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Hydraulic applications of geosynthetics

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ABSTRACT: After more than 30 years of successful experience, geosynthetics are very well established for many applications in hydraulic engineering (dams, canals, basins, erosion protection, scour countermeasures, coastal protection, …) and the possible uses are growing continuously. There is a great variety in materials as well as in products and application areas. Geosynthetics in hydraulic applications cover mainly the following functions: separation, filtration, draining and lining. The manufactured products comprise wovens and nonwovens made from natural and synthetic fibres, bituminous and synthetic geomembranes, all kinds of composites manufactured according to the specific requirements. For all these applications, experience has been gained, tests have been developed, regulations and recommendations have been written. The paper will give insight into the different applications and the corresponding design theories, material testing, installation procedures, regulations and long-term experience. Examples are given from all over the world with the aim that this information leads to a unified approach for the application of geosynthetics in hydraulic engineering.

1 INTRODUCTION

"Hydraulic application" covers maybe the most widespread range of using geosynthetics. The search for a clear definition of that term however reveals to be rather difficult. At first, one will consider all applications that are related to surface water, i.e. offshore and coastal works, rivers and their estuaries, lakes, reservoirs and canals. At a second glance, typical hydraulic structures will come to one's mind, where geosynthetics are involved during construction or operation, e.g. drilling rigs and pipelines, coastal protection structures, river training measures, hydropower stations, locks and weirs, harbours, dams and embankments, dikes and levees, irrigation facilities. But also works related to subsurface water incorporate geosynthetics, keywords are "groundwater" and "water resources". Another wide field of the use of geosynthetics is any interaction of land and water, e.g. during floods, in wetlands and for land reclamation.

Trying to condense the uses of geosynthetics in hydraulic application, one can define hydraulic application of geosynthetics as the interaction of fabric, soil and water. Due to that interaction, theoretical and practical aspects have to be discussed in the three domains of geotechnics, hydraulics and hydrology. Loads have to be considered that are unequalled in other fields of application e.g. sudden excess pore water pressure, unsteady, turbulent, reversing flow or wave impact.

Since water is the governing element, maybe the best way to group hydraulic applications of geosynthetics is to look at the function of the geotextile in relation to water:

- geosynthetics to control the water (filters, erosion protection)
- geosynthetics to remove water (drains)
- geosynthetics to block water (impervious linings, flood barriers etc.)

Any further applications of geosynthetics in combination with hydraulic works and structures, e.g. geosynthetics to increase the strength of such structures (bank reinforcement, wall elements, etc) are as well rather important, but will not be treated here. Such applications are similar to those without the major influence of water.

The major issues mentioned above will be treated by authors of all the continents of our earth to underline the worldwide beneficial use of geosynthetics in hydraulic applications. The main topic is the control of the interaction of soil and water, which has become one of the major problems of mankind, as (unfortunately) can be seen much too often from the news casting. Geosynthetics can help to build new structures with sufficient strength to resist the thread of erosion and flooding including a sufficient factor of safety. They are able to improve the resistance of existing structures against the water, they can provide help in hazard situations and they can reduce the time necessary to rebuild structures after water induced hazards like flooding, landslide etc. And they can support the beneficial use of water in irrigation projects, navigable waterways, water power plants etc. It is impossible to cover all aspects of hydraulic application in a single paper. Therefore certain main aspects are chosen to be discussed.

2 GEOSYNTHETIC FILTERS

The primary goal of filters is to retain particles of a base soil without altering the drainage capacity of the system. Success of a filter however, involves implicitly that the equilibrium between the drag of the flowing water on the base particles and the stereometric hindrance of the filter pores, is attained after minimal washout. The overall performance of a drainage system is related to the internal movement of particles that can produce two detrimental effects:

- formation of sinkholes near the base-filter interface, creating uneven subsidence at the surface. This is particularly damageable in road structures where surface uniformity is the guiding rule;
- infilling and clogging of the downstream water conveyance system, reducing thereby its capacity to evacuate the design flow rate.

When geosynthetic filters were first proposed as alternative to granular filters, engineer's mind was immediately concerned about their thinness when compared to that of granular layers. Extensive research has been devoted to establish constriction diameters for these new materials. It disclosed that following an initial period of rearrangement of particles, geotextiles act as catalyst in the formation of a stable soil filter interface from the in situ parent soil (Lawson, 1982). Equilibrium is promoted between the drag of flowing water and the retention of the larger particles by the geotextile, resulting in the system permeability to remain constant with time. The development of filter criteria until the year 2000 is comprehensively given by Giroud (2000).

2.1 *Range of Practical Applications*

The range of hydraulic applications of geosynthetics is very broad and they are used in many civil engineering works: edge drains in road structures, first or second line of defence against internal erosion and piping in earth dams and dikes, erosion protection, silt fences, for example. Unlike reinforcement applications, filtration and drainage of soils is sensitive to the soils properties having the most important spatial variability: particles gradation and permeability cover at least six orders of magnitude in range i.e. from 10^{-3} to 10^{3} mm and from 10^{-10} to 10^{-3} m/s, respectively.

Most of the filter criteria however, have been developed and proposed from the interpretation of compatibility tests results obtained for "normal" conditions: uniform soils and static loading. These conditions are seldom encountered in practice since many applications involve either unstable soils or dynamic loads. Depending upon the geological context, problematic soils are those that can be eroded easily such as silts or well graded soils containing appreciable amounts of silt sized cohesionless particles. Difficult loading situations are cyclic or dynamic loadings from the traffic encountered in road drainage systems or from waves, tides and flow current in erosion protection works.

The interaction between a base soil and its filter is rather very complex. On one side, the rearrangement of particles near the interface is not taken into account properly in the current filter criteria and on the other side, the filtration opening size is characterized by testing procedures that carry a certain margin of error.

Retention capability of a given base soil/geotextile filter system can be formulated in terms of a retention ratio defined in general terms, as

$$R_R = \frac{\text{filter opening size } O_F}{\text{indicative base size } D_I} \tag{2.1}$$

By definition, retention is attained if this ratio is less than or equal to unity. Christopher and Fischer (1992) and later on, Gardoni and Palmeira (2002) report numerous retention criteria from the literature and one is struck by the diversity of the values of the sizes used for O_F (O_{95}, O_{90}, O_{50}, O_{15}) as well as for D_I (D_{90}, D_{85}, D_{50}, D_{15}) (the subscripts refer to the percent finer). This diversity reflects some level of subjectivity and do not rely on physical phenomena. It may explain the lack of universality of the existing retention criteria. Almost all of them do not take into account the coefficient of uniformity C_u of the base and they refer to the size of its larger particles for D_I. This misconception can lead to unconservative designs resulting in substantial washout of the particles finer than D_{85}, especially when gap-graded and/or broadly graded cohesionless soils are involved. This latter definition applies to soils having coefficients of uniformity C_u larger than 8 and coefficients of curvature C_c larger than 3. Note that the U.S.C.S. would classify these as poorly graded, which is obviously not the case.

2.2 *Severe conditions*

Filters are used in more and more situations where conditions are different from those assumed when interpreting compatibility tests. For uniform soils, the classical Terzaghi filter retention criterion based on $D_I = D_{85}$ for the base soil and on $O_F = D_{15}/4$ for the granular filter or O_F = AOS or FOS for geotextiles, has proved adequate. AOS refers to dry sieving as per ASTM 1995b) and FOS, to hydrodynamic sieving (CFGG 1986). Filtration problems reported in the literature however, involve fine cohesionless soils, some of these being uniform and some being broadly graded silty soils. Giroud (1982) was the first to introduce a retention criterion that takes into account the coefficient of uniformity C_u. It was recognised at that time that soils were potentially unstable when $C_u > 20$. Internal instability occurs when the finer particles of a soil can move within the skeleton formed by their coarser particles upon flow of water. Kenney and Lau (1985) have developed a graphical procedure to evaluate the potential for internal instability of cohesionless soils based on the shape of the gradation curve.

The base filter interaction is essentially related to the internal rearrangement of the base soil in the interface zone. With uniform particles, arching effect at the pore entrance can signify that failure is attained for R_R values as high as 4 (Gourc and Faure, 1990). When the particles size distribution is broad, the self-filtration mechanism in the interface zone is complex. Lafleur (1999) has proposed to incorporate this interaction by evaluating the retention of soils on the basis of a retention ratio R_R defined with values for D_I that take into account the internal stability of the base soil. These are a function of the shape of the gradation curve classified according to the three general types summarized on Figure 2.1:

- *rectilinear*: a substantial part of gradation is linear in the middle size range ($15\% < F < 85\%$) where F is percent finer;
- *gap-graded*: some intermediate size particles are missing and the gradation curve shows a flat part when F is less than 30%;
- *concave upward*: this curve is quite similar to that of the gap-graded soil but the transition between the coarse particles and the finer, is smoother.

The arrows on the figure give the corresponding indicative sizes D_I values as assessed from the results of compatibility tests: they are equal to D_{50}, D_G and D_{30} respectively and D_G is the minimum gap size.

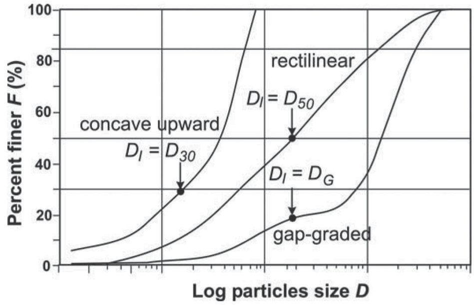

Figure 2.1. Classification of gradation curves of broadly graded soils.

The smaller particles have the most significant influence on retention of broadly graded soils and a filter has to be selected according to their size.

2.2.1 *Occurrence of internally unstable soils*

Geotextiles are used extensively in two classes of applications where broadly graded soils are often encountered: wrapping of drainage systems for foundation under road pavements and filters to prevent internal erosion in water retaining structures. The internal stability of the protected soils has to be evaluated to ensure that no excessive erosion is to take place.

2.2.2 *Road pavements*

In order to minimize the detrimental effect of water in road foundation aggregates and subgrades, geotextiles are used to wrap plastic or granular cores in edge drains. The geotextiles are in contact with either the subgrade soils that covers a wide range of gradations or the foundation aggregates. These latter are constituted of gravel and sand size particles containing traces of fines defined within acceptable limits. Figure 2.2 gives the boundaries defined by MTQ (1995), (a) for base and (b) for subbase materials. Although the limits for the former are relatively narrow, the subbase can be any sandy and gravelly material, provided it contains less than 9% fines. Road aggregates often have concave upward gradations and in some instances, these later fines can move inside the coarse skeleton and be blocked at the filter interface (blinding of the base) or washed through it (piping) and fill the downstream drainage pipe.

A so-called screen test program was undertaken to evaluate the internal stability of aggregates of which gradations are within the specifications of the MTQ (Lafleur and Savard, 2004). Reconstituted broadly graded bases (gradations given on Fig. 2.2) are placed in a permeameter having at its bottom a standard 100 mm diameter sieve with opening sizes O_F varying between 0.15 and 4.75 mm. Water is circulated downward under a gradient of 10 during 150 minutes and piezometer readings at 65, 105, 155 and 205 mm

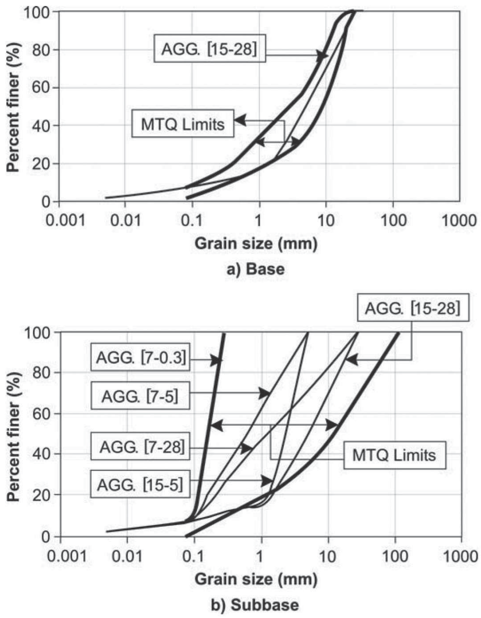

Figure 2.2. MTQ specifications for road aggregates and materials tested for internal stability (Lafleur & Savard, 2004).

from the interface are made periodically. At the end of the test, the amount of piped particles per unit area M_P is recorded. Aggregates 15-5 and 15-28 were unstable as per piezometer readings. The others (7-0.3, 7-5 and 7-28) were stable according to test results and confirmed the validity of the Kenney and Lau's method (1985). The Figure 2.3 gives a log-log plot of the measured M_P-values as a function of the retention ratio R_R as defined by Lafleur (1999). Results obtained from identical test programs (Lafleur et al., 1989) and (Lafleur, 1998) are included and they give the same trend: gradual increase in M_P with R_R. In all

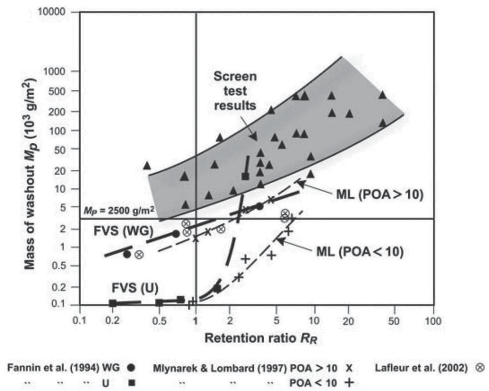

Figure 2.3. Mass of washout M_P versus Retention Ratio R_R.

cases however, M_P is much larger than the threshold value of 2500 g/m^2 proposed by Lafleur et al. (1989); this corresponds to the maximum M_P-value beyond which no equilibrium is attained i.e. continuous piping occurs. These high M_P-values can be explained by the fact that the thickness of the mesh is minimal and the Percent Opening Area, large; POA is the area of the openings compared to the total specimen area.

