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Soil-geotextile compatibility in filtration

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ABSTRACT: Geotextiles have been used in drainage and filtration for over 30 years in different applications in civil and environmental projects. Empirical design criteria have been used in the selection of geotextiles for these applications, and the overwhelming success of geotextiles as drainage and filter materials has yielded confidence in current practice. However, it has been acknowledged that current practice is far from taking into account the actual behaviour of geotextile filters in view of the complex nature of the conditions to which they are subjected in the field. This may lead to overly conservative designs, a rejection of the use of geotextiles in some works, or a very restricted use of these materials in major engineering projects. This paper presents a critical review of current design practices for geotextile filters. Emphasis is placed on characterising soil-geotextile interaction, given the in service conditions. The results and conclusions address the extent to which the actual filter behaviour may vary from conditions assumed in testing and design, suggesting a need for more realistic approaches to the study of the interaction between soil, geotextile and fluid.

1 INTRODUCTION

Geotextiles have been used as drains and filters in geotechnical engineering works for over thirty years. Their use has also been extended to geoenvironmental problem, particularly in the last two decades. Over the years the factors affecting the behaviou– of geotextile filters have been studied by many researchers and summarised in some classical works in the literature (Gourc and Faure, 1990, and Giroud, 1996, for instance). Indeed, several factors can influence the performance of filters in general and these structures are supposed to function appropriately for a long time in several engineering works. Regarding geotextiles, some engineers (particularly old fashioned professionals) still fear to use them even in what some times might be considered as rather ordinary projects. Some fear that geotextiles are very thin and compressible layers, very different from the characteristics of traditional granular filters with whose there are decades of experience. Under this perspective, these characteristics might yield to the conclusion that geotextiles would be apparently more likely to fail in retaining soil particles, increasing the potential for soil piping. Others fear that there is not enough long term experience with such synthetic materials to assure that geotextiles will last long enough, or behave well enough with time, to fulfil their role in major engineering projects. Research results and case-histories over the last decades have proved that most of these concerns can be dealt accordingly to accomplish a safe and lasting performance. Indeed, most of the design methodologies involving geotextile filters are quite conservative. One of the aims of this work is to show that there is already enough evidence on the behaviour of geotextile filters in order to improve design methodologies with implication to the extension of the use of these material to a broader range of engineering projects.

As in the case of any filter material, besides piping, another important reason for concern is clogging. With respect to geotextiles, failure to provide an efficient filter layer can occur due to blinding, blocking or internal clogging, as schematically presented in Figure 1. Blinding can occur when the geotextile is in contact with a internally unstable soils, which may be subjected to suffusion, yielding to the movement of finer soil particles that are retained by the geotextile layer. This layer of fine particles form a low permeability film causing a severe reduction of flow rate. Internal clogging may occur due to the retention of a large number of soil particles in the geotextile pores or as a result of the precipitation of chemical substances or bacteriological activity. The mechanisms depicted in Figure 1 show that for a good performance of the system soil and geotextile have to be compatible, i.e., special requirements related to physical characteristics of these materials must be met. Chemical compatibility is also required to avoid chemical clogging or geotextile degradation. Even so, depending on the project characteristics, fluid flow may not be stable or unidirectional in time or project type and soil characteristics may favour biological clogging, which impose additional complications to the problem. However, the constraints mentioned above are not exclusive for geotextile filters and, being these materials manufactured, some of these constraints may be dealt with in a more comprehensive way and under greater control than in the case of natural filter materials.

This work aims to present a critical analysis of the evolution of the use of geotextile filters in the last decades emphasising the common empiricism and conservatism of current design criteria and, at the same time, to point out the complex nature of the interaction between soil, water and geotextile and the challenges yet to be fought for a broader a and more scientific use of geotextiles.

2 DESIGN CRITERIA

2.1 *Current Design Practice*

The design, specification and construction of filters are important to the performance of many geotechnical structures. Appropriate measures for filtration are often required for long-term serviceability of the structure. The filter itself is a porous medium that acts primarily to retain the base soil against which it is placed without impeding the through-flow of groundwater seepage. As such, criteria for soil retention and permeability govern filtration applications. When properly designed, this protective measure ensures unimpeded groundwater flow while preventing any unacceptable movement of fine particles at the interface of the base soil and filter.

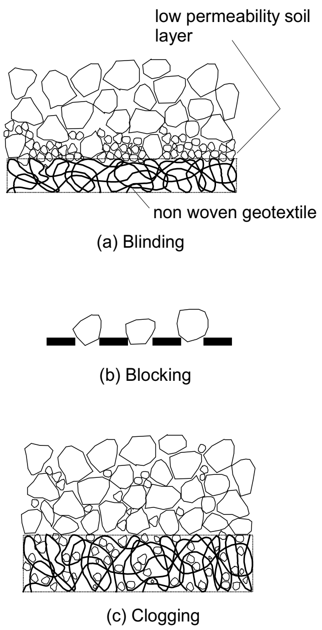


Figure 1. Clogging mechanisms of geotextiles.

Typically, the filter is either constructed from granular soils that are placed to meet a recommended specification or, alternatively, it comprises a geotextile that is manufactured and selected in the knowledge that the material properties conform to a recommended specification. Occasionally, the compatibility of base soil and candidate filter is assessed from laboratory permeameter tests, in order to validate a proposed specification of materials in design.

Design approaches for both granular and geotextile filters are based largely on experience and are wholly empirical in origin. Given this empiricism in design practice, it is important to properly characterise both the nature of the hydraulic regime and the gradation of the base soil. With regard to the influence of hydraulic regime, a distinction is made between a condition of steady unidirectional flow and a condition of unsteady reversing flow that is dynamic, pulsating or cyclic. The distinction is made in the belief that a natural bridging network tends to form in the base soil adjacent to the filter (Lawson, 1982). However, it may not develop under conditions of reversing flow, where the influence of changing direction of seepage forces acts to destabilise any bridging network (Giroud, 1982; Kohler, 1993). With regard to the influence of base soil gradation, a distinction is made between soils that are internally stable and those with a potential for internal instability (Kenney and Lau, 1985). Stable soils, for example uniform gradations and linearly-graded broad gradations, do not experience a loss of particles due to disturbing forces such as seepage and vibration. In contrast, unstable soils are susceptible to internal migration of fine particles. Examples include gap-graded soils and soils with a gradation curve that exhibits an upwardly concave shape (Lafleur, 1984; Lafleur et al., 1989). The latter study yielded a proposed threshold value for the mass of base soil loss per unit area, which was used to distinguish between a stable and unstable filtration system, and defined as 2500g/m^2.

Early laboratory studies of granular filters examined, for conditions of unidirectional flow only, the compatibility of uniformly-graded filters placed against uniformly-graded base soils (Bertram, 1940). The objective was to validate filter criteria for base soil retention and permeability advocated by Terzaghi and subsequently reported in Terzaghi and Peck (1948). Later studies (notably, Karpoff, 1955; Sherard et al., 1984a; Sherard et al., 1984b) sought to establish and evaluate arbitrary limitations on granular filters placed against broadly-graded base soils, again for conditions of unidirectional flow. Lafleur et al. (1989) extended the approach to include internally unstable soils. Throughout all of these studies using laboratory permeameters, the need to ensure a full saturation of the test specimen and permeate it with de-aired water was noted, in order to obtain reliable measurements of permeability with time. To summarise, the impact of these permeameter studies on geotechnical practice has been to impart great confidence to empirical design criteria for granular filters against uniformly-graded base soils, and reasonable confidence to design of granular filters for broadly-graded and gap-graded soils, for conditions of unidirectional flow. The empiricism lies in criteria established from interpretation of laboratory observations, rather than criteria arising from theoretical analysis.

Empirical design criteria, for base soil retention by a granular filter, limit the ratio of a perceived characteristic opening size of the filter (O_F) to a selected indicative particle size of the base soil (d_i), where:

$$O_F < d_i \tag{1}$$

Experience shows that, for a stable base soil, retaining the d_{85} particles leads to retention of the finer particles. Laboratory observations also suggest the characteristic opening size of a dense uniform granular filter is closely related to the D_{15} particle size (Sherard et al., 1984a), yielding the classic expression:

$$D_{15}/d_{85} < RR \tag{2}$$

where RR is a retention ratio. Bertram (1940) established the limit of compatibility to occur at RR = 6, and Terzaghi and Peck (1948) advocated a value of RR = 4 for purposes of design. While the difference in values may be interpreted as a margin of safety for practical purposes, it would be incorrect to define it as a factor of safety.

In situations of soil retention by a geotextile filter, the basic design criteria are predicated on the same methodology that governs granular filters. They limit the ratio of a characteristic opening size of the geotextile to an indicative particle size of the base soil. In contrast to a granular filter, the variation of opening sizes in a geotextile can be established with relative ease. Several standard test methods have been developed, which employ dry sieving, wet sieving, hydrodynamic sieving, pore intrusion methods or image analysis respectively. The characteristic opening size O_{95} defines a pore size for which 95 percent of the openings are the same size or smaller. Hence for a geotextile (after Calhoun, 1972; Carroll, 1983; Christopher and Holtz, 1985; Rollin and Lombard, 1988; Luettich et al., 1992; Lafleur, 1999):

$$O_{95}/d_{85} < RR \tag{3}$$

Table 1 summarises a large number of retention criteria found in the literature (Palmeira and Gardoni, 2000a). This table shows the variety of approaches and different design philosophies currently used by designers in different parts of the world.

Bhatia et al. (1996) report a significant difference in the value of O_{95} for a geotextile established from each standard test method, confirming earlier findings by Faure et al. (1986). While not surprising, it does confirm the need to ensure any designated value of retention ratio be coupled with use of a specific standard test method to measure the characteristic opening size of the geotextile.

Early laboratory studies of geotextile filters examined, for conditions of unidirectional flow only, compatibility of the soil and fabric. The objective, as in the initial studies of granular filters, was to validate design criteria for base soil retention and permeability. In addition, recommendations were made based on theoretical analyses and inspection of criteria for granular filters (Giroud, 1982). More recent laboratory studies of geotextile filters have examined conditions of dynamic, cyclic and pul-

Table 1. Geotextile retention criteria (modified from Palmeira and Gardoni, 2000).

