$ = 11.95

$C'_u$ = $d_{30}/d_{10}$ = 4.300 > 3

$d'_{50}$ = 0.235 mm

**$0_{95}$ < $(9 / C'u).\ d'_{50}$ = 0.492 mm**

**Table 5.29: Geotextile pore opening vs. Leuttich *et al*. criteria for soil sample 3**

| Geotextile | Pore opening size (mm) | Recommended pore opening size (mm) | Geotextile compatibility |
|---|---|---|---|
| **NW-N-CF-PET** | 0.205 | <0.492 | Unacceptable |
| **NW-N-SF-PP** | 0.085 | <0.492 | Unacceptable |
| **NW-HB-CF-PP** | 0.140 | <0.492 | Unacceptable |
| **W-SLF-PP** | 0.670 | <0.492 | Unacceptable |

Once again, all the nonwoven geotextiles had a pore opening size of less than 0.492 mm. However, the test results still showed high degree of clogging for all of them (Table 5.29). The woven geotextile had an opening size greater than 0.492 yet also showed results indicating excessive blocking.

It can be concluded from the above results that the method proposed by Luettich *et al*. has not entirely satisfied the design geotextile filter requirements of the Berea sand.

## 5.8 COMPUTER PROGRAM

The computer program used in this study was based on an Excel spreadsheet produced by Kaytech. The flowchart is shown again below for easy reference.

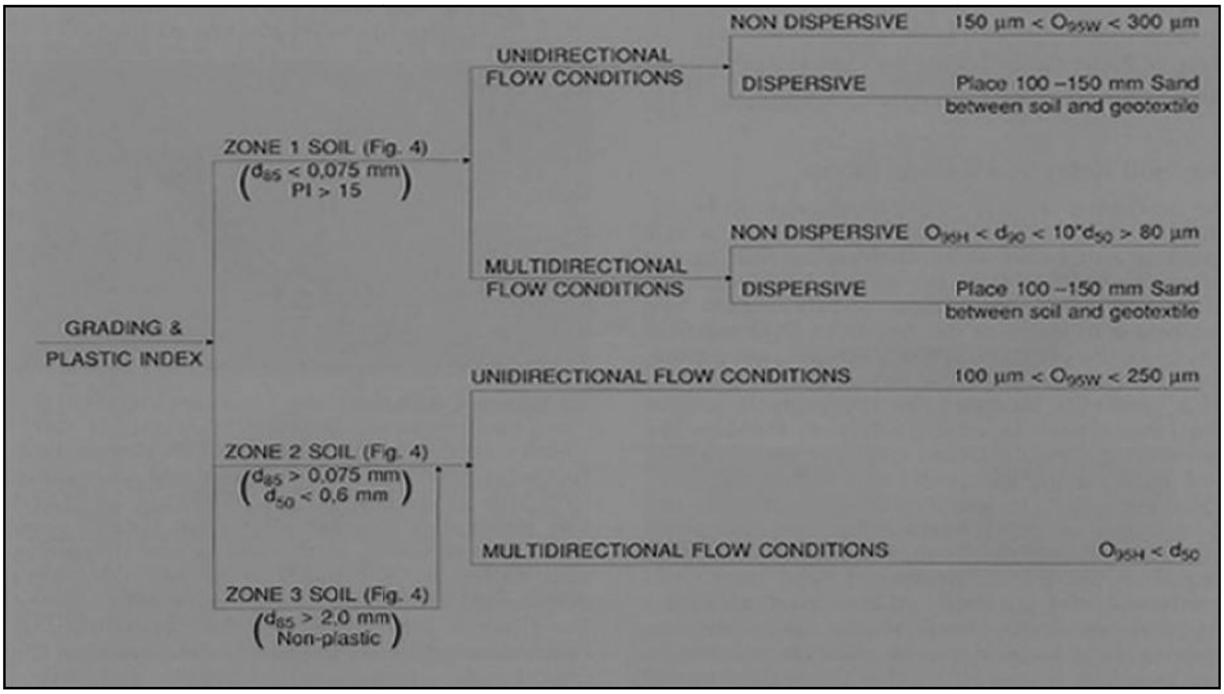



**Figure 5.2: Flowchart for computer spreadsheet (Kaytech)**

Based on the input, the spreadsheet produced the following geotextile criteria:

**100 µm<$O_{95}$>250 µm**

A summary of the actual geotextile pore opening sizes and those recommended by the computer spreadsheet for all soils is illustrated in Table 5.30 below.