2.2.3 *Embankment dams*

In zoned earth and rockfill dams, the impervious core is often made of compacted cohesionless moraine that can be internally unstable, given the broadness of its grain size distribution. Internal erosion and piping of core materials have been highlighted as the main geotechnical cause of dam failure (Foster et al., 2000). If the retention ratio is larger than one, continuous internal erosion develops and sinkholes gradually enlarge due to progressive piping. At the opposite, if R_R is much less than one, the smaller particles of the unstable material are moved by water flow through the core mass and concentrate at the filter interface to form a cake because O_F is too small. This local reduction in permeability results in an increase in porewater pressures on the downstream slope of the dam that can result in a decrease of the factor of safety against sliding.

In order to check the internal stability of the broadly graded cohesionless moraines, a compatibility tests program was undertaken (Lafleur and Nguyen, 2005). The tests were made in a 150 mm diameter permeameter on 340 mm high moraine samples compacted above and upstream of a filter paper. During the downward flow under a gradient of 10, pore pressures were recorded by four piezometers installed along the walls at 85 mm intervals. The tests were ended when pore pressures and flow rates remained constant with time; the tests lasted between 0.5 and 51 days. The gradation curves of the tested soils are given on Fig. 2.4. The results disclosed that the moraines with a percent of fines (< 80 µm) P_f less than 12%, were internally unstable and caking was observed near the filter paper interface. Blinding was further assessed from the first piezometer readings at 85 mm from the filter, that has shown a marked increase in hydraulic gradients in this interface zone. However, for the moraines with P_f larger than 12%, normal behaviour was observed i.e. linear head losses versus depth. In a second series of downward flow tests in the same permeameter cell, a 85 mm thick granular filters was placed under the moraine. Unstable moraines with P_f of 6 and 12% were tested against a coarse filter (C on Fig. 2.4) and a fine filter F. When R_R was less than one with filter F (R_R = 0.3), the head losses were linear, M_P was small developed. With the coarse filter C, R_R was larger than one (R_R = 2.2), piping was observed and M_P was substantially higher than the above mentioned threshold value of 2500 g/

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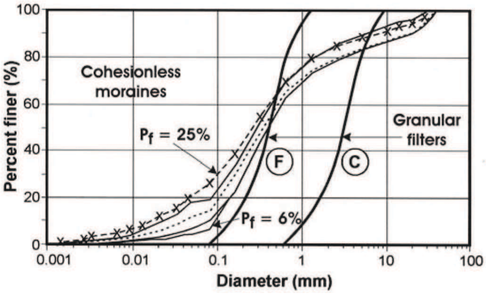


Figure 2.4. Gradations of cohesionless moraines tested for internal stability (Lafleur & Nguyen, 2005).

m^2. For the particular case of moraines with appreciable contents of fines, the validity of the Kenney and Lau's (1985) approach was not supported by these results since it indicated stability in all cases.

2.2.4 *Amount of Piped Particles*

These two examples of applications illustrate that the internal movement of particles involved in filtration situations can influence the overall performance of a system. The amount of piped particles is seldom measured in soil/geotextile compatibility tests. The Gradient Ratio test defined in Section 2.5 for example, does not require such measurements. Fannin et al. (1994) however, give results of modified Gradient Ratio tests with measures of M_P performed on uniform (FVS-U on Fig. 2.3) and well graded soils (FVS-WG, defined by $3 < C_u < 7$) with nonwoven geotextiles. The smoothed curves of the results are lower than those of the screen tests. This reflects the influence of the filtration length which is higher for nonwoven geotextiles. For the WG soils ($D_I = D_{85}$, since $C_u < 6$), the M_P - R_R relationship is linear and for $R_R = 1$, $M_P \approx 2500$ g/m^2. For the U soils with $C_u \approx 1$, there is a sharp bend at $R_R \approx 1$ and for $R_R \leq 1$, M_P is minimal. Gradient Ratio tests results obtained from Lafleur et al. (2002) on nonwovens and a broadly graded soil confirm the trends observed by Fannin et al. (1994) for WG soils on Fig. 2.3.

Also plotted on Fig. 2.3 are the compatibility test results of Mlynarek and Lombard (1997) obtained on woven geotextiles. The influence of the percent open area POA is obvious. At smaller values (POA < 10%) and for $R_R = 1$, M_P is equal to 120 g/m^2 and compares to those for nonwoven. The M_P-values for POA > 10% are consistent with those of the screen tests for which 62% < POA < 75%; at $R_R = 1$, M_P is equal to 1900 g/m^2.

2.3 *State of the art of design rules*

The fourth edition of the Canadian Foundation Engineering Manual (CFEM) is currently under press (Canadian Geotechnical Society, 2006) and since its first edition some 25 years ago, it is reviewed periodically to account for new developments in Geotechnical Engineering. It contains acceptable design guidelines for the solution of routine foundation engineering problems as based on sound engineering practice in Canada. One of the chapters of the Manual covers design using geosynthetics and the filter rules have been modified to reflect the most recent improvements in this topic.

2.3.1 *Retention*

Although the formulation is different in CFEM, the retention ratio described in Eq. 2.1 must be equal to or less than one. No distinction however, is made between AOS and FOS for O_F when greater than 150 µm. For lower values, FOS is recommended. The influence of the coefficient of uniformity is recognised since the indicative size D_I is defined as a function of C_u:

for $C_u < 2$, $D_I = D_{85}$, (2.2)
for $2 < C_u < 4$, $D_I = 2\ D_{85}/C_u$
for $4 < C_u < 8$, $D_I = 8\ D_{85}/C_u$
for $C_u > 8$, $D_I = D_{50}$ for linearly graded soils
$D_I = D_{30}$ for soils with concave upward gradation curves
$D_I = D_G$ for gap-graded soils, where D_G is the minimum gap size

These latter classes of gradations have been discussed earlier and they correspond to those on Fig. 2.1.

2.3.2 *Clogging*

This is probably the most misunderstood phenomenon when dealing with filtration and this comes probably from the confusion in the use of terms. Christopher and Fischer (1992) define it as *the result of fine particles penetrating into the geotextile and blocking off pore channels or caking on the upstream side of the geotextile thereby reducing its permeability.* This definition however, does not refer to the causes of the phenomenon that can be quite different. In the first case, it would be more accurate to say internal clogging i.e. the particles that block off pore channels come either from solids in suspension, which is the condition most likely to promote complete filling of the geotextile pores, or from a structured parent soil which has left some finer particles in the process of self-filtration or piping. In the second case when referring to caking, the phenomenon is essentially related to the internal instability of the base material itself and this affects mainly the permeability of the base that is reduced, although there may be some interpenetration in the geotextile structure, especially if it is nonwoven. With woven, interpenetration is unlikely and the term clogging is totally inappropriate. For the second case of caking, we should talk about external clogging or blinding. Since the Kenney and Lau (1985) method provides a fair evaluation of the internal stability of a cohesionless soil, external clogging can be predicted with some level of accuracy. The Gradient Ratio test is also a recommendable performance test to evaluate the potential for external clogging or blinding.

The CFEM has added a prescription stating a minimal opening size stipulating that

$$O_F > 3 \cdot D_{15} \text{ of the base} \tag{2.3}$$

for soils with $C_u > 3$ and low hydraulic gradients under steady flow conditions. Internal clogging is also prevented by requiring that for retention of fines, $k_n > 10\ k_s$, where k_n is the normal geotextile permeability and k_s the permeability of the base soil.

2.3.3 *Cyclic loading*

For applications with a dynamic, pulsating or cyclic flow, different soil retention criteria must be used because the loading on the particles is more aggressive than a static, continuous flow. Dynamic flow conditions may occur in pavement edge drain applications. Geotextiles placed below slope protection or embankment riprap layers in tidal areas or other shoreline applications can be subjected to cyclic water flows. Fannin and Pishe (2001) have performed Gradient Ratio tests under cyclic flow applications on two uniform soils and 3 different geotextiles. They have shown that for R_R varying between 0.6 and 1.5, the M_P values were much less than the above mentioned value of 2500 g/m^2. Although the following criteria are conservative, the CFEM recommends that where the pulsating flow is large, the geotextile should be sufficiently open to prevent blow up and should be weighted down. The opening size of the geotextile should satisfy the lesser of:

$$O_F < 0.5 \cdot D_{85} \text{ of the base}$$

$$\text{or} \tag{2.4}$$

$$O_F < 0.3 \text{ mm}$$

Nevertheless, these criteria should not be used for pulsating loads on horizontal geotextile layers placed at the base of highway and railway granular base materials where the flows are small and cyclic loads large. For these conditions, the relationships of eqs. 2.2 should be used.

2.3.4 *Silty and clayey soils*

For static conditions and for soils with more than 50% passing the 80 µm sieve, the CFEM indicates that the maximum O_F-value should not exceed 0.3 mm. If the base soil is composed completely of passing the 80 µm sieve, provisions should be inspired by the design of granular filters that stipulates that $D_{15} < 0.5$ mm for clayey soils. Following the same logics, the O_F-value of a geotextile filter should be less than $D_{15}/4$ or 125 µm. When dynamic, pulsating or cyclic flow is involved, the geotextile must be separated from these soils by a concrete sand continuously graded down to 1% finer than the 80 µm-sieve. Distinction should be made however, if the clayey soil is intact or remoulded. In this latter case, the cohesion keeping the clay particles in flocs, is destroyed and particles are more likely to move. The ability or the likeliness of a cohesive soil to sustain open cracks should also be considered. The velocity of the seeping water along cracks is higher and increases the possibility that particles or flocs are eroded.

2.4 *Conclusions*

It should be kept in mind that the first goal of a good filter is to retain particles. The retention ratio R_R between the opening size of a filter O_F and the indicative size of the base soil D_I, must be kept lower than or equal to unity for filter design. When uniform soils for which $D_I = D_{85}$, are filtered, the mechanisms are relatively simple and the existing retention rules have proved adequate: for $R_R > 1$, piping occurs. With broadly (and widely) graded soils for which D_I is related to the shape of the gradation curve, the rearrangement of particles is complex and their retention necessarily involve some washout to attain equilibrium. The amount of washout M_P is related mainly to the grain size distribution of the soil to be filtered: M_P increases proportionally to R_R but equilibrium is attained for $R_R \leq 1$. It has also been showed from test evidence that the geotextile structure has some effect in promoting equilibrium. Larger Percent Open Area for woven geotextile means larger amount of washout. Finally, the test results have demonstrated that the internal stability of filtered soil plays a major role in a physical phenomenon that is often misunderstood: clogging. Further research is still needed to evaluate it properly and to give objective and clear cut rules for design purposes.

3 FILTRATION TESTS

3.1 *Gradient ratio test under pressure*

Gradient ratio tests have been carried out to investigate soil-geotextile compatibility. These tests are particularly important in situations where the geotextile will work under severe and/or critical conditions. Figure 3.1 presents schematically the conventional type of equipment used.