Source	Criterion	Remarks
Ragutzki (1973)(*)	$O_f \le 0.5D_{50}$ to $0.7D_{50}$ $O_f \le 0.5D_{50}$ to $1.3D_{50}$ $O_f \le 0.5D_{50}$ to $1.5D_{50}$	Wovens and non wovens, dynamic/reverse flow, unconfined soil Wovens, dynamic/reverse flow, confined soil Non wovens, dynamic/reverse flow, confined soil
U.S. Corps of Engineers (1977)	0.149 mm $\le O_{95} \le 0.211$ mm 0.149 mm $\le O_{95} \le D_{85}$	$D_{50} > 0.074$ mm $D_{50} \le 0.074$ mm Geotextiles should not be used if D_{85}<0.074mm
AASHTO Task Force #25 (1986)	$O_{95} < 0.59$ mm $O_{95} < 0.30$ mm	If 50% ≤ 0.074 mm If 50 % > 0.074 mm No limitations on geotextile type nor soil type
Calhoun (1972)	$O_{95}/D_{85} \le 1$ $O_{95} \le 0.2$ mm	Wovens, soils with ≤ 50 % passing no. 200 sieve Wovens, cohesive soils
Zitscher, 1974 (from Rankilor, 1981)	$O_{50}/D_{50} \le 1.7–2.7$ $O_{50}/D_{50} \le 2.5-3.7$	Wovens, soils with $C_u \le 2$, $D_{50} = 0.1$ to 0.2 mm Non wovens, cohesive soils
Ogink (1975)	$O_{90}/D_{90} \le 1$ $O_{95}/D_{85} \le 1.8$ $O_f \le D_{85}$(*) $O_f \le D_{15}$(*)	Wovens Non wovens Dynamic/reverse flow, wovens and non wovens, with formation of a natural filter Dynamic/reverse flow, wovens and non wovens, without the formation of a natural filter
Sweetland (1977)	$O_{15}/D_{85} \le 1$ $O_{15}/D_{15} \le 1$	Non wovens, soils with $C_u = 1.5$ Non woven, soils with $C_u = 4$
Schober & Teindl (1979)	$O_{90}/D_{50} \le 2.5–4.5$ $O_{90}/D_{50} \le 4.5–7.5$	Woven and thin non wovens, dependent of C_u Thick non wovens, dependent of C_u, silts and sand soils
Teindl & Schober (1979)(*)	$O_f \le D_5$ to D_{85}	Dynamic/reverse flow conditions, wovens and non wovens, dependent of hydraulic gradient
Millar et al. (1980)	$O_{50}/D_{85} \le 1$	Wovens and non wovens.
Rankilor (1981)	$O_{50}/D_{85} \le 1$ $O_{15}/D_{15} \le 1$	Non wovens, soils with $0.02 \le D_{85} \le 0.25$ mm Non wovens, soils with $D_{85} > 0.25$ mm
Giroud (1982)	$O_{95}/D_{50} < C'_u$ $O_{95}/D_{50} < 9/C'_u$ $O_{95}/D_{50} < 1.5C'_u$ $O_{95}/D_{50} < 13.5/C'_u$ $O_{95}/D_{50} < 2C'_u$ $O_{95}/D_{50} < 18/C'_u$	$I_D < 35\%$, $1 < C'_u < 3$ $I_D < 35\%$, $C'_u > 3$ $35\% < I_D < 65\%$, $1 < C'_u < 3$ $35\% < I_D < 65\%$, $C'_u > 3$ $I_D > 65\%$, $1 < C'_u < 3$ $I_D > 65\%$, $C'_u > 3$ Assumes fines in soil migrating for large C_u
Carroll (1983)	$O_{95}/D_{85} \le 2 – 3$	Wovens and non wovens
Heerten (1982)	$O_{90} < 10D_{50}$ and $O_{90} \le D_{90}$ $O_{90} < 2.5D_{50}$ and $O_{90} \le D_{90}$ $O_{90} < D_{50}$ $O_{90} < 10D_{50}$ and $O_{90} \le D_{90}$ and $O_{90} \le 0.1$mm	Cohesionless soils, with CU ≥ 5 and static load conditions Cohesionless soils, with CU< 5 and static load conditions Cohesionless soils, dynamic load conditions Cohesive soils and all load conditions
Mlynarek (1985), Mlynarek et al. (1990)	$2\,D_{15} < O_{95} < 2\,D_{85}$	Non wovens
Lawson (1986)	$O_{90}/D_n = C$	Developed for residual soils from Hong Kong Values of n and C are obtained by a chart defining regions of acceptable filter performance
Lawson (1987) (from GEO, 1993).	$O_{90}/D_{85} \le 1$ 0.08 mm $\le O_{90} \le 0.12$ mm 0.03 mm $\le O_{90} \le D_{85}$	For predominantly granular soils with D > 0.1 mm, e.g., residual soils which are granular in nature and alluvial sandy soils For non-cohesive soils, e.g., silts of alluvial or other origin, and for non-dispersive cohesive soils. For dispersive cohesive soils
John (1987)	$O_{95}/D_{50} \le (C'_u)^a$	a is dependent of the of the size of the particle to be restrained (a = 0.7 for D_{85})
FHWA-Christopher & Holtz (1985)	$O_{95}/D_{85} \le 1–2$ $O_{95}/D_{15} \le 1$ or $O_{50}/D_{85} \le 0.5$	Dependent of soil type and C_u. Dynamic, pulsating and cyclic flow if soil can move beneath geotextile
CFGG (1986)-French Committee on Geotextiles and Geomembranes	$O_f/D_{85} \le 0.38–1.25$ $O_f \le 0.5D_{85}$(*) $O_f \le 0.75D_{85}$(*)	Dependent of soil type, compaction, hydraulic and application conditions Reverse flow, wovens and non wovens, loose soil Reverse flow, wovens and non wovens, dense soil
Fischer, Christopher & Holtz (1990)	$O_{50}/D_{85} \le 0.8$ $O_{95}/D_{15} \le 1.8–7.0$ $O_{50}/D_{50} \le 0.8–2.0$	Based on geotextile pore size distribution, dependent of C_u of soil.
Rollin et al. (1990)	$O_{95} < 1$ to $1.5\,D_{85}$	Tests with a fine sandy soil and 3 non woven , needle-punched geotextiles using an upflow filtration apparatus.
Luettich et al. (1992)	Design charts	Based on geotextile void size, soil size and type, hydraulic conditions and other factors

CGS (1992)	$O_f/D_{85} < 1.5$ $O_f/D_{85} < 3.0$	Uniform soils Broadly graded soils
OMT (1992)	$O_f/D_{85} < 1.0$ and $O_f > 0.5\ D_{85}$ or 40 μm	Non wovens geotextiles preferred, $t_{GT} > 1$mm, avoid thermally bonded geotextiles.
UK DTp – Murray and McGown (1992), from Corbet (1993)	$O_{90}/D_{90} = 1$ to 3 $O_{90}/D_{90} < 1$ to 3 $O_{90}/D_{50} < 1.8$ to 6	Soils with $1 \le C_u \le 5$, wovens and non wovens. Soils with $5 < C_u < 10$, wovens and thin non wovens ($t_{GT} \le$ 2mm) - alternative criterion Soils with $5 < C_u < 10$, thick non wovens ($t_{GT} > 2$mm) - alternative criterion
Fannin et al. (1994b)	$O_f/D_{85} < 1.5$ and $O_f/D_{50} < 1.8$ $O_f/D_{85} < 0.2$, $O_f/D_{50} < 2.0$, $O_f/D_{50f} < 2.5$ and $O_f/D_{15} < 4.0$	Non woven geotextiles, $1 < C_u < 2$ Non woven geotextiles, $3 < C_u < 7$
Bhatia and Huang (1995)	$O_{95}/D_{85} < 0.65\text{-}0.05C_c$ $O_{95}/D_{85} < 2.71\text{-}0.36C_c$ $O_{95} < D_{85}$	$n < 60\%$ and $C_c > 7$ $n < 60\%$ and $C_c < 7$ $n < 60\%$
Lafleur (1999)	$O_f/D_I < 1$ $1 < O_f/D_I < 5$	Stable soils ($C_u \le 6$ and $D_I = D_{85}$ in this case), soils with $C_u > 6$ but linearly graded ($D_I = D_{50}$ in this case), gap graded ($C_u > 6$) internally stable soils ($D_I = D_G$) and soils with $C_u > 6$ with gradation curve concave upwards and internally stable ($D_I = D_{30}$) Unstable soils with: $D_I = D_{30}$ for gap graded internally unstable soils and for internally unstable soils with gradation curves concave upwards (risk of piping of fines) Criteria developed for cohesionless soils

Notes: C_c = coefficient of curvature of the soil = $D^2_{30}/(D_{60}D_{10})$, C_u = coefficient of uniformity of the soil = D_{60}/D_{10}, C'_u = linear coefficient of uniformity of the soil = $(D'_{100}/D'_0)^{0.5}$, D_G = minimum soil gap size, D_I = indicative size of the protected base soil, D_{50f} = mean particle size of the soil fraction smaller than the value of O_f for the geotextile, D_Y = soil particle size corresponding to "Y" percent passing, D'_Y = soil particle size corresponding to "Y" percent passing obtained from a straight line fitting of the central part of the soil gradation curve, I_D = Density index (relative density), n = geotextile porosity, O_f = filtration opening size based on hydrodynamic sieving, O_X = geotextile opening size corresponding to "X" particle size based on dry glass bead sieving, t_{GT} = geotextile thickness, (*) from Faure (1988).

sating flow (Cazzuffi et al., 1999; Fannin and Hameiri, 1999; Chew et al., 2000; Fannin and Pishe, 2001). Retention ratios in the range 1 to 2 are typically proposed for unidirectional flow, and lower values of 0.5 for reversing flow (Table 1). To summarise, the impact of permeameter studies has been to impart great confidence to empirical design criteria for geotextile filters against uniformly-graded base soils, and reasonable confidence to design of geotextile filters for broadly-graded soils, under conditions of unidirectional flow. Experience with reversing flow remains limited, but suggests the empirical design criteria may be conservative.

A common and very important aspect to filtration design practice is that, for critical or severe applications, the compatibility of base soil and filter be assessed through laboratory testing. Compatibility testing of a base soil and geotextile filter is often made with reference to a permeameter test described in the ASTM Test Method for Measuring the System Clogging Potential by the Gradient Ratio (D5101-90), based on early work by Calhoun (1972). Filtration compatibility is defined qualitatively as no unacceptable piping of soil through the geotextile and, conversely, no clogging of the filter interface. Observations of water head distribution and hence hydraulic gradient in the soil specimen (i_s) and hydraulic gradient across the soil/geotextile zone (i_{sg}) are used to quantify this definition of compatibility, where the value of gradient ratio is given by:

$$GR = i_{sg} / i_s = k_s / k_{sg} \qquad (4)$$

The ratio of permeability (k) follows directly from the principle of continuity of flow. A value less than one implies some particles of soil adjacent to the geotextile have migrated through it, or "washed out", which often occurs in the development of a stable filter ($k_{sg} < k_s$). A continued decrease might suggest incompatibility, with potential for piping. In contrast, a value greater than one implies some impediment to flow adjacent to the geotextile ($k_{sg} > k_s$). Again, a continued increase might suggest incompatibility, with potential for clogging. Therefore, it is reasonable to impose bounds to the value of gradient ratio that quantify filtration compatibility. However, experience on this matter of test interpretation is limited and there is, as yet, no clear consensus on the selection of appropriate bounds.

Recognising that design criteria for geotextile filters share many similarities with criteria for granular filters, it is interesting to reflect on the one significant difference that exists between the two porous media. As noted, the pore size openings of a geotextile are relatively easy to measure, while the pore size openings of a granular filter are relatively difficult to measure. Further recognising that confidence in filtration design is based primarily on the compatibility that exists between particle size of the base soil and opening size of the filter, the need to properly understand factors governing the filtration opening size of geotextile becomes apparent. In this paper those factors are examined, making reference to two construction applications in order to contrast approaches used in design to specify a candidate geotextile.

2.2 *Construction Applications and the Design Process*

A distinction may be drawn, in filtration, between applications in which the base soil and groundwater flow regime are characterised as critical or severe and, in contrast, applications that are characterised as routine (Holtz et al., 1997). The distinction is based on a consideration of risk to life or property, costs of construction and repair, indicators of poor performance, gradation of the base soil, and nature of the seepage flow (Carroll, 1983).