**Table 5.30: Geotextile pore opening vs. computer spreadsheet output for all soils**

| Geotextile | Pore opening size (mm) | Recommended pore opening size (mm) |
|---|---|---|
| **NW-N-CF-PET** | 0.205 | >0.100 and <0.250 |
| **NW-N-SF-PP** | 0.085 | >0.100 and <0.250 |
| **NW-HB-CF-PP** | 0.140 | >0.100 and <0.250 |
| **W-SLF-PP** | 0.670 | >0.100 and <0.250 |

**Table 5.31: Summary of computer output filter criteria and geotextile performance**

| Test no | Soil Sample no | Gtx Pore opening criteria met | Overall Gtx performance in laboratory |
|---|---|---|---|
| 1 | 1 | Yes | Pass |
| 2 | 1 | No | Pass |
| 3 | 1 | Yes | Fail |
| 4 | 1 | No | Pass |
| 5 | 2 | Yes | Pass |
| 6 | 2 | No | Fail |
| 7 | 2 | Yes | Fail |
| 8 | 2 | No | Pass |
| 9 | 3 | Yes | Fail |
| 10 | 3 | No | Fail |
| 11 | 3 | Yes | Fail |
| 12 | 3 | No | Fail |

From the computer results it can be seen that there is a poor correlation between the recommended pore opening size and the actual filter performance (Table 5.30). From the computer results it is apparent that the computer spreadsheet cannot be applied in isolation to the Berea sands tested.

## 5.7 FINE SOIL FRACTION ANALYSIS

An analysis was done on the fine soil fraction that was lodged in the geotextile and the fine soil fraction passing through? the geotextiles. The amount of fine soil passing through the geotextiles was too small to enable the researcher to carry out further grading analysis. The results are illustrated in Table 5.31.

**Table 5.32: Fine soil fraction lodged within and passing geotextile**

| Test no. | Total mass of soil sample (g) | Gtx mass before test (g) | Gtx mass after test (g) | Mass of fine soil fraction lodged inside Gtx (g) | Mass of fine soil fraction passing Gtx (g) |
|---|---|---|---|---|---|
| **1** | 1200 | 1.46 | 5.52 | 4.06 | 2.08 |
| **2** | 1200 | 1.92 | 3.26 | 1.34 | 3.01 |
| **3** | 1200 | 1.46 | 2.34 | 0.88 | 0.67 |
| **4** | 1200 | 1.70 | 1.89 | 0.19 | 0.42 |
| **5** | 1200 | 2.10 | 4.80 | 2.70 | 1.07 |
| **6** | 1200 | 1.81 | 2.33 | 0.52 | 0.88 |
| **7** | 1200 | 1.49 | 1.76 | 0.27 | 1.18 |
| **8** | 1200 | 1.73 | 1.97 | 0.24 | 1.10 |
| **9** | 1200 | 1.90 | 11.40 | 9.50 | 1.56 |
| **10** | 1200 | 1.89 | 10.08 | 8.19 | 1.84 |
| **11** | 1200 | 1.56 | 4.40 | 2.84 | 0.70 |
| **12** | 1200 | 1.82 | 4.08 | 2.26 | 0.21 |

Tests 1- 4

From tests 1-4, it can be seen that most fines had piped through the geotextile in test no. 2. Although this geotextile had the smallest opening size of the tested geotextiles, the author is of the opinion that some localised flow and piping occurred at the soil to geotextile interface. The NW-N-CF-PET geotextile which was used in test 1 was the thickest and had the most soil particles lodged within it.

Tests 5- 9

The soil had some cohesion and it was evident by observation that a lesser amount of fine soil particles had piped through the geotextiles compared to tests 1- 4. This means that the soil, due to its cohesive nature, held back the finer soil particles within the soil matrix. There were also fewer soil particles lodged within the geotextile compared to those found during tests 1-4.

Tests 9- 12

During this set of tests, the geotextiles experienced the highest degree of clogging. This was apparent by the mass of fines lodged within the geotextile. The nonwovens, because of their thickness, had the greatest amount of fines trapped within them. The amount of soil particles that piped through the geotextiles was minimal.

## 5.8 DISCUSSION OF RESULTS

From the analysis of the laboratory test results and the evaluation of the available geotextile filter design criteria, much has been learnt and discovered about the interactive behaviour between Berea sand and some of the commercially available geotextiles in South Africa.

### 5.8.1 Geotextiles

The geotextiles used in the study were all considered as filtration grade geotextiles in the current industry. The geotextiles were made of different polymers as well as by different manufacturing techniques. It was evident that none of the geotextiles met the criteria for the number of constrictions according to Giroud. The only way this could have been improved on was to use much thicker geotextiles in the study. This could be one of the important reasons why there were unexpected failures in some cases.