Variations of the conventional equipment have been presented (Palmeira and Fannin 2002) to allow the investigation of other aspects of soil-geotextile compatibility, such as influence of stress level, hydraulic heads closer to the geotextile layer and influence of reverse flow regimes, for instance. The gradient ratio is defined as

$$\text{GR} = \frac{i_{sg}}{i_s} \tag{3.1}$$

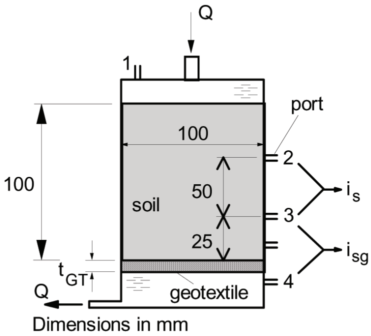


Figure 3.1. Schematic view of the gradient ratio test equipment.

Where i_{sg} is the hydraulic gradient in a region comprising a certain amount of soil and the geotextile and i_s is the hydraulic gradient in the soil, a certain distance from the geotextile (in the region between 25 mm and 75 mm above the geotextile face).

According to ASTM (1995a), the value of i_{sg} should be calculated using a port 25 mm above the geotextile plus the geotextile thickness for the calculation of GR (GR_{ASTM}). Different definitions of GR can be found in the literature using ports at lower elevations for the calculation of i_{sg} aiming to capture the mechanisms taking place closer to the geotextile filter and to help the interpretation of the test results (Fannin et al. 1996, Palmeira et al. 1996, Gardoni 2000, Palmeira and Fannin 2002). However, the definition of GR by the ASTM (1995a) is the approach most commonly used for the acceptance or rejection of a candidate geotextile filter.

The value of GR obtained by Equation 3.1 can be affected by several factors. In non homogeneous undisturbed soil specimens or in internally unstable soil specimens the interpretation of the values of GR obtained can be very complex because of the influence of soil heterogeneities or soil particles migration on the value of i_{sg} or i_s, or on both. The movement of particles in reconstituted specimens of internally unstable or gap graded soils can occur already during sample preparation by vibration, which yields to segregation. Thus, when flow of water takes place the soil specimen may already be in a non homogeneous condition, which will affect the values of i_{sg} and i_s measured. Long-term filtration tests would be recommended in these situations with proper analysis of the stability of the system and flow rate values obtained.

The measurement of water heads very close to the geotextile may be useful for research purposes and to provide additional information on the system behaviour. However, readings of water heads along the permeameter walls very close to the geotextile surface (3 mm above it, for instance) may be very sensitive to test conditions and to factors such as geotextile compression during the test, for instance. Therefore, it is always useful to have more than one port at each elevation for readings of water heads.

There is a growing interest on the use of geotextiles and geocomposites for drainage in large earth works, such as large embankments or dams. In such works the filter layer will be subjected to stress levels well above those of works were geosynthetics have been traditionally used. High normal stresses on filter systems can also occur in tailing dams of moderate heights, in case of large tailings unit weights. Thus, concerns regarding the behaviour of synthetic filters under high stress levels, durability and effects of mechanical damages do exist and should be considered seriously in such applications.

Palmeira et al. (2005) have conducted gradient ratio tests on soil-geotextile systems under normal stresses up to 2000 kPa. Non woven geotextiles were used in the tests and the soil samples comprised glass beads, residual soils and mining wastes. Tables 3.1 and 3.2 summarise the main properties of the materials tested. The results obtained showed that the value of GR increases with the stress level, particularly those where the values of i_{sg} are calculated based on readings of heads close to the geotextile. Figure 3.2 shows some of the results obtained for the values gradient ratios at a normal stress of 2000 kPa. Note that different definitions of the gradient ratio were investigated depending on the elevation of the port used to calculate i_{sg} in Equation 3.1 (elevations of 3 mm, 8 mm and 25 mm above the geotextile). The results in Figure 3.2 show that the values of GR_{ASTM} varied between 1 and 2 for the mining wastes tested while values between 2.8 and 4.6 were obtained in the tests with residual soils. Some of these values are above the usual limit for GR_{ASTM} of 3. However, one could argue whether the geotextile should be rejected if the long term behaviour of the system is satisfactory. This highlights the importance of long term tests in such cases. As expected, the values of GR for i_{sg} calculated based on hydraulic heads measures in ports 3mm and 8mm above the geotextile were considerably higher than the values of GR_{ASTM}.

Table 3.1. Geotextile characteristics.

Geotextile	t_{GT} [mm]	M_A [g/m^2]	FOS [mm]
G0	1.5	150	0.15
G1	2.3	200	0.13
G2	2.6	300	0.11
G3	3.7	400	0.09
G4	4.5	600	0.06

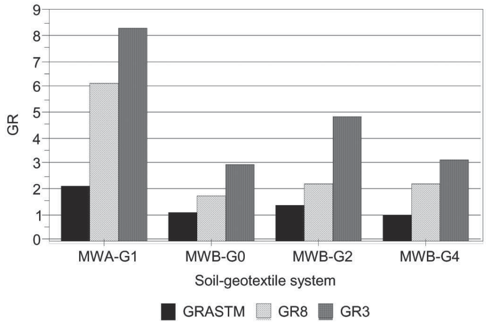
Notes: t_{GT} = geotextile thickness under 2 kPa normal stress; M_A = mass per unit area; FOS = filtration opening size.

Soil particles can intrude the geotextile layer during sample preparation in gradient ratio tests or in the

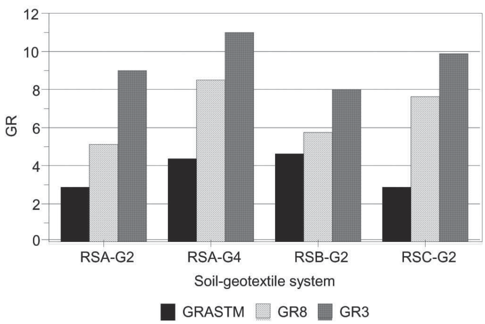
Table 3.2. Soil characteristics.

Soil	D_{10} [mm]	D_{85} [mm]	C_u [–]
RSA	0.015/0.01$^{(2)}$	0.18/0.34$^{(2)}$	8.7/21$^{(2)}$
RSB	–/0.009$^{(2)}$	0.19/0.22$^{(2)}$	–/3.3$^{(2)}$
RSC	0.065/0.028$^{(2)}$	0.20/0.22$^{(2)}$	2.4/6.8$^{(2)}$
MWA	0.07	0.35	2.6
MWB	0.061	0.408	4.0

Notes: (1) D_{10} and D_{85} = diameters of particles for which 10 and 85% of the soil in weight are smaller than that diameter; (2) number on the left is the result of the grain size test with the use of dispersant and number on the right is the result of the test without the use of dispersant (3) C_u = soil coefficient of uniformity (= D_{60}/D_{10}).



(a) Tests on mining wastes



(b) Tests on residual soils

Figure 3.2 Results of GR tests under 2000 kPa normal stress.

field. The possible effects of the impregnation of the geotextile by soil particles on its filtration behaviour were initially pointed out by Masounave et al. (1980) and Heerten (1982). It is particularly interesting that rather large soil particles can be found in the geotextile matrix at the end of GR tests. These particles may have entered the geotextile during vibration for soil densification or being pushed into the geotextile as a consequence of stress level increases. Figure 3.3 shows a large soil particle in the voids of a geotextile after a GR test under pressure (Palmeira et al. 2005).

The intrusion of soil particles in a non woven geotextile (mechanically bonded) layer in the field is

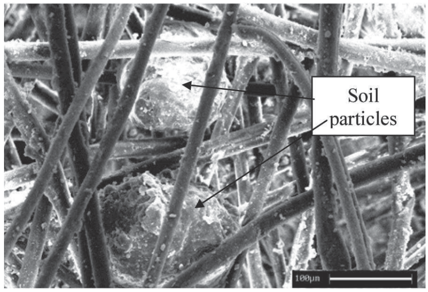


Figure 3.3. Soil particle inside the geotextile at the end of a GR test (Palmeira et al. 2005).

also possible during spreading and compaction of fills on the geotextile, particularly for fine non cohesive soils. Further impregnation of the geotextile may take place due to migration of soil particles towards the geotextile filter. A measure of the impregnation level (λ) of a geotextile can be made based on the ratio between masses of entrapped soil particles and geotextile fibres per unit area. Palmeira and Gardoni (2000) report values of λ between 0.3 and 15 for different types of soils and non woven geotextiles in tests carried out in the laboratory and in the field as well as from geotextile specimens exhumed from real works. Beirigo (2005) obtained values of λ varying between 2 and 10 in specimens of non woven geotextiles exhumed from a drainage system in a tailings dam. The value of λ depends on the dimensions of the geotextile openings, soil characteristics and compaction technique and the largest values of λ are observed for fine grained cohesionless soils. For a value of λ of 8 in a typical unconfined, needle-punched, non woven geotextile made of polyester with porosity of 90% the fraction of the total geotextile void space occupied by the soil particles is of the order of 46%. For lower porosities this value can be significantly greater. The distribution of the soil particles in the geotextile pore space is not necessarily uniform. Uniform distributions of particles in the voids of light weight non woven geotextiles (say, mass per unit area smaller than 300 g/m^2) have been observed, while in heavier geotextiles there is a tendency of concentration of soil particles entrapment in their first top millimetres.

The presence of entrapped particles in the geotextile has two important consequences. First, it increases the retention capacity of the geotextile. Second, it changes the condition for further clogging of the geotextile. Current clogging criteria do not assume impregnation of the geotextile. With the presence of the entrapped particles the available void spaces and constrictions diameters change and the analysis of geotextile clogging becomes more complex. Palmeira et al. (2005) presented measurements of the diameters of soil particles entrapped in non woven geotextiles.

3.2 *Long-term filtration tests*

Long-term filtration tests can provide important information on the acceptance or rejection of a candidate geotextile filter that will to work under severe or critical conditions. Under such conditions the test must be carefully conducted and its duration may reach several weeks to allow reliable conclusions to be taken. Of particular importance are those applications where the geotextile filter will be in contact with undisturbed soils. The gradient ratio test can be employed in such tests. However, the value of GR may be useless if undisturbed soil specimens are to be tested, because the heterogeneities of the soil specimen may influence the values of i_{sg} and i_s, compromising the value of GR obtained by Equation 3.1. Nevertheless, the performance of the soil-geotextile system can be assessed by the variation of flow rate with time and post-test investigations in the soil and in the geotextile specimens.

Gardoni and Palmeira (1998) reported results of long-term filtration tests on systems comprising non woven geotextiles and a residual soil from quartzite. This soil is known in the Federal District, Brazil, to have clogged sand filters of highways. Thus, the Federal District Highway Department was interested in investigating the potentials of the use of geotextile filters as substitutes for granular filters. The main characteristics of the soil and geotextile investigated are presented in Tables 3.3 and 3.4. Figure 3.4 shows the results of a long-term filtration test on the soil-geotextile system with duration of 2700 hours (113 days). After an initial drop the flow rate reached a constant value under a steady state flow regime. In spite of some retention criteria not recommending the use of the geotextile, the test conducted validated its use. The satisfactory performances of highway

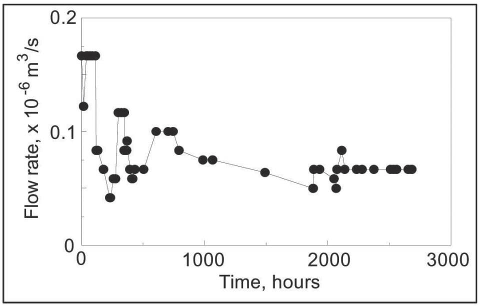


Figure 3.4. Results of long-term filtration tests (Gardoni and Palmeira 1998).

geotextile filters in this residual soil for over 15 years corroborate the laboratory test results. Exhumed specimens of geotextiles of such filters were subjected to normal permeability tests and the permeability coefficients of these specimens were of the order of 25 times the soil permeability coefficient, which satisfies current permeability criteria such as the ones presented by Carroll (1983), Giroud (1982) and Christopher and Holtz (1985), for instance.

Table 3.3. Soil properties.

Natural moisture content, %	11.4
Unit weight (in situ) [kN/m^3]	22.6
Void ratio	0.33
Density of the soil particles	2.71
Permeability coefficient [cm/s]	80×10^{-4}
D_{85} [mm][(1)]	0.22
D_{10} [mm]	0.0086/0.08[(2)]
C_u	15.1/1.63[(2)]

Notes: (1) Symbols as in Table 3.2; (2) number on the left is the result of the grain size test with the use of dispersant and number on the right is the result of the test without the use of dispersant.