Selection of a candidate geotextile for routine filtration applications is often made with reference to a standard specification document, and without recourse to laboratory tests to evaluate soil-geotextile compatibility. For example, AASHTO M 288-99 describes requirements for permanent erosion control that would typically govern the routine placement of geotextile beneath riprap stone. The standard specification defines a minimum strength requirement, with reference to values from index tests. It also defines a minimum value of permittivity and opening size for the geotextile. Consider an application involving stabilisation of a small tributary channel to the Capilano River, at Vancouver, British Columbia, where it is incised through erodible clayey silts. Placement of riprap stone directly on the base soil, a glacio-lacustrine soil, may result in excessive scour around the stone. The geotextile provides coincident functions of separation and filtration (see Fig. 2). AASHTO M 288-99 requires a minimum permittivity of 1.4 sec-1 and, for this base soil, a maximum Apparent Opening Size of 0.100 mm. The needle-punched nonwoven geotextile used in construction exceeds these requirements and, from subsequent inspection two years later, was found to work well (Fannin, 2000).

Figure 2. Routine application: placement of riprap stone over the geotextile for streambank stabilisation.

In contrast, selection of a candidate geotextile for critical or severe applications must be supported by laboratory tests to evaluate compatibility, one example of which is the gradient ratio test (Holtz et al., 1997). Consider an application at the Alouette Dam, near Vancouver, British Columbia, where a filter was placed to protect the underdrain of a spillway channel (Fig. 3). The alignment of the spillway cuts through a broadly-graded sandy silt with a trace of clay. Again the geotextile provides coincident functions of separation and filtration. Requirements of the geotextile were established from empirical design criteria for soil retention and permeability. Compatibility was evaluated from laboratory tests, including three gradient ratio tests performed on specimens of the sandy silt, against samples of the candidate needle-punched nonwoven geotextile. The results yielded no indication of a potential for unacceptable piping of the base soil through the geotextile, nor any likelihood of unacceptable clogging.

The design process used in each of these construction applications differs significantly. The standard specification of a geotextile for routine applications does not invoke empirical design criteria, or involve materials testing to assess filtration compatibility. Design for a critical or severe application does however make use of empirical criteria, as well as laboratory testing to validate the recommendation. In the latter case, confidence in such a design is predicated on both a thorough understanding of laboratory compatibility tests, and of factors influencing the pore size openings of a geotextile.

3 EXPERIMENTAL INVESTIGATIONS ON SOIL-GEOTEXTILE COMPATIBILITY

3.1 *The behaviour of soil-geotextile systems in filtration tests*

The prediction of soil-geotextile compatibility is a complex task, not only due to the variety of soil types, conditions and behaviour under flow but also to the variety of geotextile characteristics and conditions under which these products are employed. A complete theoretical approach to the problem is still not available, yielding to the need of specific laboratory tests, particularly for severe conditions. As mentioned above, the most common laboratory tests used to assess soil-geotextile compatibility are filtration tests such as the Gradient Ratio (GRT) and the Hydraulic Conductivity tests (HCT). The former is recommended to soils presenting high permeability coefficients, such as sandy soils, whereas the latter is recommended to low permeability soils, such as silts and clayey soils. Figures 4 (a) and (b) present schematically both types of tests.

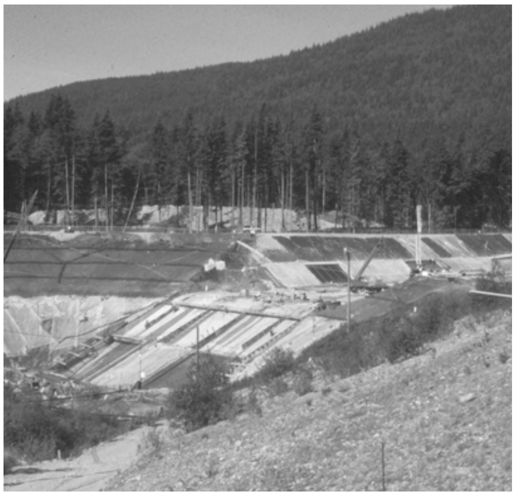


Figure 3. Critical application: construction of an underdrain for a dam spillway.

The Gradient Ratio test is standardised by ASTM D5101-90 (ASTM, 1995) and in this standard procedure water heads are measured in piezometers installed at specific locations along the soil specimen height, as shown in Figure 4 (a). The soil specimen is 102 mm high and 102 mm in diameter and flow of water is induced through the sample under stages of constant total gradients varying from 1 to 10. The measurement of water heads allows the calculation of the Gradient Ratio (GR), defined by ASTM as

$$GR_{ASTM} = \frac{i_{34}}{i_{23}} \qquad (5)$$

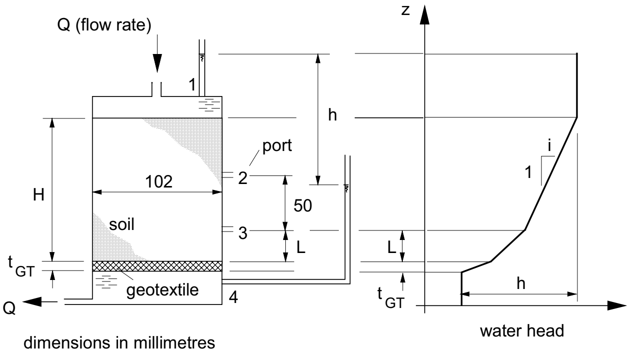
where: GR_{ASTM} = gradient ratio as defined by ASTM, i_{34} = hydraulic gradient between ports 3 and 4 (Fig. 4a), i_{23} = hydraulic gradient between ports 2 and 3.

Port 3 in Figure 4 (a) is located 25 mm above the geotextile specimen (L = 25 mm) in ASTM D5101-90 and, as such, the value of i_{34} in Equation 5 is based on the head loss in the 25 mm thick soil segment plus the head loss in the geotextile. The value of i_{23} is intended to provide a reference as the hydraulic gradient of flow in the soil, satisfactorily away from the geotextile boundary and its influence. Different values of total gradient (i_{14} in Fig. 4a) between the ends of the soil sample are used. The accuracy and implications of these assumptions will be discussed later in this work.

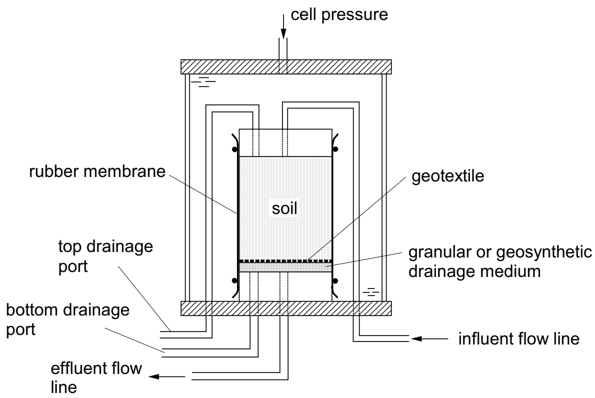
Recent researches have been conducted with some variations of definitions of GR and with modifications in the original apparatus or additions to the standard testing procedure. For instance, provided small changes in the original apparatus and availability of equipment to test small quantities of soil for grain size, the soil particles that pipe through the geotextile during the test can be collected for grain size measurements, allowing to check the accuracy of current retention criteria. To address the differences in GR definition, this value can be expressed in general terms as

$$GR = \frac{i_{LG}}{i_{23}} \quad (6)$$

where L is the distance between port 3 and the geotextile layer (Fig. 4a) and i_{LG} is the hydraulic gradient along the L and the geotextile thickness.



(a) Gradient ratio test apparatus



(b) Hydraulic conductivity test apparatus

Figure 4. Tests for the investigation of soi-geotextile compatibility.

Fannin et al. (1994a) presents a definition of GR (referred hereafter as GR_{Mod}) based on the measurement of water heads closer to the geotextile layer than recommended by ASTM (1995). In this case, the distance L in Figure 4 (a) is equal to 8 mm and the intention is to capture the interaction between soil and geotextile closer to the interface between both materials. Palmeira and Matheus (2000) and Gardoni (2000) used permeameter cells with values of L as low as 4 and 3 mm, respectively.

A theoretical equation relating GR and relevant variables can be derived from the study of water flow through a stratified medium. Palmeira et al. (1996) derived the following equation relating the value of GR to the relevant physical properties and dimensions of the geotextile based on the flow of water normal to a series of permeable layers

$$GR_L = \frac{k_{23}}{k_G} \frac{\frac{k_G}{k_L} + \frac{t_{GT}}{L}}{1 + \frac{t_{GT}}{L}} \quad (7)$$

where k_G is the geotextile permeability, k_L is the permeability of the soil layer with thickness L above the geotextile used for the definition of GR_L and t_{GT} is the geotextile thickness.

Equation 7 shows the factors affecting the value of the gradient ratio expected to be obtained in a laboratory test. The value of GR_L is dependent of the ratio t_{GT}/L and of the ratios between permeability coefficients present in Equation 7. If t_{GT}/L is small, the value of GR_L will tend to the value k_{23}/k_L. In an ideal situation (with t_{GT}/L negligible), where the permeability coefficient of the materials are not affected by the fluid flow, the value of GR will be equal to one. Values of GR below unity indicate soil piping trough the geotextile and above unity indicates some level of clogging of the geotextile. However, the permeability coefficients in Equation 7 are likely to vary during the test. In the case of the geotextile, its permeability may drop as a consequence of geotextile impregnation by soil particles and compression. As for the soil, its permeability may vary because of different intensities of soil particles movement along the specimen height. The latter is particularly relevant to soils susceptible to suffusion (gap graded soils).

Palmeira and Matheus (2000) presented results of gradient ratio tests on artificially clogged nonwoven geotextile and fine to coarse sands. The geotextile specimens were clogged by paraffin impregnation to target effective porosities, calculated from the available free void space remaining in the geotextile. This might simulate the case of geotextile physical or biological clogging, for instance. Figure 5 shows the variation of GR_{ASTM} versus geotextile porosity for some of the tests performed. These results show that, as the geotextile porosity was reduced, the effect of geotextile clogging was first felt in the coarser material (soil A), with the gradient ratio increasing at a greater rate for geotextile porosities below 68%. For the medium (soil B) and finer (soil C) sands the threshold geotextile porosity was of the order of 40%. This behaviour can be anticipated from the analysis of the relative magnitude of the terms in Equation 7, and shows that internal clogging of geotextiles under coarser base soils will lead to higher values of gradient ratios for smaller reductions of geotextile porosities. GR_{ASTM} values above 300 were obtained for geotextile porosities of 15%.

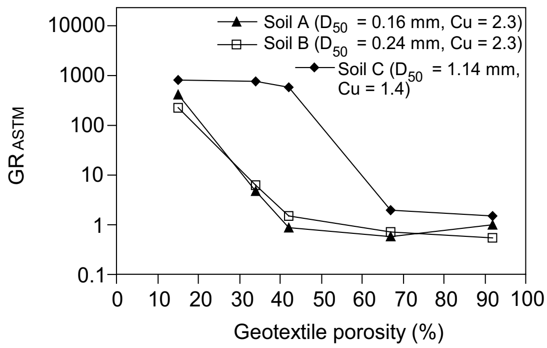


Figure 5. Gradient ratio tests on artificially clogged geotextiles (i_{14} = 2, see Fig. 4a).