### 5.8.2 Soil to geotextile interaction

Out of the 12 long term gradient ratio tests performed it was found that only 5 of the tests were acceptable; those where the geotextiles had not clogged and which also met the soil to geotextile permeability requirement. It was also found that the two geotextiles with the bigger pore opening sizes performed better overall than the two geotextiles with smaller sized pore openings. This is indicative that the Berea sand is unique in the sense that it has a very high degree of variability in terms of is engineering properties. It is also common to find a high degree of variability of the Berea sand when two investigative test pits are dug even only a few meters apart. The Berea sand proved to be highly problematic when the geotextiles were expected to perform as a filter in conjunction with the Berea sands in this study.

Piping of fine soil particles was observed in various magnitudes across the range of geotextiles. The amount of fines that piped through the geotextile was considered negligible as a percentage of the sample of soil used. Therefore, from this study, piping would not be considered as a cause of failure. It was observed that the thicker nonwoven geotextiles retained the fines within themselves due to their thickness. It is the author's opinion that some piping might also have occurred during the test setup, when the permeameters were manually tapped by hand.

### 5.8.2 Failure mechanisms

It was found from the study that the geotextiles either:

i) Clogged due to high gradient ratio
ii) Failed the soil to geotextile permeability criteria, but had an acceptable gradient ratio
iii) Suffered from a combination of the above inadequacies

From the results of the study it could be deduced that there were more cases of the soil to geotextile permeability falling short of the criteria than cases in which the geotextiles were clogged.

### 5.8.3 Soil properties

The three soil samples exhibited varying co-efficients of uniformity. It can be deduced that the highest coefficient of uniformity encountered, that of soil sample 3, together with its non-plastic nature, made it the most problematic of all the soils. It can also be deduced that the even when the gradings of soil samples 2 and 3 exhibited the same grading, they differed in PI, which yielded different results when in conjunction with the various geotextiles.

### 5.8.4 Regional filter criteria

When the regional filter criteria were compared to the laboratory results, there were poor correlations throughout. This proves that the Berea sand is a challenge to the design of geotextile filters, because of its high variability in engineering properties. It is suggested that a new set of filter criteria should be developed for Berea sands.

### 5.8.5 Computer results

From the computer results it was observed that there was a poor correlation between the suggested filter requirements and the outcomes of the laboratory study. The computer program is therefore not recommended for use in isolation as a predictive tool for geotextile filters to be used in conjunction with Berea sands.

# 6 Conclusions and Recommendations

## 6.1 INTRODUCTION

Through this study, a greater understanding has been achieved of how common commercially available geotextiles behave as filters when used in intimate contact with the Berea sands found along the KwaZulu-Natal coastline of South Africa. The study focussed only on uni-directional flow conditions through the soil. The gradient ratio according to ASTM D5101 proved to be a satisfactory method for determining the geotextile's clogging potential and, ultimately, its suitability as a filter in the Berea sands.

The analysis of the laboratory test results highlighted and confirmed the high variability of the Berea sands and the challenges that confront engineers wanting to design geotextile filters to operate in it. The study also highlights that fine gap and semi-gap graded soils with little or no cohesion proved to be the most problematic to the geotextiles.

The co-efficients of uniformity of the soils were widespread and no inter-soil correlation could be made as to which co-efficient of uniformity performed best with the tested geotextiles. It could be concluded that the non-plastic soil with the highest co-efficient of uniformity was the most problematic to the geotextiles in terms of clogging.

It was confirmed that out of twelve soil to geotextile tests performed, only five permutations behaved satisfactorily, equating to a failure rate of 58.33%. Although the gradient ratio provides an early warning that a geotextile might clog or block, the study also proved that the permeability criteria are just as important.

None of the geotextiles met the criteria regarding number of constrictions. The low number of constrictions encountered in the geotextiles tested could possibly be the reason why there were a high number of geotextile failures. In order for the geotextiles to have met this criterion, they would have to be thicker. This would have cost implications which could render the use of geotextiles as questionably non-competitive against conventional drainage techniques.

With all the uncertainties that confront engineers wanting to design for the use of geotextile filters in Berea sands, it is recommended that the severity of the conditions and whether the drain is critical or not should be evaluated. It should then be decided whether laboratory tests are necessary to complement their design. In the case of Berea sands it is highly recommended that these laboratory tests be considered in the design process.

## 6.2 FILTER DESIGN CRITERIA

It has been established that the existing recommendations for geotextiles in drainage works in a South African context is out of date and that they do not consider the soil to geotextile interaction. Most of the guidelines seem to concentrate on the mechanical characteristics of the geotextile such as tensile strength and CBR, whereas very little is mentioned by way of the geotextiles' hydraulic characteristics such pore size, thickness and number of constrictions. Some of the international filter criteria guidelines were used, in conjunction with the properties of the Berea sand, to predict geotextile pore opening sizes. It was found that there was poor correlation between the recommended filter opening sizes versus the results of the laboratory tests.