Table 3.4. Geotextile characteristics.

FOS [mm]	0.130
AOS [mm]	0.12–0.21
t_{GT} [mm]	2.2
k_n [cm/s]	0.55

Notes: FOS, AOS as described in Section 2.2. t_{GT} = geo-textile thickness; k_n = geotextile permeability coefficient normal to its plane (ASTM 1995c).

3.3 *Drainage characteristics under confinement*

Geotextiles are compressible draining media and stress level can significantly influence their drainage capacity. Pioneer works by Gourc (1982), Gourc et al. (1982) and Rollin et al. (1982), for instance, have identified the loss of permittivity or discharge capacity of confined geotextiles. Semi-empirical and analytical solutions based on equations developed initially for granular medium (Kozeny-Carman, for instance) have also been developed relating the permeability coefficient of a non woven geotextile with its porosity and the properties of the fluid (Gourc 1982, Giroud 1996).

Giroud (1996) presented the following rather simple equation for the estimate of geotextile permeability of non woven geotextiles

$$k = \frac{\beta \cdot \rho_w \cdot g}{16 \cdot \eta_w} \cdot \frac{n^3}{(1-n)^2} \cdot d_f^2 \qquad (3.2)$$

Where k is the geotextile permeability coefficient, ρ_w is the specific gravity of the fluid, g is the acceleration due to gravity, η_w is the fluid dynamic viscosity, d_f is the geotextile fibre diameter and β is a shape factor to account for the shape and characteristics of the flow path followed by the fluid.

Properties of the fluid can be easily obtained as well as values of geotextile porosity under different stress levels. Surprisingly, even at present times values of fibre diameters (d_f) are very uncommon in products catalogues.

Good comparisons between predictions by Equation 3.2 and laboratory test results were obtained, as shown

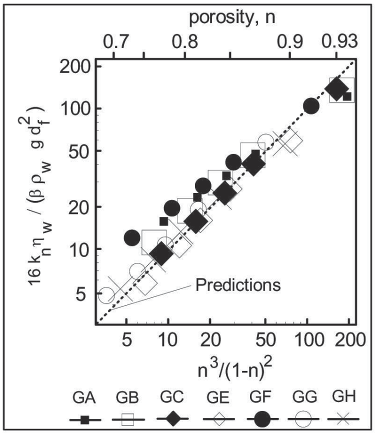


Figure 3.5. Comparisons between predicted and measured geotextile normal permeability.

in Figure 3.5 (Palmeira and Gardoni 2000). The agreement is better for geotextiles porosities greater than 80% and for geotextiles with mass per unit area greater than 300 g/m^2.

By manipulating Equation 3.2, Giroud et al. (2000) derived practical equations for the relation between permeability coefficients, or transmissivities, and thicknesses of non woven geotextiles

$$\frac{k_2}{k_1} = \frac{t_1}{t_2}\left(\frac{t_2 - \dfrac{M_A}{\rho_f}}{t_1 - \dfrac{M_A}{\rho_f}}\right)^3 \tag{3.3}$$

and

$$\frac{\theta_2}{\theta_1} = \left(\frac{t_2 - \dfrac{M_A}{\rho_f}}{t_1 - \dfrac{M_A}{\rho_f}}\right)^3 \tag{3.4}$$

Where k_1 and k_2 are the geotextile coefficients of permeability at thicknesses t_1 and t_2, respectively, M_A is the geotextile mass per unit area, ρ_f is the density of the fibers and θ_1 and θ_2 are the geotextile transmissivities at thicknesses t_1 and t_2, respectively.

As several products catalogues present geotextile thickness, permeability coefficient and transmissivity at a given normal stress, Equations 3.3 and 3.4 allow estimates of these properties for different values of normal stresses, if the geotextile thickness is known. However, variations of geotextile thickness with normal stress can be obtained by simple laboratory tests.

Current permeability criteria for geotextile filters require ratios between geotextile and soil coefficients of permeability between 1 and 100 (Giroud 1982, Carrol 1983, Christopher and Holtz 1985, Corbet 1993 and Lafleur 1999, for instance), depending on the criterion considered. When the geotextile is confined its value of permeability coefficient under confinement should be the one con-sidered in the permeability criteria.

The presence of soil particles inside the geotextile also affects its hydraulic properties. Giroud (1996) presented solutions for the estimate of the geotextile permeability in case of the presence of particles in its voids or attached to the geotextile fibres. For the former situation, comparisons between predictions and laboratory test results yielded satisfactory agreement (Palmeira and Gardoni 2000). Under confined conditions the presence of the soil particles in the geotextile does not necessarily mean that the permeability of the partially clogged geotextile will be smaller than that of a virgin (clean) geotextile under the same normal stress (Palmeira and Gardoni 2000, Palmeira et al. 2005). The impregnated geotextile is less compressible than the virgin one under the same normal stress (Palmeira et al. 1996) and this may influence favourably the permeability coefficient of the former in comparison to that of the latter. The reduction in geotextile permeability due to impregnation will depend on the characteristics of the impregnation process and on the soil and geotextile characteristics.

The equation proposed by Giroud (1996) for the permeability coefficient of a geotextile with soil particles in its voids was employed by Palmeira et al. (2005) to back-analyse the ratio between geotextile permeability ($k_{G\sigma}$) and soil permeability (k_s) in gradient ratio tests under pressure (Figure 3.6). The impregnation level (λ) of each geotextile specimen was measured in the tests and the results of $k_{G\sigma}/k_s$ presented in Figure 3.6 are those obtained for different stages of normal stresses in the range from 0 to 2000 kPa. Despite rather large values of gradient ratio being observed in some of these tests, the back-analysed permeability ratios were above unity in all cases and above 10 in 82% of the cases.

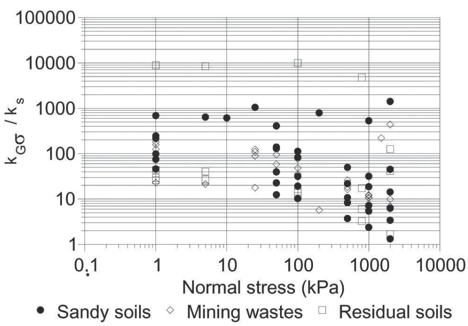


Figure 3.6. Permeability ratios back-analysed from gradient ratio tests (Palmeira et al. 2005).

3.4 *Fine fraction filtration test (F^3 test)*

In the fine fraction filtration test (F^3 test) the geotextile specimen is subjected to the flow of a mixture of water and soil. It may be particularly interesting for the evaluation of the geotextile performance in silt fences in erosion control works. Figure 3.7 shows schematically the test equipment used (Sansone/ Koerner 1992). Mixtures of water with different concentrations of soil particles are forced to pass through the geotextile layer due to the difference of water heads at the ends of the equipment.

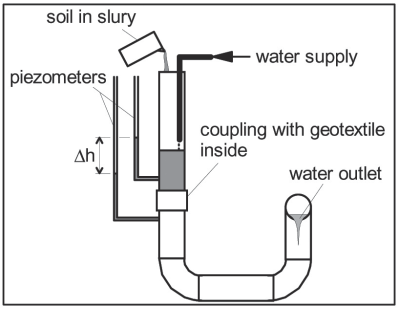


Figure 3.7. Typical equipment for fine fraction filtration tests (modified from Sansone and Koerner 1992).

Figure 3.8 depicts results of F^3 tests on a light non woven geotextile with mass per unit area of 75 g/m^2 and filtration opening size of 0.153 mm, in terms of flow rate per unit area (q) versus cumulative mass of soil added (Palmeira and Farias 2000). The slurry used in the tests was prepared with soils from erosions in the Federal District, Brazil. The results show that the flow rate tends to stabilise for all soils but one (soil C – D_{85} = 0.133 mm, D_{50} = 0.144 mm, D_{10} = 0.0029 mm and C_u = 12.1), for which the flow rate is significantly reduced at the early stages of the test. This result suggests that clogging of that geotextile might be expected if it was used in a silt fence for soil C. Besides, in case of high precipitations the premature clogging of the geotextile may cause overtopping of the fence.

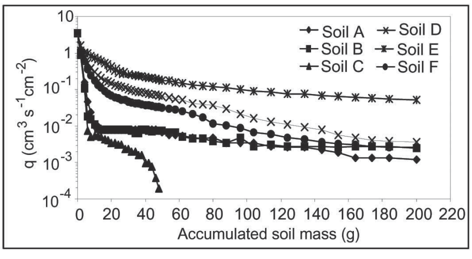


Figure 3.8. Results of F^3 tests on a light non woven geotextile (Palmeira and Farias, 2000).

3.5 *Conclusions*

Laboratory test results have shown that geotextiles can be successfully employed as filters in most situations, even in some cases when available design criteria would not recommend their use. This highlights the need for more realistic filter design criteria where filter in-service conditions are more accurately simulated. On the other hand, some experimental techniques have also to be improved and complementary tests may be needed if the range of application of geotextile filters is to be expanded or better understood. Complex clogging mechanisms such as biological clogging and filter interaction with internally unstable soils are also issues yet to be properly understood.

Investigations on exhumed geotextile specimens from old works can provide significant contributions to the understanding of filter behaviour and to the improvement of design criteria. This is particularly relevant to filters subjected to severe or critical conditions.

The use of geotextiles in major engineering projects requires assurances on their durability and endurance, besides traditional hydraulic and filter requirements. In this context, degradation and mechanical damages are factors that have to be carefully addressed. Because geotextiles are manufactured products and as natural granular materials become scarce or limited in use due to environmental restrictions, the industry and academia should make efforts for the development of alternative efficient synthetic filters and drainage systems capable of enduring the conditions found in major engineering works.

4 EROSION CONTROL

Erosion can be caused by wind, gravity, or water. However, water-generated erosion is the most damaging factor. This kind of erosion will be discussed exclusively here. ISSMGE established the Technical Committee TC 33 "Soil Erosion". (http://ceprofs.tamu.edu/briaud/Scour-tc33/ index.htm).

That committee subdivided the scope into three task forces: "Surface Erosion", "Scour of Foundations" and "Dam Erosion". "Surface Erosion" is defined as water-generated erosion due to occasional water load, often applied to a non saturated soil. "Scour" is mostly recognized as removal of submersed material by waves and currents. "Dam Erosion" was created to group scour effects that are not necessarily covered by the other tasks, even though some of them could be classified also in one of the before mentioned tasks. It incorporates explicitly all scour of rock and concrete. In all three tasks geosynthetics can play a major role, why the following sections are named accordingly.

4.1 *Surface erosion*

4.1.1 *Phenomenon and countermeasures*

Five types of surface erosion can be identified and techniques for minimizing them respectively, outlined in Table 4.1 (Johnson et al., 2003).

Table 4.1. Types of surface erosion.

Type of Erosion	Minimization Technique
Raindrop splash (Raindrop impact of the raindrop dislodges soil, causing it be splashed into the air. The splash effect also increases compaction and destroys open soils structure.)	Stabilize the soil to prevent erosion. Mulch.
Sheet erosion (Transportation mechanism of soil loosened by raindrop splash, removal of soil from sloping land in thin layers. Dependent on soil type, depth and flow velocity.)	Minimize by diverting flow away from the slope.
Rill erosion (Occurs where sheetflow becomes concentrated in small, defined channels a few cm deep. Form of erosion in which most rainfall erosion occurs.)	Prevent by slope stabilization and diverting flow. Repair immediately with disking or tilling
Gully erosion (Concentrated flow in unrepaired rills.)	Requires extensive repair. Prevent by dispersing and diverting sheetflow.
Channel erosion (Occurs at bends and inconstrictive areas.)	Smooth bends, add riprap. Use of bendway weirs or bioengineering methods.

Surface erosion and runoff is a problem mostly during and after construction processes. A construction process e.g. like road building often disturbs soil, which is then vulnerable to being washed downstream when it rains, causing a build-up of soil and other matter in waterways that is known as sedimentation. Excessive sedimentation can destroy fish habitat; clog streams, storm drains, and culverts; and pollute waterways, among other problems. Erosion can also result in additional maintenance and costly repairs. Effective erosion control requires an integrated approach, which considers government regulations, a broad knowledge of temporary and permanent erosion control methods; design, construction, and maintenance considerations; and new technology.

In general, surface erosion can be reduced by: (i) slowing water velocity, (ii) dividing runoff into smaller quantities, (iii) allowing for water infiltration, (iv) providing mechanical or structural retention methods.