The effect of the reduction of the permeability of the zone including the geotextile layer, and of a certain thickness of soil above it, is to increase the pore pressure in that region. Figure 6 (Palmeira and Matheus, 2000) shows the increment in pore pressure at a point in the interface between a soil layer and a zone of reduced permeability coefficient in comparison to the situation, at the same point, without the presence of the reduced permeability layer (geotextile). The results show that the increase in pore pressure may be relevant for combinations of high GR values and low ratios between undisturbed soil heights and reduced permeability layer thickness. A similar study with emphasis to permeability criteria for geotextiles was presented by Giroud (1982).

High values of gradient ratios can be mistakenly taken as geotextile clogging or soil-geotextile incompatibility. It can be easily seen from Equations 4 to 7 that the value of GR increases with the reduction of the geotextile permeability (k_G) or in case of blinding of the geotextile (reduction of k_L). However, high values of GR can also be observed in the case of non uniform instability of the soil specimen structure during tests on reconstituted soil specimens or in tests on heterogeneous undisturbed soil specimens. In these situations, the permeability in the soil layer (k_{23}, in Figure 4 a) can increase during the test of unstable soil samples or can be greater than those in other regions of the soil specimen in test with heterogeneous undisturbed soil samples. As a result, non uniform distributions of head losses along the soil sample height may occur and a greater number of piezometers distributed along the sample height will help understanding the behaviour of the soil-geotextile system under these circumstances. Therefore, GR tests with unstable or undisturbed soil specimens have to be approached with due care, as well as the rejection of a geotextile product based only on a superficial analysis of the value of the gradient ratio obtained. An upper bound equal to 3 for the value of GR has been suggested for the acceptance of a geotextile filter (USACE, 1977, Haliburton and Wood, 1982).

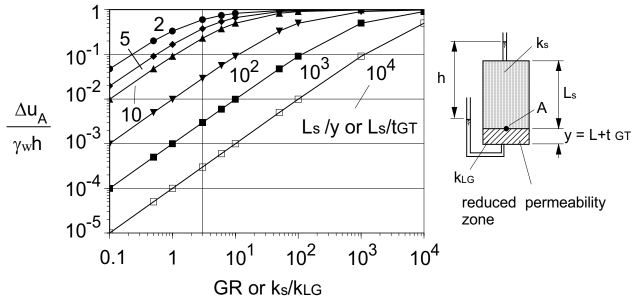


Figure 6. Porepressure increment at soil-reduced permeability layer interface.

The results shown in Figures 5 and 6 raise the question on how conservative the limit $GR_{ASTM} = 3$ may be. Would it be a problem a value of GR above 3 in a stable soil-geotextile system? It seems logical that for GR values above 3 the available flow rate should be compared to the required flow rate for the project. To investigate this assumption, the following equation for the flow rate per unit area can be derived

$$\frac{Q_a}{A} = \frac{k_{23}}{GR_L} i_{LG} \tag{8}$$

where Q_a is the available flow rate, A is the area crossed by the water, k_{23} is the soil permeability under steady flow conditions, GR_L is the gradient ratio based on the length L (Fig. 4a) and i_{LG} is the hydraulic gradient in the region with thickness $L + t_{GT}$.

The maximum value of i_{LG} in Equation 8 is $h/(L + t_{GT})$ (Fig. 4a), where h is the total head loss of the system, assuming that it occurs in the region comprising the soil close to the geotextile and the geotextile itself. In fact, when i_{LG} reaches this maximum value, GR_L tends to infinite and Q_a/A to zero. Equation 8 can be re-written as

$$\frac{Q_a}{A} \approx \frac{k_{23}}{GR_L} \frac{ah}{L + t_{GT}} \geq Q_{req} \tag{9}$$

where a is the fraction of the total head loss in the region comprising the soil thickness L and the geotextile and Q_{req} is the required design flow rate. In a gradient ratio test (ASTM) with a value of GR_{ASTM} equal to 3, approximately 52% (a = 0.52) of the total head loss of the system takes place in the region comprising the bottom 25 mm thickness of soil and the geotextile. For a gradient ratio of 100 the fraction of total head loss in the region close to and including the geotextile reaches 97 %. For the same conditions, a value of GR equal to 3 and a much thicker soil layer the fraction of the total head loss in that region would be considerably smaller.

Equation 8 shows that the available flow rate of the system is inversely proportional to GR. So, for high values of gradient ratio in steady flow conditions, the available flow rate (Q_a) should be compared to the required flow rate (Q_{req}), similar to what is done when Hydraulic Conductivity tests are used, and the relevance of pore pressure increases in the region of the filter (Fig. 6 for one dimensional flow) and changes in the flow characteristics evaluated for stability and safety reasons. Nowadays, these analyses can be conducted using current numerical tools for the simulation of fluid flow in geotechnical works. Nevertheless, the long-term performance of the filter remains as the unknown factor that may affect filter behaviour.

3.2 *Some Considerations on Laboratory Testing Procedure and Implications on Test Interpretation*

As for many laboratory tests, gradient ratio tests results can be particularly sensitive to the testing methodology used (Fischer et al., 1999). Generally the test is appropriate for reconstituted soil specimens, although filtration tests with undisturbed soil samples can be of very important practical relevance. The preparation of reconstituted soil specimens has to lead to an uniform specimen. For uniform soils the procedure recommended by Vaid and Negussey (1988), based on pluviation under water, has proved successful in many research works (Shi, 1993, Gardoni, 1995 and 2000, Palmeira et al., 1996, etc.). For broadly graded soils pluviation techniques may cause undesired segregation of grain dimensions. In this case the procedure recommended by Kuerbis and Vaid (1988) based on slurry deposition should be used. These techniques provide uniform soil samples for filtration tests. In both cases, the soil specimen must be kept saturated. Therefore, the material to be deposited in the permeameter cell can be previously saturated by boiling in deaired water. Saturation of the geotextile specimen has also to be accomplished and that can be obtained by water jetting the geotextile specimen under submersion and further vacuum or by boiling for a few minutes in deaired water. The later technique should be employed only under the assurance that the geotextile to be tested is not sensitive to heat to the extent to affect test results.

The saturation of undisturbed soil samples prior to their placement in the permeameter cell is very complicate and, depending on the technique employed, likely to cause soil disturbance. In the permeameter, flow of CO_2 through the sample can be used, as recommended in ASTM D5101-90 (ASTM, 1995), or saturation can be accomplished by water flow, as the test progresses, which can take a long time, depending on soil permeability. Even if the geotextile specimen was previously saturated it may lose saturation due to air bubbles coming from the soil under flow. Under these circumstances the final degree of saturation of the specimens at the end of the test should be assessed. Due care has also to be taken regarding preferential flow through the cell walls. A layer of bentonite on the internal cell walls or a coat of paraffin on the soil specimen lateral faces (fitting tightly the cell chamber volume) plus wax, have proved successful to avoid that type of problem in tests with undisturbed soil specimens (Gardoni, 1995).

Standard GR tests are performed without the application of normal stresses on the soil specimen top. Therefore, the normal stress reaching the geotextile layer would approximately be caused by the soil specimen weight minus the side friction along the cell internal faces. However, it should be pointed out that seepage forces in tests with large gradients applied to the system can cause important increases of normal stress on the geotextile layer. If the permeameter wall friction is small, for the usual height of soil specimens, a total gradient above 20 can impose normal stresses in excess of 20 kPa on the geotextile layer, due to the action of seepage forces. It should be noted that significant reductions in geotextile pore spaces can occur under these levels of normal stress, as will be seen later in this work.

For a given total gradient applied to the soil-geotextile system, the stabilisation of piezometers readings and flow rate in filtration tests can take from a few hours up to weeks, depending on soil permeability and initial degree of saturation. Distilled water should always be used, with the addition of algaecide when necessary, particularly in long-term filtration tests. Lubrication of the internal walls of the cell has to be provided in GR tests under stress to minimise the effect of side friction on the magnitude of the normal stress reaching the geotextile specimen. Palmeira et al. (1996) and Gardoni (2000) used a lubricating system consisting of plastic sheets and grease to avoid the effects of side friction in the permeameter cell.

Reconstituted sandy soil specimens are often compacted to a target density in current filtration tests. This compaction can be achieved by vibration with a vibrator or cell tapping with a rubber hammer, for instance. The vibration of the specimen cause soil particle to intrude the geotextile pore space to a level still difficult to quantify. The maximum diameter of the soil particles that pass through the geotextile during sample preparation are likely to be related to the geotextile filtration opening size, although the amount of energy used in the compaction may affect the value of that diameter. When water flow starts in the test, the available pore channels in the geotextile are smaller than those under virgin conditions, due to the presence of the entrapped particles in the fibre matrix. Thus, the diameters of the soil particles piped during fluid flow can be considerably smaller than the value of the maximum particle diameter piped obtained in filtration opening size tests (Palmeira et al., 1996, Palmeira and Fannin, 1998). A similar mechanism of geotextile impregnation by soil particles may take place under field conditions.

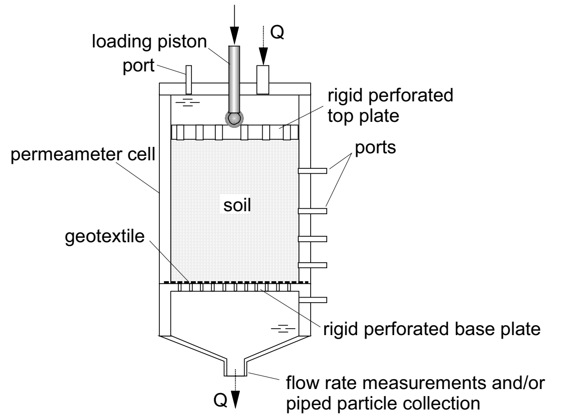
4 THE EFFECTS OF STRESS LEVEL ON SOIL-GEOTEXTILE COMPATIBILITY

4.1 *Gradient ratio tests under confinement*

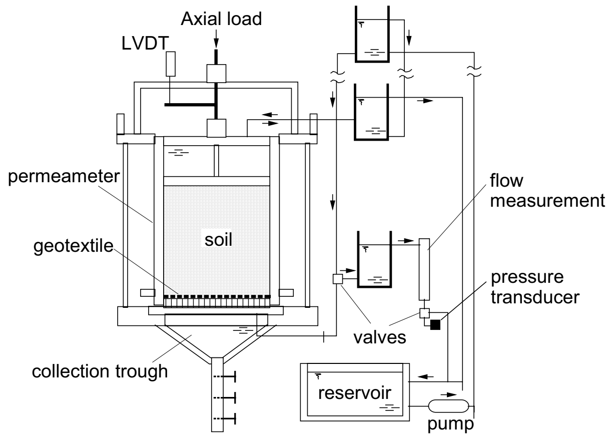
Stress level can affect the value of GR, because of the reduction of geotextile thickness and permeability with normal stress. Up to date most of the GR tests reported in the literature have been performed under unconfined conditions. However, working conditions of geotextile filters in geotechnical and geoenvironmental works always involve some level of confinement, which may vary from a few to thousands of kPa. Large embankments or moderate high mining waste piles can easily induce normal stresses in excess of 1000 kPa on a filter layer. Figures 7 (a) and (b) present equipment arrangements used for tests of soil-geotextile systems under confinement. In Figure 7 (a) a typical arrangement used for tests under one way flow is presented and has been used by several researchers (Shi, 1994, Fannin et al., 1996, Palmeira et al. 1996, Gardoni, 2000). Figure 7 (b) shows an apparatus for testing soil-geotextile systems under confinement and reverse flow (cyclic or dynamic) conditions (Fannin and Hameiri, 1999, Hameiri, 2000). As an alternative to the principle of head control, Cazzuffi et al. (1999) described a device using flow control.