## 6.3 RECOMMENDATIONS FOR FURTHER STUDIES

The study was limited to four variants of geotextile and three variants of Berea sand. It is therefore recommended that more variants of Berea sand be tested with the same geotextile types. This would be useful in obtaining repeatability of results upon which to develop specific geotextile filter criteria for Berea sands. The relevance of geotextile fibre diameter and stiffness were not discussed in this study and it would be useful to know what effect these parameters would have on the geotextile compatibility with Berea sands. Lastly, the study was limited to a hydraulic gradient of 1 and more tests should be conducted at higher hydraulic gradients to ascertain how these geotextiles would perform in deep drains or in conjunction with sand tailings, especially on the KwaZulu-Natal north coast.

There is much to gain from knowing more about how geotextiles perform in a challenging environment, such as the Berea sands. It is not the intention that the conclusions of this study should suggest refraining from the use of geotextiles in Berea sands, but merely to highlight the potential risks associated with it. If approached in the correct manner, there are many benefits to be gained by the use of geotextiles.

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# APPENDIX A – Soil grading analysis

| | | | |
|---|---|---|---|
| **Client** | Justin November | **Test Number** | 20130312 |
| **Consultant** | | **Date Received** | |
| **Project** | Soil to Geotextile research - Berea sand | **Date Start** | |
| | | **Date Fin** | |
| **Sample ID** | Sands | **Tested by** | JN |
| | | **Compiled by** | JN |

| **Sample Details :** | Sample 1 | Sample 2 | Sample 3 | Data Form 4 | Test Status |
|---|---|---|---|---|---|
| **Total mass** | | | | | |

| **Particle**<br><br>**Size - mm** | % passing | % passing | % passing | % passing | |
|---|---|---|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **5.6000** | 100 | 100 | 100 | | |
| **4.7500** | 100 | 100 | 100 | | |
| **2.0000** | 99 | 100 | 100 | | |
| **1.0000** | 95 | 100 | 98 | | |
| **0.4250** | 80 | 99 | 96 | | |
| **0.2120** | 41 | 63 | 58 | | |
| **0.1500** | 24 | 35 | 38 | | |
| **0.0750** | 14 | 27 | 31 | | |
| **0.0450** | 11 | 22 | 16 | | |
| **0.0120** | 5 | 14 | 15 | | |
| **0.0036** | 2 | 11 | 12 | | |
| **0.0020** | 1 | 10 | 10 | | |
| **0.0015** | 1 | 9 | 8 | | |

# APPENDIX B – Calculated soil parameters

**SOIL SAMPLE 1**

| | | | |
|---|---|---|---|
| AUTOSTABILITY | d85 / d15 > 5 | 7.17 | Soil Stable |
| COEF. OF UNIFORMITY | Cu = d60 / d10 < 15 | 7.95 | Soil Well Graded |
| COEF.OF CURVATURE | d30^2 / d60 * d10 | 2.36 | Soil Internally Stable |
| CLAY TO SILT RATIO | % Clay (0.002) / % Silt (0.075) > 0.5 | 0.097 | Problem Soil |
| COHESION | | | non-cohesive |
| Critical Area: Percent between 20 and 100 micron. | 0.02 mm < d < 0.1 mm > 50% | 0.107 | OK |

**SOIL SAMPLE 2**

| | | | |
|---|---|---|---|
| AUTOSTABILITY | d85 / d15 > 5 | 20.62 | Soil Stable |
| COEF. OF UNIFORMITY | Cu = d60 / d10 < 15 | 95.2 | Soil Broadly Graded |
| COEF.OF CURVATURE | d30^2 / d60 * d10 | 23.2 | Soil Internally Unstable |
| CLAY TO SILT RATIO | % Clay (0.002) / % Silt (0.075) > 0.5 | 0.57 | Soil OK |
| COHESION | | | cohesive |
| Critical Area: Percent between 20 and 100 micron. | 14% Soil OK | | |

**SOIL SAMPLE 3**

| | | | |
|---|---|---|---|
| AUTOSTABILITY | d85 / d15 > 5 | 32.69 | Soil Stable |
| COEF. OF UNIFORMITY | Cu = d60 / d10 < 15 | 111.56 | Soil Broadly Graded |
| COEF.OF CURVATURE | d30^2 / d60 * d10 | 12.05 | Soil Internally Unstable |
| CLAY TO SILT RATIO | % Clay (0.002) / % Silt (0.075) > 0.5 | 0.48 | Problem Soil |
| COHESION | | | non-cohesive |
| Critical Area: Percent between 20 and 100 micron. | 0.02mm < d < 0.1mm > 50% | 0.18 | Soil OK |