Erosion countermeasures should be installed as early as possible in the process of erosion, therefore the best way is to hinder the initiation of erosion (step 1). If erosion has started already, measures have to be provided that hinder the accumulation of material transport (step 2), and if there is already a certain sediment flow, it should be guided not to result in detrimental effects (step 3). Tables 4.2 to 4.4 provide an overview over countermeasures according to the steps mentioned. Silt fences are treated more in depth in section 4.2.

Table 4.2. Countermeasures to prevent surface erosion

Control measure	Geosynthetic contribution
surface roughening (disking; tilling; slope checks)	confinement (netting) of check elements
mulching	netting, fixed on the ground, over mulch
soil retention blanket	natural, biodegradable or synthetic mats (nettings, wovens)
seeding and sodding (turf establishment)	

Table 4.3. Countermeasures to prevent accumulation of material transport

Control measure	Geosynthetic contribution
berm	
slope drain	geosynthetic tubing, geotextile filter
bale check	confinement (netting) of bales
silt fence	geotextile fabric
geotextile triangular dike	urethane foam elements in woven geotextile
riprap	geotextile filter below riprap layer
wattles	confinement (netting) of straw wattles

Table 4.4. Countermeasures to control sediment flow

Control measure	Geosynthetic contribution
bale checks	confinement (netting) of bales
silt fence	geotextile fabric
geotextile triangular dike	urethane foam elements in woven geotextile
riprap	geotextile filter below riprap layer
rock ditch checks	geotextile filter below rock
sandbag barrier	geosynthetic bag material
floating silt curtain	impermeable geosynthetic sheet
sediment trap	geotextile filter below structural elements geosynthetic bag material

Erosion and sediment control measures and practices are actions often taken on an interim basis pre, during, and post construction to minimize the disturbance, transportation, and unwanted deposition of sediment. If installed temporarily, for many application also natural fibres can be used. Such fibres will disintegrate after a certain time, but may remain strong enough until the final situation is reached. Erosion control measures also are installed in natural areas susceptible to erosion due to heavy rainfall, flooding etc. Even though such measures often are man made, erosion protection should not detract from the natural environment. Geotextile can help to realize measures that maintain a natural appearance.

Usually there are no specific design requirements for geosynthetics in erosion protection applications. Certain minimal requirements are given in several handbooks like e.g. “The Erosion and Sediment Handbook” (North Dakota DOT, 2004). If riprap or rock is placed on top of the geotextile, the fabric should be of sufficient robustness (Heibaum, 1998).

4.1.2 *Silt fences*

Erosion barriers can control sediment transportation and contribute to environmental recovery of areas degraded by erosive processes. In this context geotextiles can be used in structures such as silt fences, embankments or gabions to retain the soil particles carried by runoff. Laboratory tests such as the F^3 test described in section 2.5.4 can be used for the selection of a suitable geotextile. Tests using flumes can also be used for the study of the geotextile behaviour in silt fences (Koerner 1998, ASTM 1995d). In this test the geotextile is placed vertically in an inclined flume and subjected to the flow of water with sediments. The results obtained in the test by ASTM (1995d) are geotextile filtering efficiency and flow rate. Compared to the F^3 test, the flume test subjects the geotextile to conditions closer to those expected in a silt fence in the field, but testing equipment and methodology are more complicated.

Farias (2005) carried out large scale flume tests to evaluate the performances of some light and low cost non woven geotextiles as sediment barriers in silt fences. The silt fences tested were 1 m high and 0.4 m wide. Figure 4.1 shows a general view of the flume used in the tests. Soils samples were collected from large gullies to prepare the slurry used in the tests. The results obtained were the variation of flow rate versus time, concentration of the soil-water mixtures downstream the fence, grain size analyses of the soil that passed through the geotextile and permittivity tests on the geotextile specimens after the tests. Figure 4.2 shows the comparisons between the diameters (d_{95}) of the soil particles that piped through the geotextile versus geotextile filtration opening size (O_{95}) for tests with three different soils (ErCe1: D_{95} = 0.35 mm, ErCe2: D_{95} = 0.163 mm and ErAn: D_{95} = 1.22 mm, where D_{95} is the diameter for which 95% of the soil particles are smaller in grain size analysis). The particle diameters of the soils tested and of the particles that piped through the geotextile were obtained using a laser beam grain size analyser.

Figure 4.1. Flume for tests on silt fences (Farias 2005).

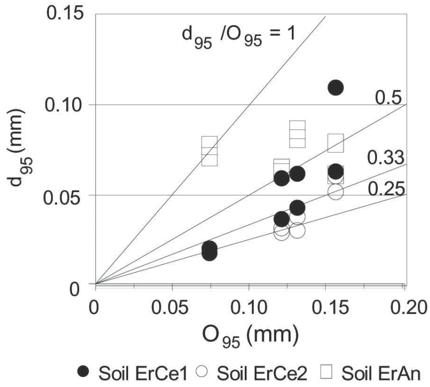


Figure 4.2. Comparisons between diameters of piped particles and geotextile filtration opening sizes.

In Figure 4.2 it can be observed that the diameter of the particles that passed through the geotextile were considerably smaller than the geotextile filtration opening size and than the particle diameters upstream the fence. These results can be attributed to the smaller openings available in the geotextile due to partial clogging caused by the sediments that first reach it. Besides, the flow through the geotextile is not entirely normal to its plane along the entire fence height, being similar to what would occur in an under-designed chimney filter not able to allow free water flow along its height. In this case the soil particles have to travel a distance greater than the geotextile thickness to reach the downstream region of the fence, increasing the probability of them being entrapped in the geotextile layer.

The use of light geotextiles in silt fences is particularly interesting because it reduces the costs of the structure. However, the geotextile has also to attend mechanical and survivability requirements for a satisfactory performance (Holtz et al. 1997).

Figures 4.3 and 4.4 show examples of low cost silt fences for the stabilisation of gullies (Farias 2005). The non woven geotextile used was selected based on flume tests performed in the equipment shown in Figure 4.1. Figure 4.3 shows the fence being assembled, with the geotextile layer resting on a wire mesh supported by wooden poles. Some of the advantages of this type of structure are that it is easy and quick to install. Figure 4.4 presents an upstream view of the fence showing the sediments retained by the geotextile after a period of heavy precipitations.

Figure 4.3. Downstream view of the silt fence construction.

Figure 4.4. Upstream view of sediments retained by the fence.

It should be pointed out that large surface discharges can cause overtopping of the fence. Thus, appropriate hydraulic requirements must also be taken into account in designs under such conditions. Stability of the fence shoulders has also to be guaranteed, as erosion in these regions may compromise the fence stability.

4.2 *Scour*

4.2.1 *General*

According to the dictionary, scour is the "Removal of soil or fill material by the flow of floodwaters. The term is frequently used to describe storm-induced, localized conical erosion around pilings and other foundation supports where the obstruction of flow increases turbulence. See Erosion." This explanation states very clearly, that scour and erosion are basically the same process. More in detail: "Scour is (i) Removal of sand or earth from the bottom or banks of a river by the erosive action of flowing water, (ii) Erosion of a concrete surface, exposing the aggregate, (iii) The action of a flowing liquid as it lifts and carries away the material on the sides or bottom of a canal, conduit, or pipeline, (iv) The enlargement of a flow section of a waterway through the action of the fluid in motion carrying away the material composing the boundary."

A limited local scour may be tolerated, but as soon as scour increases with time, countermeasures are needed. Either the action has to be reduced or the resistance has to be increased. Changing the action means to alter the flow pattern such that scouring is stopped. This can be done for instance by river training works or by a construction design that anticipates scouring. But often no alteration of actions can be realised, or the comparison of costs shows that increasing the resistance might be the better way. In both cases scour countermeasures include in many cases geosynthetic material in a large variety of products.

Minor scouring at the borders of a scour protection is inevitable but they can be accepted to a certain extent. From this a general demand arises: a good scour protection system has to be flexible. The demand for flexibility holds for all elements of a scour repair and prevention work, i.e. fill, filter and armour. When using rigid systems, it has to be guaranteed that neither below nor beside the armour any erosion will develop. This can hardly be achieved, so a flexible system always performs better.

4.2.2 *Scour countermeasures to increase the resistance*

Increasing the resistance means at first to strengthen the water exposed surface to hinder the hydraulic transport of material. Often just coarse material is placed as an armour layer, since it is a problem to place the necessary filter on top of the subsoil and below the armour. But if there is no filter at the interface subsoil–armour, the coarse material will sink into the subsoil due to fluidisation of the subsoil. This process will take place until so much armour material is placed that there is an equivalence of load and resistance due to the mere thickness (that might not be sufficient for a following hydraulic load). Geotextiles allow for the perfect design of scour countermeasures and scour repair, but often special methods are required to realise it.

Scour countermeasures often have to be build in flowing water or under wave action. To place geotextile filters in such an environment, special equipment is needed, but even then the placement depth is limited to approximately 20 m. To keep the filter mat in place despite flow or wave action e.g. steel chains are connected to the fabric or a sand fill in between two geosynthetic cloths ("sandmat") is used. Such sand mats filled with 5 kg/m^2 sand have proved in tests to remain in place loaded by currents up to 0.8 m/s. The maximum fill available today is ca. 9 kg/m^2 (with more sand fill the needles for sewing or needle punching will degrade too fast).

Traditional large single elements for scour protection and repair that can be installed in (nearly) any depth are fascines, originally large willow bundles that have a core of rubble or riprap, to provide a sufficient resistance against the current. Instead of these large elements today often geosynthetic containers are used (Heibaum 2004). Containers are treated in the Giroud Lecture of this conference and will not be discussed here.

To protect a larger area, mattresses are used since long, especially in coastal protection works and in large rivers. The oldest form of a mattress is the fascine mattress (or willow mat, Fig. 4.5), i.e. willow bundles with a diameter of 10 to 40 cm fixed crosswise to form a large grid. In the beginning only fascines and brushwood were combined to a mattress. But brushwood and fascines are a bad filter. Only coarse soil may be retained efficiently. Erosion may be slowed down due to the damping of the erosive effect of the current, but it will not be stopped. The important step forward was made when combining fascines and geotextiles to a fascine mattress. Modern fascine mattresses usually comprise a base woven geotextile with willow bundles tied on it. The fascines ensure the spreading of the geotextile and the floating of the mattress during the transport to the point of installation. The geotextile acts as a filter that today can be improved by using a geocomposite of woven and nonwoven fabric. This way, sufficient strength and perfect filter design according to the recommendations given in section 2.3 can be combined. Fascine mattresses are prefabricated according to the desired geometry on land, then they are pulled to the desired position and drowned by dumping armour material upon. Placement is possible even in great depth.

Figure 4.5. Preparing a fascine mattress (willow mat).

An alternative are geosynthetic mattresses filled with concrete or mortar. They can be placed "endless", i.e. without overlaps that always bear a certain risk of improper covering. The fabric is sewn together as needed before the mattress is filled. With this solution the same problems arise as with geotextile filters: The fabric tends to float before it is filled. Mattresses of uniform thickness are inflexible and impermeable. To achieve a certain flexibility and permeability, mattresses consisting of columns and rows of "pillows" are used. The seams between the concrete filled pillows provide the necessary permeability of the layer (usually an extra filter fabric is needed) and the desired flexibility for good adjustment to any deformation of the subsoil. Such mattresses belong to the family of geosynthetic containers that are treated in the Giroud Lecture. An alternative are prefabricated mats with concrete elements cast directly on a geosynthetic fabric. Such "panels" will withstand flow and wave loading but they are of limited size and are usually placed in low depth only.

4.3 *Dam erosion*

The following aspects of dam erosion can be distinguished according to TC 33:

(1) "Dam foundation erosion, including of erosion of foundations that may occur due to overtopping of gravity and arch dams, as well as erosion that might occur due to the presence of earth fissures." In such cases similar scour countermeasures as in flowing water can be installed. Since this can be done mostly in the dry, the installation of a geotextile filter needs no special equipment. Because of the mostly large armour elements, only robust fabric should be used.

(2) "Plunge pool scour due to impingement of free-falling jets." Scour countermeasures in such cases ask for high strength of the surface. Geosynthetics may be involved but are not essential for the success of such works

(3) "Scour of auxiliary spillways including scour in earth, vegetated earth material and rock." The contribution of geosynthetics to the success of countermeasures in this context is similar to paragraph (1).