Palmeira et al. (1996) presented results of filtration tests (GRT) under normal stresses up to 200 kPa applied to the sample top. Different types of soils were used in the experiments. The results of these tests are reproduced in Figure 8 and show little influence of normal stresses on the value of GR_{ASTM}, but a clear influence on the value of $GR_{L\,=\,8mm}$ can be identified in some cases. This is caused by the greater role played by term t_{GT}/L (Eq. 7) in the value of GR in the latter case.

Recent results of GR tests carried out under normal stress up to 2000 kPa have been presented by Gardoni (2000). Some these results obtained for a medium sand and a reconstitut sample of a residual soil from quartzite are summarised in Figu 9. The results obtained show that the value of GR_{Mod} (for L = mm) and $GR_{L\,=\,3mm}$ are very sensitive to the increase of norm stress, particularly for the high stress levels employed in the tests. The smaller the value of L (Fig. 4a) used in the calculati of GR the greater is the influence of the normal stress on G For the medium sand (Fig. 9 a) the value of GR_{ASTM} was rath insensitive to the increase of normal stress. For the finer soil (r sidual soil from quartzite, Fig. 9 b) all the values of gradient r tio were affected by the increase of the normal stress. These r



(a) Typical permeameter for filtration tests under confinement

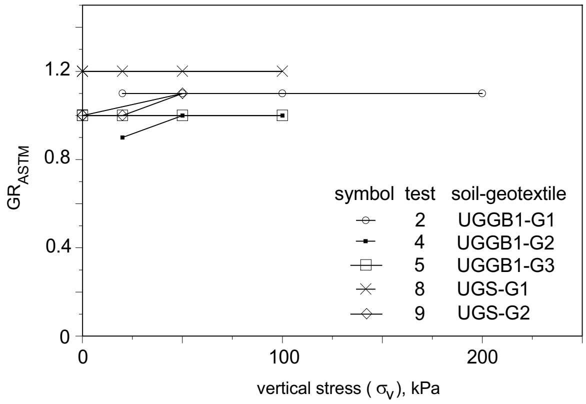


(b) Typical permeameter for filtration tests under confinement and cyclic flow (after Hameiri, 2000)

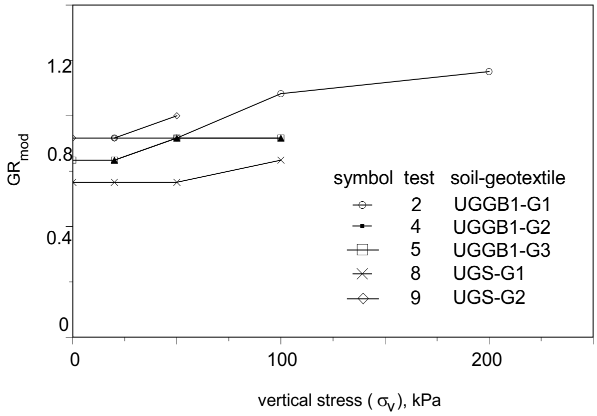
sults suggest that under high normal stresses the conditi $GR_{ASTM} \leq 3$ may not easily be met.

Figure 7. Tests for the investigation of soil-geotextile compatibility u der confinement.

The difficulty in interpreting the results of gradient ratio te is the impossibility of isolating accurately the effect of ea component of the system. For instance, a reduction of geotexti thickness (due to normal stress) also causes a reduction of ge textile permeability in a manner still difficult to predict in the tests. Impregnation of the geotextile by soil particles duri sample preparation or caused by water flow also affects geote tile permeability and compressibility. Figures 10 (a) and (show the variation of thickness and permeability coefficient o nonwoven geotextile with normal stress in filtration tests, f geotextile specimens subjected to different levels of soil impre nation (Palmeira and Gardoni, 2000b). The impregnation lev (λ) in this figure is defined as the ratio between the mass of s particles (M_s) entrapped in the geotextile per unit area and the mass of fibres of the geotextile (M_f) per unit area. The results in Figure 10 show that the presence of entrapped soil particle can significantly affect physical and hydraulic properties of the geotextile.



(a) GR_{ASTM} versus σ_v



(a) GR_{mod} versus σ_v

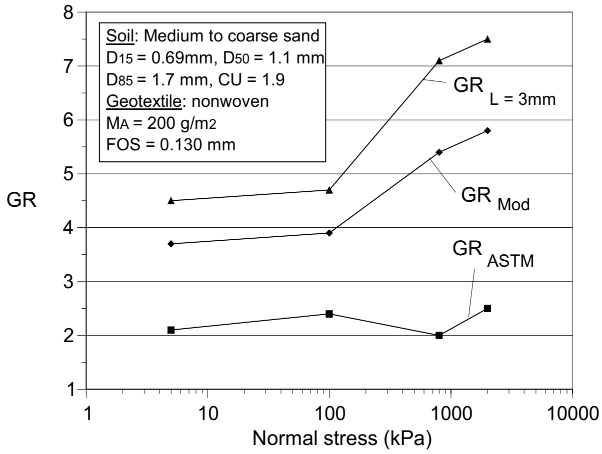
Figure 8. Values of GR in tests under confinement (i_{14} = 2, see Fig. 4a).

It is important to note that the presence of soil particles entrapped in the geotextile, although reducing its normal permeability, may not be necessarily detrimental to the geotextile transmissivity. The rigid soil particles between the fibres reduce the geotextile compressibility. Thus, depending on the level of particle impregnation, the transmissivity of the impregnated geotextile may not be as reduced by the entrapped particles as that of the virgin specimen under the same normal stress (Palmeira and Gardoni, 2000b).

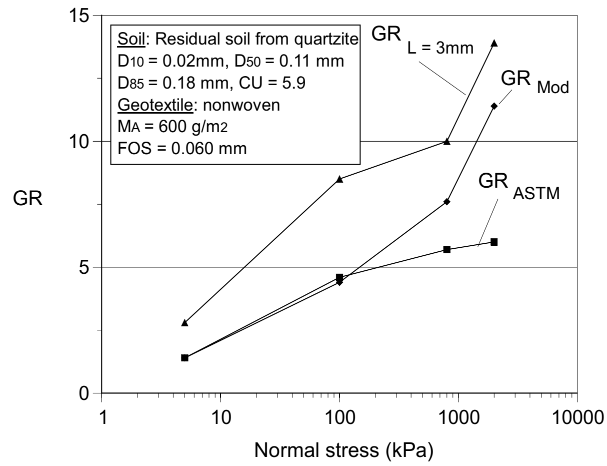
4.2 *Retention capacity of geotextiles under confinement and partially clogged conditions*

4.2.1 The effect of confinement on geotextile pore dimensions

Several important contributions can be found in the literature regarding the distribution of pore dimensions of unconfined non woven geotextiles (Bhatia and Smith, 1996, Bathia et al., 1996, for instance). However, little is found with respect to the pore space dimensions of confined and/or partially clogged geotextiles, which would be the actual working conditions of geotextiles in the field. The effect of the impregnation of the geotextile by entrapped soil particles on its filtration behaviour has been anticipated in pioneer works by Masounave et al. (1980a) and Heerten (1982). Estimates of filtration opening sizes based on geotextile porosity, which can be related to applied stress levels, have been proposed in works by Faure et al. (1989) and Giroud (1996), for instance.



(a) Tests on a medium sand

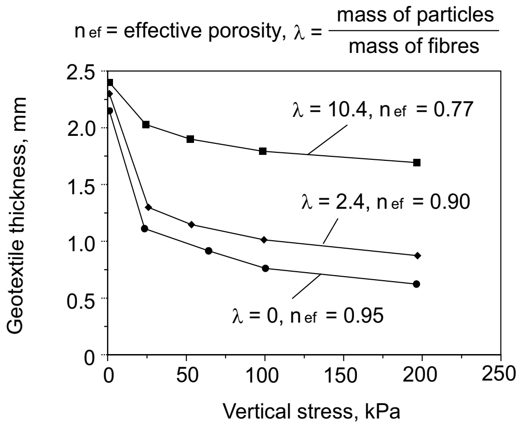


(b) Tests on a residual soil from quartzite

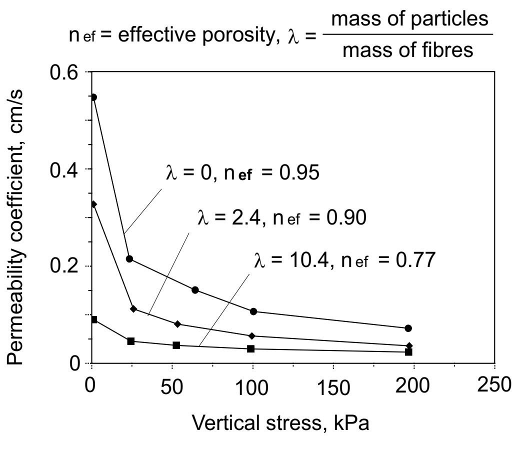
Figure 9. Gradient ratio tests under high normal stresses (Gardoni, 2000).

Gardoni (2000) and Gardoni and Palmeira (2001) presented results of Bubble Point tests and Image Analyses of virgin geotextiles specimens confined under normal stresses up to 1000 kPa. Figure 11 shows curves of distributions of pore constriction diameters under stress obtained from Bubble Point Tests for one of the geotextiles tested (mass per unit area = 200 g/m^2). It can be seen that a considerable amount of reduction of pore constriction dimensions occurred for normal stresses of the order of 20 kPa, which is a value easily reached in rather shallow drains and even in filtration tests under large gradients, as commented before. Little additional reductions were observed in pore constriction dimensions for normal stresses in excess of 50 kPa for the light geotextile tested. Figures 12 (a) and (b) show the variation of pore constriction diameters for different percentages of constrictions smaller than specific values with normal stress for two non woven, needle-punched, geotextiles with masses per unit area equal to 200 and 600 g/m^2, respectively (Gardoni and Palmeira, 2001). The values of O_{98} and O_{95} were more sensitive to the increasing stress level, particularly for the lighter geotextile which exhibits larger opening sizes. On the other hand, the values of O_{50} and O_2 were rather insensitive to the normal stress applied to the geotextile specimens.

The effect of high normal stress levels on geotextile microstructure can be observed in Figures 13 (a) and (b), where cross-sections of virgin geotextile specimens under 2kPa and 1000 kPa normal stress, obtained in a Image Analyser, are shown (Gardoni and Palmeira, 2001). For the unconfined specimen large pores and very few direct contact between geotextile fibres can be identified. Under high stress levels packs of fibres in contact are formed and the pore channels and constriction diameters controlling the passage of soil particles are defined by the space between packed geotextile fibres. The results in Figure 14, from the same authors, show the frequency distribution of opening distances for virgin geotextile specimens under 0 and 1000 kPa normal stresses. The opening distance is defined as the minimum distance between surfaces of neighbour adjacent fibres, which is the dimension available for the passage of a soil particle between two fibres in the plane of the geotextile cross-section. The results show a marked effect of the stress level on the dimensions of geotextile openings, with evident implications to the filtration performance of the geotextile.



(a) Geotextile thickness versus normal stress



(b) Geotextile normal permeability versus normal stress

Figure 10. Effects of geotextile impregnation by soil particles on its physical and hydraulic properties.

To exemplify the aspects discussed above, Figure 15 (Palmeira et al., 1996, Palmeira and Fannin, 1998) shows grain size distributions of glass beads that passed through a non woven geotextile layer in filtration tests (hydraulic gradient of 2) under a 20 kPa normal stress on the sample top. The results show the smaller the geotextile opening size, the finer were the particles that piped through it. The migration of such fine particles, only, is attributed to the entrapment of particles during sample preparation and earlier stages of water flow.

In spite of the complexity of the problem, some theoretical or empirical equations for the estimate of geotextile opening dimensions and filtration opening sizes are available in the literature. Table 2 summarises some of these expressions. To assess the accuracy of these equations Figures 16 to 18 show comparisons between predicted pore dimensions and results from tests on non woven geotextiles under normal stresses varying from 2 kPa to 1000 kPa performed by Gardoni (2000). Geotextiles GA, GB and GC were needle-punched, non woven products, made of polyester, with masses per unit area equal to 200, 400 and 600 g/m^2, respectively. Figure 16 shows the best fit obtained for the predictions by the geometrical model (Laflaive and Puig, 1974, Fayoux and Evon, 1982, Giroud, 1996), which was achieved for a value of δ (Table 2) equal to 1.6. The approach by Masounave et al. (1980 a and b) for the estimate of the average pore diameter compared reasonably well with the measurements from image analyses of geotextile cross sections, as shown in Figure 17. Figure 18 shows that the predicitions of filtration opening sizes by the expression presented by Giroud (1996) compared well with results from Bubble Point tests on confined virgin geotextile specimens when the parameter ξ in the proposed equation (Table 2) was equal to 15.

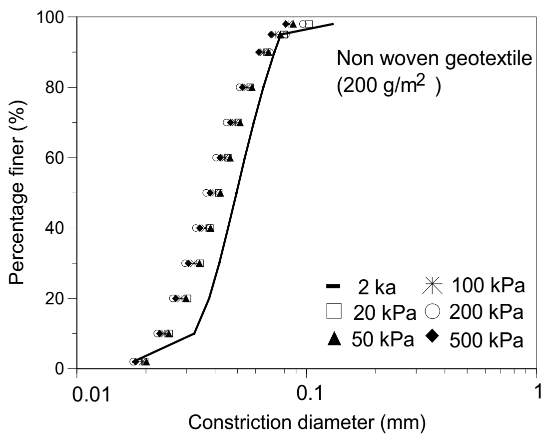


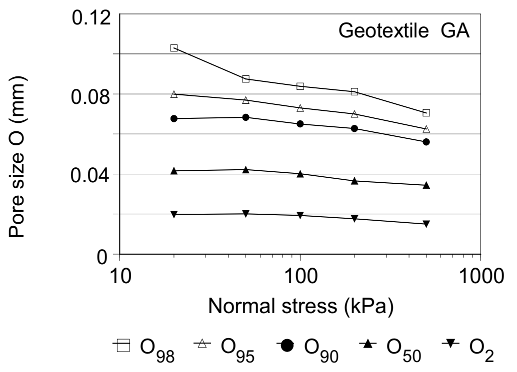
Figure 11. Geotextile constriction size distributions for different stress levels (Gardoni, 2000).

Table 2. Some simple equations for the estimate of geotextile pore dimensions.

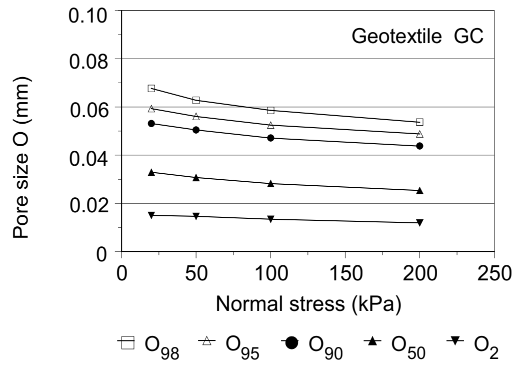
Reference	Equation	Remarks
Geometrical Model: Laflaive and Puig (1974), Fayoux and Evon (1982), Giroud (1996)	$\frac{O_F}{d_f} = \frac{\delta}{\sqrt{1-n}} - 1$	Based on the simulation of non woven geotextiles as pack of regular cylinders; δ = parameter depending on the spatial arrangement of the cylinders.
Giroud (1996)	$\frac{O_F}{d_f} \approx \frac{1}{\sqrt{1-n}} - 1 + \frac{\xi n}{M_A/(\rho_f d_f)}$	Extension of the geometrical model.
Masounave et al. (1980 a and b)	$d_{avg} = \sqrt{\frac{1}{\upsilon'}} - d_f$	Based on a two dimensional probability analysis of a circle to be inserted in the pore area between geotextile fibres.

Notes: O_F = filtration opening size, d_f = fibre diameter, n = geotextile porosity, M_A = geotextile mass per unit area, ρ_f = geotextile fibre density, d_{avg} = average pore diameter, υ' = density of fibres in the geotextile (number of fibres per unit cross-sectional area).

The comparisons presented and discussed above suggest that reasonably accurate predictions of geotextile pore space can be obtained with the use of rather simple equations.

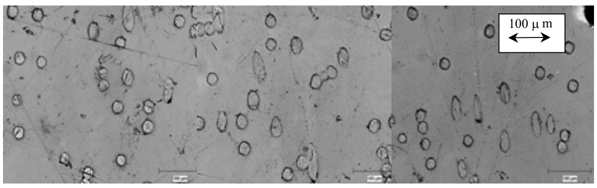


(a) Geotextile GA (M_A = 200 g/m2)

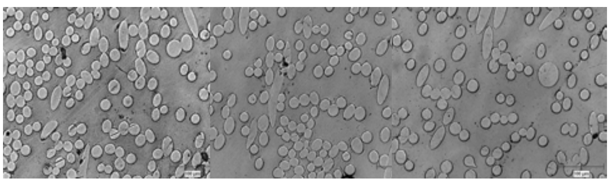


(b) Geotextile GC (M_A = 600 g/m2)

Figure 12. Pore sizes versus normal stress (Gardoni and Palmeira, 2001).



(a) Normal stress equal to 2 kPa.



(a) Normal stress equal to 1000 kPa.

Figure 13. Geotextile cross-sections under confinement (Gardoni and Palmeira, 2001).

4.2.2 Effect of partial clogging and confinement on the retention capacity of geotextiles

The presence of entrapped soil particles in the geotextile adds to the complexity of the filtration mechanism with these materials. Soil particles in the geotextile voids reduce the pore space available for flow and passage of particles from the base soil, as well as the compressibility of the geotextile layer, as mentioned before. Partial clogging of geotextiles can occur in the field during spreading and compaction of soil layers on the geotextile or during flow regime. Figures 19 (a) and (b) show images of an artificially impregnated specimen of a non woven geotextile under different normal stresses, where glass beads were used for the impregnation and Figure 20 shows bridges of fine particles or soil particles clusters in samples of non woven geotextiles exhumed from drains in residual soils (Gardoni, 2000). In Figure 19 (a) the specimen is under 2kPa normal stress and in Figure 19 (b) under 1000 kPa normal stress. The impregnation level (λ) used in this test was equal to 12. It is important to note that in the field the value of λ will be dependent of the type and characteristics of the geotextile, the characteristics of the soil and of the type of compaction procedure used. Table 3 shows the results of impregnation levels of geotextiles below compacted soil layers under laboratory and field conditions (Palmeira and Gardoni, 2000b). Exhumed samples of geotextile under compacted fills in the laboratory and in real works showed values of λ varying between 0.3 and 15. Values of λ varying from 0.3 to 10 were back-analysed from geotextiles specimens exhumed from Valcros Dam (Faure et al., 1999) . It can be demonstrated that a value of λ equal to 12, for instance, corresponds to a reduction of pore space in a non woven geotextile, made of polyester, 1.5mm thick, with mass per unit area equal to 200 g/m^2, of the order of 66%. The results presented in Figures 19 and 20 show a significant reduction on the pore space due to the combined effect of confinement and soil impregnation. Therefore, if significant geotextile impregnation takes place before fluid flow, the reduced pore space will increase the retention capacity of the geotextile and change the conditions for clogging. The same applies for dispersive soils, fluids carrying solids in suspension or unstable flow conditions caused by reverse flow or by the increase in hydraulic gradient inducing the movement of soil particles towards the filter. Regarding situations like the ones shown in Figure 20, stress level increases, loss of the strength of the bridges of particles or clusters or change of flow characteristics may wash out the fine particles up to a new equilibrium condition be reached.

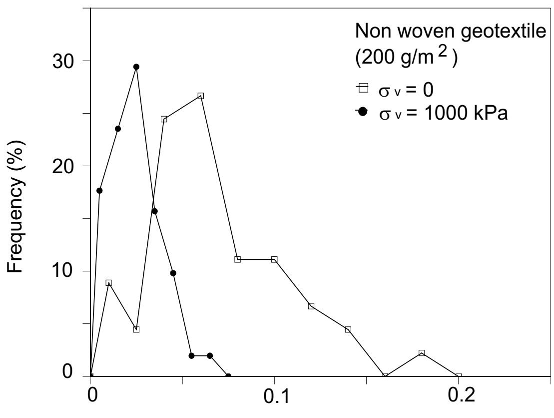


Figure 14. Frequency distributions of geotextile pore sizes.

The individual effects of confinement and partial clogging of the geotextile are somewhat conflicting in reducing the geotextile pore space, because the latter reduces geotextile compressibility, attenuating the influence of the former on pore space dimensions. Figure 19 (b) also shows some breakage of the glass beads due to the high normal stress applied. That mechanism will be relevant for large embankments, waste piles, highly weathered soil grains or soil grains consisting of clusters of smaller soil particles (as in tropical residual soils, for instance).

As the opening dimensions of confined and partially clogged geotextiles are reduced, a more realistic approach to retention criteria might be expressed preliminary based on the actual geotextile filtration opening size, defined as

$$O_{95}^{*} = \frac{O_{95}}{K_{\sigma} K_{pc}} \quad (10)$$

where: O^{*}_{95} is the actual filtration opening size of a confined and partially clogged geotextile, O_{95} is the geotextile filtration opening size of the unconfined virgin geotextile, K_{σ} is a reduction factor for the effect of confinement and K_{pc} is a reduction factor for the effect of partial clogging.

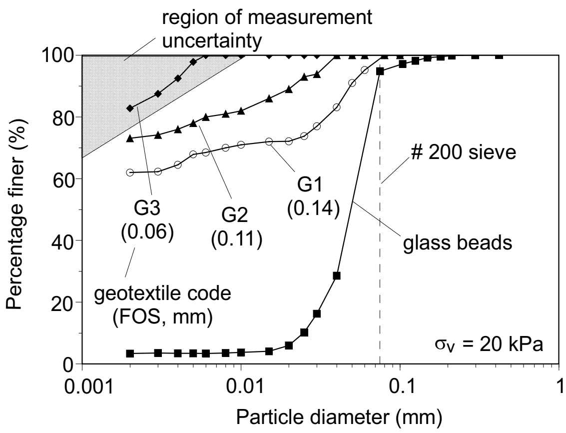


Figure 15. Diameter of piped soil particles in filtration tests under confinement.