(4) "Dam breach due to overtopping or internal erosion (flow through cracks in embankments and flow leading to piping failure)". Scour countermeasures for such risks of earth dams often include revetments incorporating geotextiles as a filter either in the area of fluctuating water level or as a protection when overtopped. In both cases the filter has to be designed to "severe applications" as treated in section 2.2. Another countermeasure is the installation of a toe drain with a geotextile filter. In such application the capacity of ochre formation or calcification has to be checked carefully.

5 GEOTEXTILES AS CAPILLARY BARRIERS

One of the greatest problems facing the waste management industry worldwide is the issue of the safe and environmentally acceptable closure of waste disposal sites. This applies to all waste sites, from municipal solid waste landfills, through mine tailings deposits, to industrial and hazardous waste disposal sites. Integral to the design of closure systems is invariably the use of some form of capping system. This capping system may have many functions, including the provision of a suitable growth medium for vegetation, minimising erosion of the topsoil, protection of the entombed waste from disease vectors and many others. However, probably the single greatest requirement is the prevention of moisture infiltration (from precipitation) into the stored waste. If this is not controlled, infiltration of moisture will inevitably result in the generation of leachate, which will eventually emerge from the base of the waste deposit. Although most modern waste disposal sites have some form of internal drainage (often incorporating geosynthetics), blockage of these systems is not uncommon and leakage of contaminated leachate to the subsurface is a continuing concern for owners and operators of waste disposal sites.

Along with the control of moisture infiltration, control of oxygen ingress to the waste may also be of concern, particularly when the waste is potentially acid generating, such as occurs with many pyritic waste rock and mine tailings material. An additional requirement may be the prevention of upward movement of contaminated moisture by capillary action. An example is the movement of highly contaminated saline water from mine tailings into the surface topsoil layer during periods of extended dry weather, which ultimately results in the death of vegetation within the cover layer system.

In the early days of cover system design, it was common to include a layer of compacted clay in the cover. This was to prevent the movement of moisture into (or out of the waste) as discussed above. There is increasing evidence (Suter et al. 1993) that solutions of this type are completely inappropriate for use in arid and semi-arid climates, as the clay material invariably dries out and desiccates, resulting in the formation of cracks that become preferential flow paths for the ingress of water to the stored refuse. Many designers have therefore turned to the use of geosynthetics in capping systems and the use of either a geomembrane or geosynthetic clay liner is not uncommon. Similar problems of desiccation drying may however occur with the latter and issues of root penetration, installation damage and differential settlement-induced damage remain a concern with geomembranes installed at shallow depths below ground surface. Attention is increasingly turning to the potential for using geotextiles as moisture controlling barriers in capping systems. This may sound counter-intuitive, given the inherent porosity and permeability of these materials, but the concept relies on the concept of a capillary barrier system, as explained below.

The concept of a capillary barrier relies on differences between the unsaturated hydraulic properties of two materials placed adjacent to one another. It was originally conceived for two different soil layers, placed in separate layers, as illustrated in Figure 5.1.

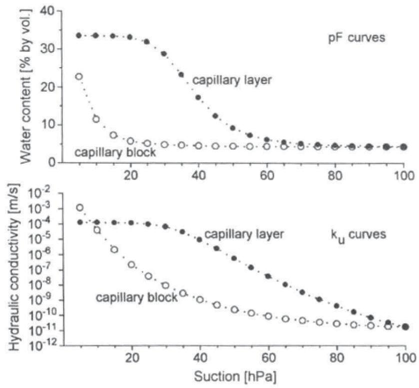


Figure 5.1. Illustration of unsaturated hydraulic properties of two soil layers that together provide a capillary barrier system.

The material designated as the 'capillary block' would typically be a coarse grained pea gravel, which is placed below the 'capillary layer', which could be a clean fine sand for example. The pea gravel acts as a barrier to downward movement of water under unsaturated conditions as well as inhibiting upward movement of moisture by capillary action from the encapsulated waste. In this figure the horizontal axis in both cases is suction (or negative pore water pressure) expressed in hPa. The top curve shows the variation of water content (usually given as volumetric water content as opposed to the geotechnical version, which is gravimetric water content) with suction and curves such as this have become known as water retention curves – the ability to retain water under an increasing applied suction. The lower curve shows the variation of hydraulic conductivity with applied suction. As can be seen, for the capillary block layer (pea gravel typically), the hydraulic conductivity drops extremely rapidly as the suction is increased. Thus although the saturated (zero suction) value of hydraulic conductivity may be extremely high (in the example shown about 10^{-2} m/sec), under an applied suction of only 40 hPa, it decreases by about seven orders of magnitude. In contrast, the hydraulic conductivity of the capillary layer (the fine sand) decreases much more slowly. At relatively low values of suction, the capillary layer has a higher hydraulic conductivity than the coarser material (the capillary block layer). Continuity requires that the suction must be the same on both sides of the interface between the capillary block and capillary layer and thus as long as both layers are not fully saturated, the capillary block layer has a hydraulic conductivity lower than that of the capillary layer. In fact, during infiltration events the capillary layer can become virtually fully saturated (with a small residual of suction of about 20 hPa in the illustration used here) while the underlying gravely layer remains virtually dry, but acts as a hydraulic barrier.

This concept has been successfully demonstrated in the laboratory, in pilot scale trials and even at full scale (Von der Hude/Jelinek, 1993). However, there are some difficulties with using mineral soil layers alone. For example, it is difficult to construct full scale capillary barrier systems to the very high tolerances required. It is imperative that no mixing of the two layers takes place and indeed any ingress of the finer material into the underlying coarser fraction can create a wicking effect that negates the exact phenomenon that one is trying to take advantage of. Although there have been some effective installations of purely mineral soil capillary barrier systems (Jelinek/Amman, 2001), the construction difficulties mentioned above, together with the difficulty in many countries of sourcing adequate clean gravel or similar material, has restricted the use of these systems. This is where geosynthetics have begun to be accepted as potentially providing a very useful function in capping systems.

Moonsammy (2003) experimented with the use of woven slit-film geotextiles as a layer placed between coarse, angular andesite gravel and fine, quartzite sand to form a composite capillary barrier system. Without the geotextile layer, the jagged and angular gravel was easily contaminated by the fine sand, resulting in loss of integrity of the interface and penetration of the capillary block layer by water under very low hydraulic heads. The system incorporating the geotextile sheet functioned extremely well in small scale laboratory tests, but remains to be tested in full scale systems. It seems that there is more likelihood of thick nonwoven geotextiles being satisfactorily used in capillary barrier systems in the future, with the potential for using them as a complete replacement for the capillary block layer, rather than solely as a separator, as was the case with the woven geotextile experiments mentioned above.

Many geotextiles have been observed to have non-wetting (hydrophobic) characteristics (Allen et al., 1983, Shoop and Henry, 1992), which will in all likelihood further add to their ability to act as capillary barriers. Early experiments carried out by Clough and French (1977) using one-dimensional unsaturated water movement in columns demonstrated that geosynthetic capillary barriers reduced the flow of water normal to the plane of the geosynthetic. However, it was almost a further twenty years before researchers began looking at this potential application for geosynthetics in some detail. The simplest characterisation test is to simply immerse one end of a geotextile strip in water and measure the height above the water surface to which the water level rises due to capillary action, as described by Henry and Holtz (1997). The height of capillary rise is considered to be the water entry value of suction and for hydrophobic geotextile fibres, a capillary depression actually occurs. This test is probably only useful as a comparative test and for a more complete characterisation, researchers turned to a variation of the conventional laboratory test used to define the water retention relationship mentioned earlier.

An apparatus used by Stormont et al. (1997) consisted of a funnel that was fitted with a porous plate, a water reservoir and tubing connecting the funnel and bottle as shown in Figure 5.2. The ceramic porous plate sealed into the funnel mouth had an air-entry value of approximately 20 kPa. To commence a test, the ceramic is saturated and the connecting tube beneath it filled with water, thus providing a continuous column of water from the top of the porous plate to the water in the reservoir. When a geotextile specimen is placed on the surface of the porous plate, hydraulic continuity transmits any head difference between the plate and the reservoir to the geotextile specimen, effectively applying a suction or negative pore water pressure. The geotextile will either absorb or release water in order to equilibrate with the water in the reservoir. The water retention function is obtained by sequentially lowering or lifting the reservoir, allowing the geotextile to equilibrate with the suction and when it has done so, weighing the geotextile specimen and calculating the associated water content. Stormont et al. (1997) tested four nonwoven needle-punched polypropylene geotextile specimens using this apparatus and a typical result is shown in Figure 5.3.

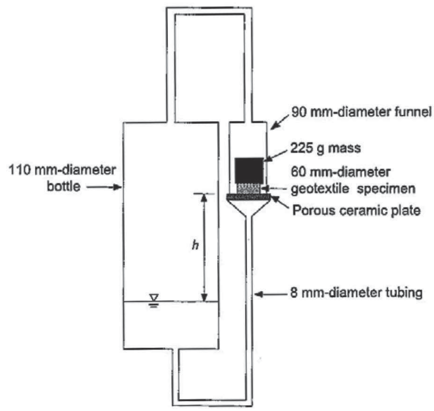


Figure 5.2. The experimental test apparatus used to measure the water retention functions of geotextile specimens reported by Stormont et al. (1997).

They tested specimens as-supplied (indicated as "new" in this figure) as well as the same specimens after washing them in tap water, squeezing by hand,

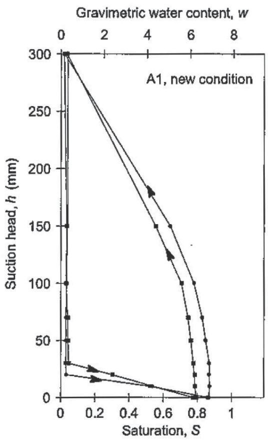


Figure 5.3. The wetting and drying water retention functions of a new (clean) nonwoven geotextile specimen (after Stormont et al., 1997).

repeating the procedure and then drying them. The reason for doing this is that it was believed that certain surfactants used in the manufacture of geotextile products could affect their "wettability" characteristics, and indeed their test results confirmed this hypothesis. Figure 5.3 is for a staple fibre product having a mass per unit area of 339 g/m^2 and an apparent opening size of 0.15 mm. During the drying phase (decreasing degree of saturation), the geotextile does not immediately desaturate, with an applied suction greater than 100 mm head (only 1 kPa) being required to meaningfully reduce the water content. At suction values greater than this value it desaturates rather quickly and under a suction of 300 mm head it becomes virtually dry. An interesting feature of the wetting curve, which is completely different from most soils, is that the geotextile does not re-wet until the suction is virtually zero (about 25 mm suction head). This value at which wetting recommences is known as the water-entry value and is again much lower than that obtained with soils. This somewhat unusual behaviour was found for all four geotextiles tested, although it was less pronounced with continuous filament geotextiles than with the staple fibre products. In the latter case, the water-entry value was of a similar magnitude, but the air-entry value was 25 to 50% lower.

The effect of cleaning the geotextile specimens was to reduce their water retention ability at all values of applied suction. This change in performance was attributed to an increase in the contact angle between the geotextile fibres and moisture as a consequence of the removal of the surfactant coating, thus rendering the geotextile less 'wettable' with water. If we consider that 10 hPa (see Figure 5.1) is equivalent to 100 mm suction head, the geotextile water retention curves are very similar to the curve for the capillary block shown in Figure 5.1. Geotextiles of the type tested in this work should therefore perform at least as well as a typical well rounded gravel would perform. In fact, it can be speculated that they would perform even better because during the wetting cycle (which after all is the condition to which a geotextile in a capping system would be subjected to during an infiltration event), no water enters the geotextile until the suction value is almost zero, i.e. a positive water head would have to build up at the interface before any water penetrates into the plane of the geotextile.

A more sophisticated apparatus for measuring the water retention curve for geotextiles was described by Knight and Kotha (2001). It was a variation on the controlled outflow testing cell originally described by Lorentz et al. (1993). It is similar to the conventional technique of the ASTM method, but has the advantage of a much shorter testing time, as it does not require that water outflow ceases completely during an increment of applied pressure (or suction) but rather relies on measuring the suction that results from allowing a known volume of outflow.