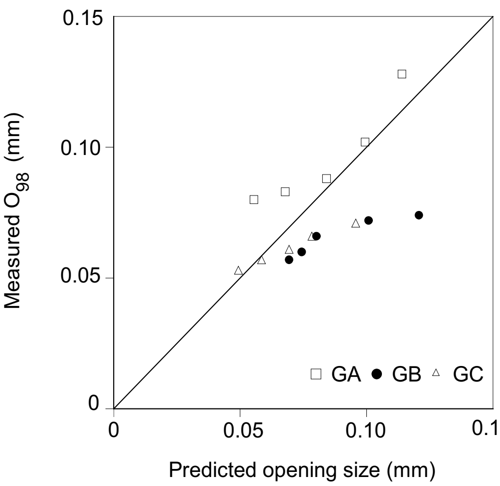


Figure 16. Accuracy of the geometrical model - Bubble point tests under confinement.

The value of K_{σ} observed for some virgin non woven geotextiles varied between 1.2 and 2.2 (Palmeira and Gardoni, 2001). However, this range may not be the same if the geotextile is impregnated with soil particles and subjected to the same stress levels, because of the smaller geotextile compressibility caused by the presence of the entrapped particles. Therefore, the values of K_{σ} and K_{pc} should not be equal to those obtained isolating the mechanisms of pore space reduction (tests with confinement or partial clogging only). Palmeira et al. (1996) presented results of maximum diameter of soil particles (glass beads) that passed through the geotextile in filtration tests (GR tests) under pressure due to the water flow only (isolating the particles piped during sample preparation). The geotextiles used in these experiments were non woven, needle punched, geotextiles made of polyester, with masses per unit area varying between 180 and 600 g/m^2 and Filtration Opening Sizes from Hydrodynamics Sieving (CFGG, 1986) varying from 0.060 to 0.140 mm. Because vibration was used during sample preparation, different levels of geotextile impregnation by soil particles existed before water flow starts. The results of these tests are presented in Figure 21 (Palmeira and Gardoni, 2001), which shows the variation of the product $K_{\sigma}K_{pc}$ in Equation 10 with normal stress on the sample top. In this case, this product varied between 1.9 and 4.4 for the geotextiles tested, which is significantly greater than the range observed for K_{σ} from tests on virgin geotextile specimens referred to above and gives a degree of the influence of the combined effect of confinement and stress level on the retention capacity of non woven geotextiles.

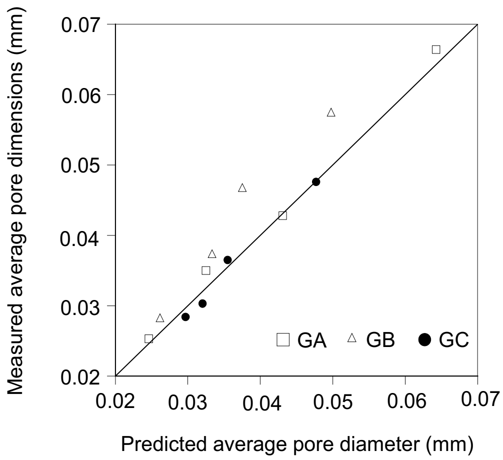


Figure 17. Predicted vs. observed pore diameters – Results from image analyses and predictions by Masounave et al. (1980).

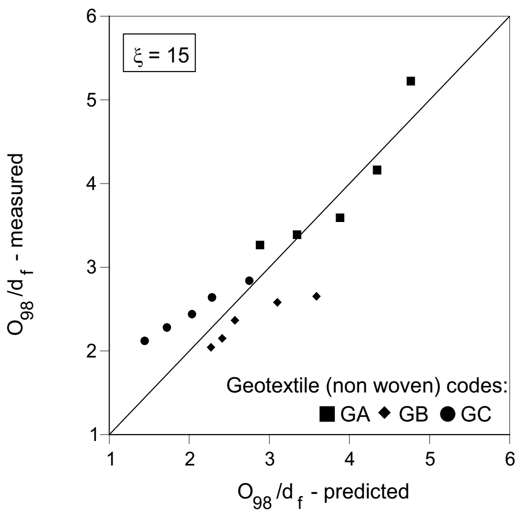
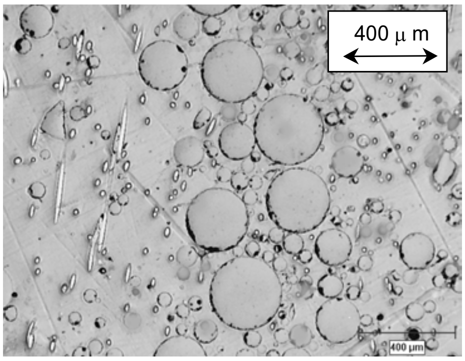
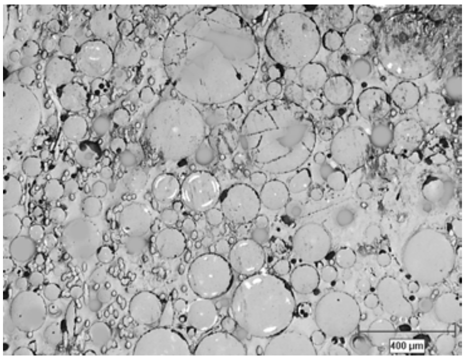


Figure 18. Comparisons between pore sizes from bubble point tests under confinement and predictions by Giroud (1996).

The reduction of pore space caused by the presence of entrapped base soil particles has also repercussion on the criterion to be adopted to verify geotextile internal clogging. Under such conditions the pore constrictions available can be considerably smaller than the usual value of D_{15} of the base soil traditionally used in current clogging criteria such as the ones presented in Holtz et al. (1997), CFGG (1986) and Fisher et al. (1990). This is particularly relevant for soils susceptible to suffusion and for situations of unsteady flow conditions, with regard to the amount and dimensions of soil particles in suspension carried by the fluid.



(a) Normal stress equal to 2 kPa.



(a) Normal stress equal to 1000 kPa.

Figure 19. Cross-sections of partially clogged geotextiles (Gardoni and Palmeira, 2001).

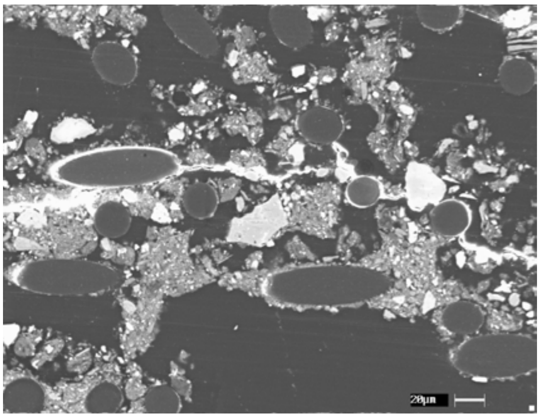


Figure 20. Soil particles entrapped in exhumed specimens of geotextiles.

5 PROBLEMATIC SOILS AND SEVERE CONDITIONS

As commented earlier in this work, special care has to be taken in the filtration of internally unstable base soils. Depending on the amount and dimensions of the fine particles capable of moving through the base soil voids, structural instability of this soil or geotextile blinding/clogging may occur. These soils are characterised by having a grain size curve concave upward or being gap-graded, yielding to large values of coefficient of curvature ($C_c = D^2_{30}/D_{60}D_{10}$) and coefficient of uniformity ($C_u = D_{60}/D_{10}$). Kenney and Lau (1985) states that a soil may be regarded as gap-graded and potentially unstable if the finest 30% does not meet the condition $W_{4D} > 2.3\ W_D$, where W_{4D} and W_D are the percentage weights of particles smaller than given diameters D and 4D, respectively.

Table 3. Level of impregnation of geotextile layers (Palmeira and Gardoni, 2000b).

Geo-textile	Soil Code	Soil Type	Impregnation technique/ condition	λ value or range
GA	SD to SG	Glass beads	vibration/lab.	2 to11
GB	SA	Resid. Soil	water flow	3.01 and 4.76
	SC	Clay	compaction/lab.	0.55
	SC	Clay	compaction/field	0.70
	SD to SG	Glass beads	vibration/lab.	5 to 15
GC	SB	Sand	compaction/field	5.46
	SC	Clay	compaction/field	0.52
GE	SC	Clay	compaction/field	0.37
	SF	Glass beads	vibration/lab.	6.9
GA, GC and GE(2)	See note 1	several(2)	vibration/lab./ water flow	0.8 to 3.3
GX and GY(3)	See note 2	See note 2	compaction/ water flow/field	0.3 to 10

Notes: (1) Impregnation after vibration and filtration tests with glass beads, sand and a silt. Further information on these soils can be found in Palmeira et al. (1996), (2) Nonwoven geotextiles (M_A = 300 and 400 g/m^2) exhumed from the Valcros Dam (Faure et al., 1999). Geotextile porosity (n) assumed as 0.9 in the calculations of λ.

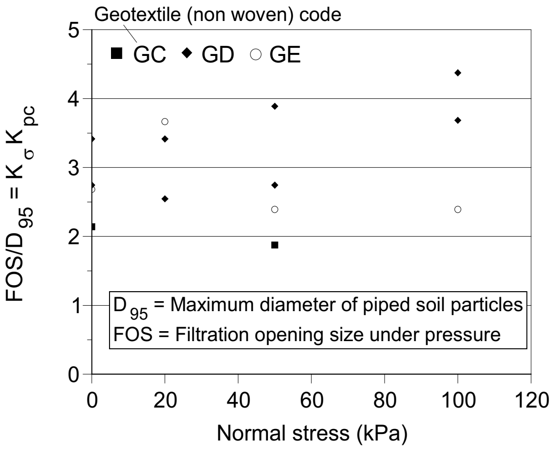


Figure 21. Retention capacity of partially clogged geotextiles under pressure.

Several authors have addressed the problem of filters in internally unstable base soils. Giroud (1982) suggests that the filtration opening size of the geotextile be compared to the largest diameter of the finer fraction of the soil. Giroud also criticises the use of criteria requiring O_{95} very close to or much smaller than D_{85} of the soil due to the possibility of piping or geotextile clogging, respectively. Lafleur (1999) suggests the use of an indicative diameter soil equal to D_{30} for gap graded internally unstable soils and for internally unstable soils with gradation curves concave upwards (risk of piping of fines) in his retention criterion for cohesionless soils.

Bhatia and Huang (1995) observed that the formation of a bridging network (vaults) on the geotextile in internally unstable soils was a function of the hydraulic gradient and suggest that soils with values of coefficient of curvature above 7 should be considered as internally unstable and below this value internally stable. The authors stated that the filter criteria developed for internally stable soils are not applicable to internally unstable soils and proposed a specific filter criterion for the latter (see Table 1).

Lawson (1986) presented a criterion developed for two dominant residual soil types from Hong Kong (a completely decomposed granite soil and a completely decomposed volcanic soil). The criterion was based on long term filtration tests, where it was observed that the geotextiles performed well. A chart allows the determination of the relation between geotextile opening size and soil particle diameter (Table 1) to be used. Gardoni (1995) also performed long term filtration tests on residual soils of the region of the city of Brasilia, Brazil, observing a good performance of the non woven geotextiles tested.

A problematic situation for the design of filters may occur in residual soils, where larger grains can be composed of clusters of finer soil particles. Figure 22 presents a typical view of this type of problem (Gardoni and Palmeira, 1998). As filter criteria for geotextiles are based on soil particle dimensions, the way these dimensions are obtained play a fundamental role in the selection of the geotextile filter to be used in the project. Therefore, the use of dispersant in grain size analysis may yield a grain size distribution curve with a much greater amount of fines than that obtained without the use of dispersant and, depending on the grain size distribution curve used in design, a geotextile product can be accepted or rejected as filter (Gardoni and Palmeira, 1998). Figure 23 shows typical grain size distribution curves from grain size analyses with and without the use of dispersant for a residual soil (Rezende, 1999). The dilemma for the designers is then the choice of the grain size distribution curve to use in design in these situations.

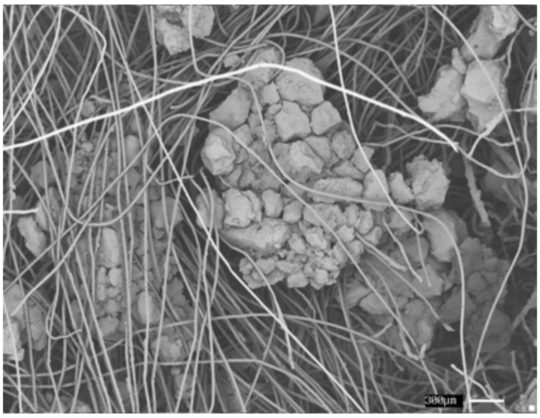


Figure 22. Clusters of soil particles entrapped in the geotextile (Gardoni and Palmeira, 1998).

Figure 24 (a) and (b) summarises the results of an exercise on the predictions of ranges of O_{95} or O_{90}, from retention criteria listed in Table 1, based on grain size data shown in Figure 23. It can be seen that a rather wide range of values of geotextile filtration opening sizes is possible for both types of tests with smaller difference between maximum allowable geotextile opening size for the calculations based on data from the gradation curve using dispersant (finer soil). Four retention criteria (20% of the criteria employed) were not applicable for the data from the grain size curve without the use of dispersant, while this number doubled (40% of the criteria employed) for the test results with the use of dispersant. It is clear that for the former case most criteria allow more open geotextiles to be used than the latter. For criteria using O_{50}, rather than O_{90} or O_{95}, for the geotextile the maximum acceptable O_{50} value from on criterion can be up to 5 times that value for another criterion. It is also worth mentioning that Figure 24 (a) shows that 6 out of the 17 criteria employed (35%) would not accept the use of geotextiles with filtration opening sizes in the range 0.053 to 0.145 mm. The filtration opening sizes of a great number of commercially available non woven geotextiles products fall within this range. Nevertheless, the large number of applications of geotextile filters within this range of opening sizes for similar base soils in tropical regions in conjunction with the very few reported filter failures, is a strong indication of good filter performance even under these conditions. Similar observations of good performance of geotextile filters under severe conditions are reported in Bathia et al. (1991) and in Gardoni and Palmeira (1998).

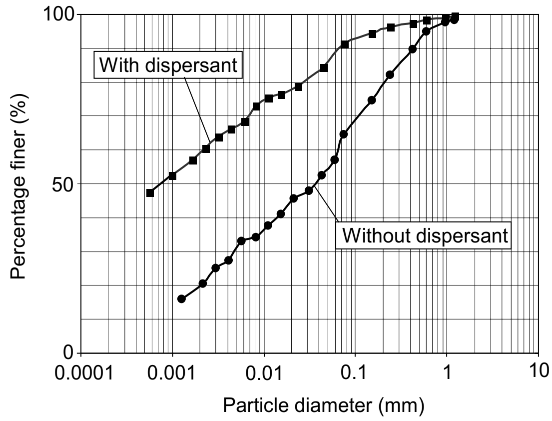


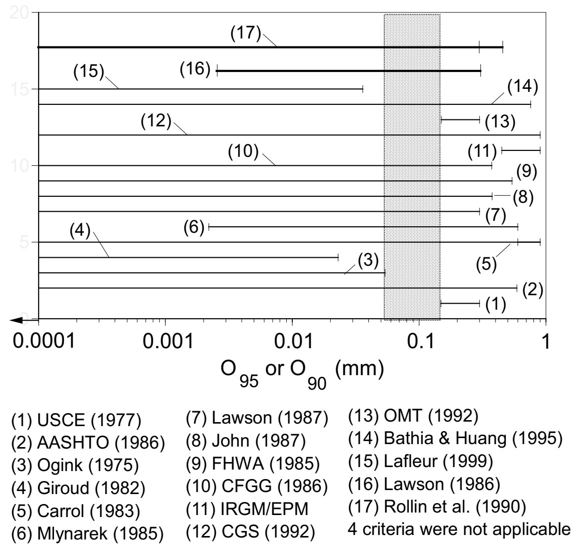
Figure 23. Influence of dispersant on grain size distribution for a residual soil (Rezende, 1999).

Some of the deviations of maximum geotextile opening sizes from different retention criteria commented above may be associated to test conditions, as commented in a previous section of this paper. The dimension of largest soil particle capable of piping through the geotextile in a test will depend on the type of test employed, soil and geotextile characteristics, hydraulic gradient used and how the soil is placed (or compacted) on top of the geotextile specimen. The latter may yield to different levels of soil particle intrusions in the geotextile, which will affect the size of the piped particles.

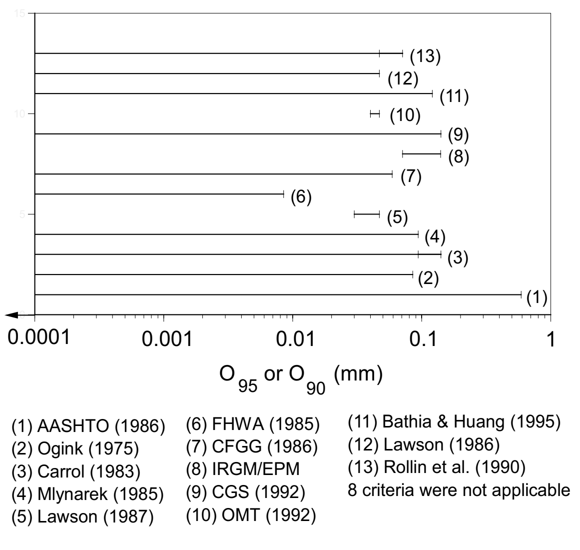
Another question is how strong the clusters of soil particles are to flow under long term conditions when cluster dimensions are considered for filter design. The possibility of base soil instability or filter clogging caused by the dispersion of the fine particles of the clusters cannot be ruled out in long term applications.

Another complex phenomena which can cause filter clogging is ochre formation, as a result of microorganisms activity and iron precipitation. Infanti and Kanji (1974), Ferreira (1978) and Lindquist and Bonsegno (1981) reported several cases of ochre formation and eventually filter clogging in granular filters in some Brazilian dams. Ford (1982), Scheurenberg (1982), Van Zanten and Thabet (1982) and Puig et al. (1986) report different levels of the influence of ochre formation on geotextile filters. Figures 25 (a) to (c) show an example of clogging of the geotextile under a gabion revetment in a channel in a erosion control work in Goiania, Brazil. Progressive structural failure of the channel base was caused by the filter clogging (Fig. 25b). The lack of a filter layer along the side wall of the channel (Fig. 25 c), also triggered erosion in that region with the suspended soil particles being carried by water towards the base geotextile filter.

Filter clogging by ochre formation is still a little known phenomenon. Mendonca (2000) performed filtration tests inducing ochre formation in sand and geotextile filters. Two woven and one non woven geotextile (200 g/m^2) were tested, with filtration opening sizes ranging from 0.13 mm to 0.8 mm. The granular filter was a coarse to medium sand with D_{50} = 0.92 mm, D_{10} = 0.31 mm , D_{85} = 1.58 mm and coefficient of uniformity equal to 3.4. Geotextile permeability reduction in long term (up to 1600 hours) filtration tests ranged from 2.4 to 45.3 times, but the general behaviour of the system was not affected by those reductions in the permeability due to the low permeability of the base soil (or the order of 1000 to 10000 times smaller than those of the filters). Although presenting a smaller permeability reduction the greater retention capacity of the sand, compared to those of the geotextiles, brings concerns with respect to long term performance of the granular filter. It is clear that chemical and bacterial clogging of geotextile is certainly a field wide open to research in the coming years.



(a) Without dispersant



(b) With dispersant

Figure 24. Predictions from retention criteria for base soil of Figure 23.

(a) Collapse of the channel.

(b) Clogged geotextile.

(c) Erosion at the channel side wall due to lack of filter.

Figure 25. Geotextile clogging caused by ochre formation.

CONCLUSIONS

Geotextiles have shown a overwhelming success as drains and filters in civil and environmental engineering applications. Its versatility and range of applications has made it increasingly gain ground from traditional granular materials. During the last decades a large number of important contributions on the study of these materials in filtration and drainage has been published and design criteria established. Quite understandable, these criteria are conservative in nature because of the consequences of a filter malfunction in an important engineering work. However, the synthesis of the knowledge of the performance of geotextiles in drainage and filtration put forward new challenges regarding the optimisation of design procedures and extension of the use of geotextiles and geocomposites to major engineering projects. This paper tried to approach these subjects based on experimental and field observations on the performance of geotextile filters.

A large number of retaining criteria for geotextiles are available, most of them heavily based on empiricism. A more scientifically based procedure for retention criteria should be pursued. In this sense, the use of probabilistic approaches, introduced to the study of granular filters decades ago (Silveira, 1965), should be extended to geotextiles, taking into account the influence of other factors such as stress level and presence of entrapped soil particles. Faure et al. (1989), Elsharief and Lovell (1996) and Urashima and Vidal (1998) have used probabilistic analyses to the study of retention capacity of virgin geotextiles. Research results have demonstrated that the retention capacity of geotextiles can increase significantly due to these factors. The migration of base soil particles through granular filters has been recently addressed by Locke (2001) with the use of probabilistic and experimental techniques. A simple solution presented by Giroud (1996) has proved encouraging to account for the influence of stress level on filtration opening sizes of non woven geotextiles.

The advance of design methodologies requires the knowledge of some important, though basic, physical characteristics of geotextiles and their measurement under more realistic conditions. As the pore space in the geotextile under in service conditions is different from that under unconfined and virgin conditions, the evaluation of the possibility of geotextile clogging has also to be re-addressed accordingly, particularly regarding long term applications, filters under critical environmental conditions or reverse or unsteady flow conditions.

Rejection of a geotextile filter under some critical conditions should be a result of a more comprehensive analysis of the problem and design tools for that are currently available for routine works in geotechnical engineering. That may be the case of problems in which values of gradient ratios are greater than 3 or geotextile permeability reductions are expected. Sound engineering analyses and judgement should be exercised in these situations.

Some situations where filters can fail to fulfil their role in geotechnical engineering structures are yet to be properly addressed in current design practice. These are the cases of long term performance and chemical and biological clogging of filters, that may take place in traditional geotechnical engineering works and, in particular, in geoenvironmental applications of these materials. Geotechnical engineers tend to feel much more at easy when physical mechanisms play the major (or preferably the only) role in a project. However, the multi-disciplinary nature of the study of chemical and biological clogging of geotextiles will require the contributions from professional with different, but indispensable, complementary backgrounds, such as biologists, statisticians, chemists, polymer engineers, etc for a better understanding of these phenomena.

Although as mature filter materials as they may be considered now, the progress in the use of geotextiles as filters and the extension of their use to major civil engineering projects will require the development of more realistic design and testing methods as well as a more comprehensive study of case histories, particularly those earlier applications of geotextile filters.

ACKNOWLEDGEMENTS

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