They validated their testing procedure by comparing results obtained using the equipment with previously published data for a fine sand. Excellent agreement was obtained. Tests on a nonwoven geotextile with an apparent opening size of 0.15 mm were carried out using the equipment. Only drying tests were done and the existence of very low water entry values (as reported in the earlier studies) could not be confirmed. Their tests showed a similar behaviour to those of Stormont et al. (1997) for the drying phase. The degree of saturation decreased from unity to less than 20% over an applied suction varying from 30 mm to 150 mm of negative water head. The results shown in Figure 5.4 appear to be very characteristic of nonwoven geotextiles subjected to drying tests, i.e. once they begin to desaturate, it happens extremely rapidly. Also shown in Figure 5.4 are results from tests on a stack

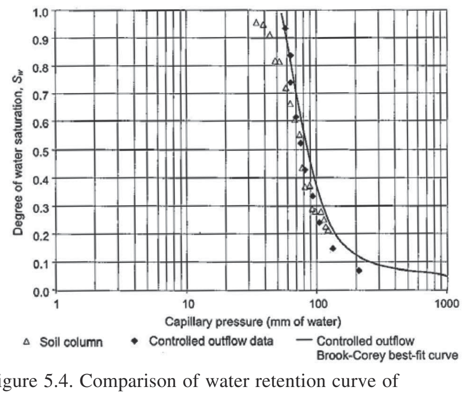


Figure 5.4. Comparison of water retention curve of nonwoven geotextile from controlled outflow test with that from column drainage experiment, (after Knight and Kotha, 2001).

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of discs of the same geotextile placed into a vertical column that was initially flooded and then drained. By measuring the mass of geotextiles from different heights above the stationary 'water table' at the base of the column, a water retention curve was constructed. As can be seen, the results are very similar, with the column test having the added advantage of providing results at extremely low values of imposed suction. Stormont and Morris (2000) confirmed the value of using a simple test procedure, in this case a hanging column test similar to that described by Stormont et al. (1997), testing two different nonwoven geotextiles, a polyester and a polypropylene product, having apparent opening sizes of 0.04 and 0.18 mm respectively. They used a slightly different testing procedure than that used previously, carrying out a drying test first, followed by a wetting test. They again found very similar results to those reported previously, with both geotextile products showing very abrupt losses of moisture for applied suctions greater than the air entry value and rapid re-saturation once the suction decreased below the water entry value. Air entry values were between 0.5 and 1 kPa, which was similar to previously reported data. Stormont and Morris also considered the impact of penetration of soil particles into the plane of the geotextile and the impact this had on the water retention capability of the materials. This scenario is a valid concern, given the potential use of geotextiles in capping systems for waste disposal sites. Their results are summarised in Figure 5.5, where the virgin polyester geotextile product (shown as solid dots) is compared with specimens that had been impregnated with an amount of either sand, silt or clay particles. The curves for all impregnated specimens are similar, but differ from the uncontaminated specimen data. The effect of intrusion of soil particles is to increase the water entry suction value, meaning that a smaller build up of water head above the geotextile can be tolerated before breakthrough occurs when used in a capillary barrier system. Despite this observed increase, the water entry value remains extremely low and is less than most values for natural soil or gravels reported in the literature. Stormont and Morris (2000) also did a series of model one-dimensional

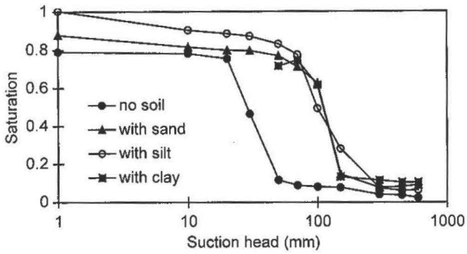

Figure 5.5. Effect of intrusion of soil particles on water retention curve of a nonwoven geotextile specimen (after Stormont and Morris, 2000).

capillary barrier tests in the laboratory which compared the performance of a barrier consisting of a silty sand (SM) overlying a coarse sand (SP) with a similar arrangement that included the polypropylene geotextile sandwiched between these two layers. The barrier that included the geotextile performed better than the soil-only system, producing a water entry (or breakthrough) suction value of 1.6 kPa compared with 3 kPa for the soil-only system. Considering the other advantages of a system that includes a geotextile sheet, i.e. the assurance that no penetration of fine particles into the underlying coarse layer occurs, it seems surprising that the concept has not been more widely used in full-scale applications. Indeed, it could be argued that the geotextile could replace the underlying coarse layer completely, thus providing simultaneous functions of separation and a capillary break.

Further test work by Morris (2000) using a hanging column technique and Lafleur et al. (2000) using the simple technique of suspending a geotextile strip in a reservoir of water confirmed the findings of the earlier studies discussed above. An inescapable conclusion was that the water characteristic curve for nonwoven geotextiles is much steeper than that for sands and the air and water entry values of these geotextiles are very small. Iryo and Rowe (2003) analysed data from all the above studies, using the van Genuchten (1980) equations to fit the experimental curves and to predict the variation of unsaturated hydraulic conductivity with suction. Using this information, they analysed data from a one-dimensional sand column having a horizontal geotextile layer inserted at about one-third the height of the column. The finite element technique was used, with van Genuchten parameters applied to both the sand and the geotextile water retention data. They produced reasonable reproductions of the experimental data, indicating that analysis of proposed full-scale installations may be carried out using such a procedure. Recently, Bouazza et al. (2006) confirmed the similarity of water retention curves obtained from hanging column and controlled outflow tests for a polyester nonwoven geotextile having an apparent opening size of 0.18 mm. They only reported drying tests, which confirmed the air entry value was of the order of 1 kPa, similar to many previously reported values.

The primary consideration in the studies discussed above was the use of geotextiles as capillary barriers to prevent downward movement of moisture. Henry (1998) considered an alternative use, namely the ability of a geotextile to prevent upward movement of moisture due to capillary action under conditions of freezing of the soil overlying the geotextile. The nonwoven geotextiles were found to be ineffective in impeding upward flow if they were already moist (greater than 30% degree of saturation). This limitation is likely to restrict the use of geotextiles in this application as it is likely that the geotextile will often be moist due to exposure to a rising water table in such situations. It would therefore seem that based on data currently available, the best use of nonwoven geotextiles would be in cover systems as capillary breaks to minimise downward movement of moisture. A refinement to the use of a single nonwoven geotextile was presented by Park and Fleming (2005). They tested the use of a product they termed a geosynthetic capillary barrier (GCB), which consisted of two layers of nonwoven geotextiles between which a layer of fine rock flour was sandwiched. The reason for introducing the added complexity was to produce a product that could ensure the capillary barrier effect develops, with a view to maintaining a high degree of saturation of the soil overlying the GCB in applications where oxygen ingress to the retained waste must be prevented. The GCB was tested using a pressure plate cell and similar results to those obtained in previous studies of single sheets of geotextiles were obtained. A numerical simulation of the intended application showed that the GCB was likely to function as intended, ensuring a high degree of saturation was maintained in the soil overlying the product. It remains for the system to be tested in a full-scale application.

With the burgeoning interest in the need to produce capping systems for waste disposal sites that will maintain their integrity for the very long term, the use of nonwoven geotextiles as an integral part of a capillary barrier system seems inevitable. It may be that the geotextile is used as a separator between the overlying finer grained material and the lower coarse grained (gravely) layer, or even as a complete replacement for the gravel layer. This is a particularly attractive option in countries such as Australia and South Africa where well rounded river gravel is not common and is expensive to source.

6 ELECTROKINETIC DEWATERING OF SOFT SOILS USING GEOSYNTHETICS

In-situ dewatering of thick deposits of soft clay is highly desirable. The most commonly used technique to achieve this dewatering includes the installation of prefabricated vertical drains (PVDs, see section 7). In order for these drains to work it is necessary to create a flow gradient in the tailings, which is usually achieved by the application of a surcharge load (e.g. by constructing a surcharge of imported fill). The drawbacks of this approach include the cost of importing and placing the fill, potential instability of the fill because of the low shear strength of the in-situ material and the long time required for the desired degree of consolidation to be achieved.

Electro-osmotic dewatering provides a potentially attractive alternative technique for in-situ dewatering. Since the electro-osmotic conductivity, k_e, is independent of pore size, the technique is particularly appropriate to very fine-grained soils. The technique, which involves the application of a potential difference between electrodes and causes the flow of water to the negatively charged cathode, has seen some limited use in civil engineering applications (Chappel and Burton, 1975, Soderman and Milligan, 1961, Bjerrum et al., 1967, Casagrande, 1983, Shang and Dunlap, 1996). However, as noted by Lo et al. (1991), despite the successful case histories, the process was considered economically impractical (primarily due to the high operating costs) and usually only entertained as a last resort. As an example, in the project described by Bjerrum et al (1967), the cost of electricity was 25% of the total project cost, which is prohibitively high. At this level of cost, the technique would most certainly only be used as a last resort and when the alternative to no treatment is failure of some sort. Other impediments to the widespread adoption of electrokinetic dewatering techniques are the corrosion of electrodes (particularly the anode) and the lack of proven practical implementations. There have been attempts to use graphite electrodes or metal electrodes with a carbon coating, but with little success (Bergado et al., 2003).

Recently developed electrokinetic geosynthetics (EKGs) may alleviate the above problems, potentially making in-situ electrokinetic dewatering a viable alternative to the use of conventional PVDs (Jones et al., 1996). Hamir et al. (2001) describe the development of electrically conductive geosynthetics (EKGs) and in comparative tests they performed as well as copper electrodes. Filter tests showed no clogging of the EKGs or loss of material through the EKG. Jones et al. (2002) conducted tests using EKGs to dewater kaolin clay and observed no deterioration of the electrodes with time. They also reported an energy consumption rate of 4.66 kWh/dry tonne, which compares very favourably with results reported in the literature. The EKGs used in the study were described in Pugh (2002). The core of the EKG is a geonet made from conductive carbon black dispersed in a modified high density polyethylene resin. A metallic stringer wire was centred in alternate ribs of the geonet. The EKG was wrapped in a non-woven heat-bonded geotextile when used as a cathode, which provides the added benefit of acting as a drainage conduit for water collected at the cathode. As described by Pugh (2002), the durability of the EKG electrodes is vastly superior to that of metallic electrodes. The applicability of EKGs to the dewatering of soft clays was demonstrated in a large indoor testing facility by Jones et al. (2002) and in a field trial by Karunaratne et al. (2002). In both cases, very rapid rates of settlement were achieved. In addition, it was found that increases in undrained shear strength greater than what would have been expected from the decrease in water content alone were achieved, indicating that some form of material alteration (perhaps due to bond formation) had occurred. This factor clearly needs additional research as it could provide an added benefit from electrokinetic dewatering. In both these studies, the benefits of polarity reversal, in which the direction of current flow is regularly alternated, was very clearly demonstrated.

These new generation products, which combine the functions of drainage with those of electrokinetic dewatering, have seen use in trials in material other than just soft soils. Recently Lamont-Black et al. (2005) and Jones et al. (2006) have reported on field pilot trials of the dewatering of lagooned sewage sludge. Excellent results were achieved, with the solids content increasing from 10% to about 27%. During this process very clean discharge water was produced, with BOD values of less than 3 mg/litre resulting. Power consumption was higher than for the tests on kaolin clay, being approximately 43 kWh/m^3. The technique has also been proven for the dewatering of fine-grained tailings derived from the mining of mineral sands, in large outdoor experiments described by Fourie et al. (2004). In these tests, energy consumption rates as low as 1kWh/dry tonne of material dewatered was measured, providing a potentially inexpensive technique for the in-situ dewatering of large volumes of soft tailings material.

Conclusion

Many of the applications of geosynthetics to solving problems of controlling moisture migration described in the preceding two sections are well known and proven applications. The two applications described above, namely the use of geotextiles in capillary barrier capping systems and the use of new-generation electrokinetic geosynthetics (EKGs) for dewatering soft clays, mine tailings and sewage sludge are relatively new but present the potential for significant development in the future. Problems of controlling moisture ingress to stored waste material, particularly the very large volumes of mine tailings and waste rock that are produced annually, would seem to be an application that could benefit significantly from the inclusion of nonwoven geotextiles in many capping systems. Active dewatering using the technique of electrokinetics could similarly revolutionise the stabilisation and management of otherwise problematical material such as large lagoons of soft mine tailings and sewage sludge, or deep deposits of sensitive natural clays.

7 PREFABRICATED VERTICAL DRAINS (PVD)

7.1 *Installation filter stress in PVDs*

Annually several million meters of prefabricated vertical drains (PVDs) are installed worldwide to accelerate consolidation of soft soil deposits. Recent projects in Singapore recorded an annual consumption of more than 20 million meters. Rates of installation as much as 8,000 m to 30,000 m per 14 hour day per machine have not been uncommon in these projects (Choa et al. 2001). New installation rigs with mandrel insertion and withdrawal speeds exceeding 1m/sec have been developed for these projects (Cortlever and Dijst, 2002).

Prefabricated vertical drains (PVD) normally consist of a core and a filter (sleeve) made with polymeric materials. PVD is normally spooled out and threaded through a hollow mandrel. It is attached to a shoe for anchoring in stiffer clay and to prevent soil intrusion into the mandrel. The shoe may vary from a simple reinforcement steel bar (re-bar) of 10 – 20 mm diameter to a thin mild steel plate to which a small mild-steel strip is welded as a handle. The PVD passed around the re-bar or mild-steel strip is stapled.

Little to profuse soil intrusion during installation would take up the annular space between the sleeve and the mandrel, the occupied length up the mandrel being proportional to the volume intruded. This soil develops shear stress on the filter surface, the magnitude increasing with volume and speed of intrusion. During the mandrel withdrawal, the shear stress on the filter develops from the intruded soil. If the PVD is properly anchored in the ground, the higher the withdrawal speed the greater the shear stress on the filter. The upward shear stress will translate into a tensile force in the filter.

PVDs are extensively tested in laboratories for QC/QA. Since there are no criteria for field assessment of QC and behaviour of PVD under kinked conditions, it is possible that filter strength in deep installations, typically more than 20 m, would be prone to greater stresses at high installation speeds. If the tensile stress exceeds the ultimate tensile strength of the filter, typically about 1000 N, then the PVD core will be exposed to the soft clay, causing channel blockage and depriving discharge flow. If a sufficiently large number of deep installations result in damaged filters at crucial PVD depths, the consolidation of the clay in that region will not meet design expectations.

Several instrumented PVDs installed in recently reclaimed lands underlain by soft marine clay in Singapore showed some startling results. Among the monitoring instrumentation were two specially prepared strain gages (SG) on the filter sleeve. One gage (SG-A) was placed at 300 mm and the other (SG-B) at 1000 mm from the folded end of the PVD at the shoe, as shown schematically in the inset of Figure 7.1 (Karunaratne et al., 2003). Figure 7.1 shows the recorded data illustrating the rise in tension in the filter with installation, at insertion and extraction from the ground.

The maximum filter tension of about 1000 N was displayed near the shoe, shortly after the

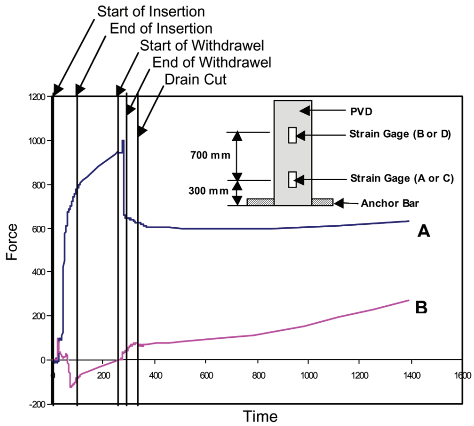

Figure 7.1. Filter stress record during installation in 28 m depth of installation (After Karunaratne et al, 2003).

commencement of mandrel withdrawal. It began to drop as the mandrel was withdrawn within about 46 seconds. The tension recorded in SG-A continued to decrease to a residual value, which practically stayed constant up to 23 minutes of monitoring. In contrast, SG-B (at 1000 mm from the shoe) recorded a smaller force throughout the episode.

Table 7.1 shows two commercially available PVDs tested using two different installation rigs with varying speeds from 0.83 sec/m to 0.35 sec/m. The installation forces varied from 377 N to 1278 N illustrating correlation between the installation speed and the characteristics of the filter. It is imperative that the strength should be sufficiently high to preclude filter damage occurring below the ground level during installation stage.

Larger tensile stresses developed in faster installed PVD. On the contrary, the higher the speed of penetration of the mandrel, the smaller the chance of soil intrusion and hence smaller the installation filter stresses. Therefore, it might be prudent to install PVDs at a faster rate and withdraw at a slower rate, which would necessitate higher filter strength and/or slower withdrawal in thicker soft deposits. The shoe and the mandrel section should also be properly designed for preventing or minimizing soil intrusion. Alternatively, a PVD with higher tensile strengths in the core and the filter may be used for deeper PVD installation. The need to watch the stresses in PVDs, especially in the filter, in relation to the material properties of the components is important.

7.2 *PVD field behavior revisited*

Much interest in the form of theory, application and field analysis has been shown in the literature on PVD projects. Notable among them are Barron (1948), who introduced sand drains, Hansbo (1981), Holtz et al. (1986), Bo et al. (2000), Bergado et al. (2003), Indraratna et al. (2001). Theoretical considerations take in to account the competitive influence of a number of PVD parameters and relevant factors. Applications deal with practical problems faced in implementing PVD projects while trying to approach the theoretical assumptions made. Divergence between the theory and the observations is generally supported by field back-analyses and recognized as shortcomings of the theory and/or application problems. It is clear that certain inherent factors in the PVD-soil system cannot be addressed sufficiently well. These factors include integrity of PVD, interaction of the soil and the PVD in respect of soil fabric and the extent of smear develop around the PVD.

Table 7.1. Shows the details of few types of PVD tested in Singapore.

Drain Type	FlexiDrain FD767	Mebra 7007	Mebra 7007
Installation Rig	PC 1000	RH 40-E (C-1)	PC1000
Mandrel shape	Rhombic	Rectangular	Rhombic
Shoe Type	Re-bar	Plate + handle	Re-bar
Date of Test	4 October 2001	11 Oct 2001	14 Nov 2003
Total depth of Installation	22.7 m	20.1 m	24.3 m
Thickness of Sand fill	4 m	4 m	5 m
Soft clay thickness	About 17 m	About 16 m	About 19.3 m
Other installation Remarks	Jacking in Sand (10s)		1.5 m
Duration (& Unit Rate) of insertion	NORMAL	FAST	NORMAL
(excluding jacking through dense sand	19 sec (0.83 sec/m)	7 sec (0.35 sec/m)	16.6 sec (1.37 m/sec)
Waiting Time at maximum depth of penetration	NORMAL	SHORT	LONG
	(3 sec)	(4 sec)	(30 sec)
Duration (& Rate) of Withdrawal	NORMAL	FAST	NORMAL
	17 sec (0.75 sec/m)	7 sec (0.35 sec/m)	17.9 sec (1.36 m/sec)
Anchored Soil	Reddish clay	Reddish brown clay	Reddish brown clay
Tension at 300 mm from toe	300 N	1278 N	7.2 ***251 N***
Tension at 600 mm from toe	320 N	Damaged	7.3 ***142 N***
Predicted maximum tensile force in filter	480 N	1278 N	377 N

The practical problems arise in relation to the PVD properties, installation machine characteristics, and the soil properties, modified during and after installation. PVD properties have been addressed by laboratory testing such as ASTM and Karunaratne and Chew (2000). It has been shown that a PVD brought for laboratory testing should be in a vertical position and embedded in soft clay with a minimum length of 1 m under simulated lateral pressure conditions (Karunaratne and Chew, 2000) to simulate the field environment. Figure 7.2 shows, for example, the effect of clay thickness from 0 mm to 40 mm on the axial flow rate. Figure 7.3 shows the effect of lateral pressure and the clay packing on the axial flow rate of a PVD.

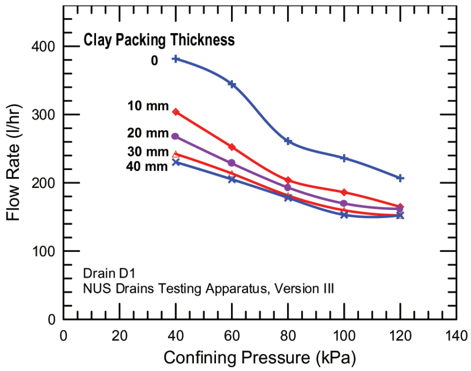

Figure 7.2. Effect of clay packing and axial compression on flow rate.

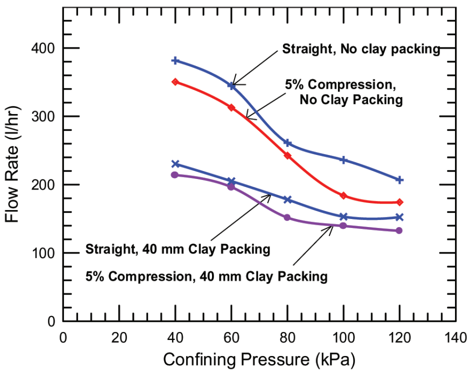

Figure 7.3. Clay thickness on flow rate.

Field behavior of PVD should be established to verify its effective (axial) discharge (flow) capacity with the lateral pressure and the hydraulic gradient likely to exist in the field. A typical set of discharge characteristics is shown in Figure 7.4 which demonstrates the parametric influence of the applied hydraulic gradient, and imposed effective lateral

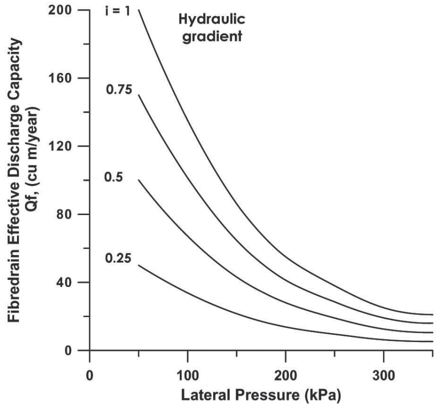

Figure 7.4. Discharge flow characteristics of a typical PVD (After Lee et al. 2003).

pressure. The operating hydraulic gradient in the field shortly after PVD installation would be high and decreasing with time. At shallow depths the effective lateral pressure on PVD is smaller and vice versa. Hence at a relatively short time after PVD installation, close to the upper surface drainage boundary, the PVD discharge capacity would be higher than at longer time or greater depths.

Installation considerations (Karunaratne et al. 2003, Cortlever, 2002) such as field PVD stress (see above for installation stress), shoe design, joints etc are important factors for a given project.

Back analysis of pilot test areas and large field projects (Choa et al., 2001) leads to an equivalent Ch of the clay soil that has been operating in the given conditions of a PVD-soil system. This is invariably different from the intrinsic coefficient of consolidation C_r of the soil. The latter, obtained with utmost care in exercising high quality sampling and improved testing techniques, is unlikely to occur due to poor field behavior of PVD, and the disturbance in the clay due to installation.

Hansbo (1981), showed that the average degree of consolidation $\overline{U}$ of a clay deposit under surcharge accelerated by PVD can be approximately expressed by

$$\overline{U} = 1 - e^{\left[\frac{-8 \cdot c_h \cdot t}{D^2 \cdot \mu}\right]} \tag{7.1}$$

where c_h = effective coefficient of consolidation with horizontal flow, D = influence diameter of a vertical drain, t = time, $\mu \cong \ln(D/d) - {}^3/_4$, and d = equivalent drain diameter = $(a + b)/2$, where a and b are the width and the thickness of the PVD cross section respectively. The average degree of consolidation $\overline{U}$ may be taken as the ratio of current settlement s to the final settlement s_f, i.e.,

$$\overline{U} = s/s_f \tag{7.2}$$

Assuming that all parameters in Equation 7.1 are relatively constant, differentiation of Equation 7.2 with respect to time in view of Equation 7.1 gives the rate of settlement as

$$\frac{\partial s}{\partial t} = s_f \cdot \frac{8 \cdot c_h}{D^2 \cdot \mu} \cdot e^{-\left[\frac{8 \cdot c_h \cdot t}{D^2 \cdot \mu}\right]} \tag{7.3}$$

The maximum rate of settlement occurs at the early stage of consolidation; taking $t = 0$, Equation 7.3 yields

$$\left(\frac{\partial s}{\partial t}\right)_{t=0} = \dot{s}_0 = \frac{8 \cdot s_f \cdot c_h}{D^2 \cdot \mu} \tag{7.4}$$

where $\dot{s}_0$ is the initial ground settlement rate. If $A = \pi \cdot D^2/4$ denotes the influence area of a PVD, then the required discharge flow rate Q_r is estimated by

$$Q_r = A \cdot \dot{s}_0 = 2 \cdot \pi \cdot s_f \cdot c_h/\mu \tag{7.5}$$

Figure 7.5 shows the variation of Q_r against s_f for various c_h and D/d values based on Equation 7.5 assuming $d = 0.055$ m. It can be seen that the larger the final settlement s_f, which depends on the thickness and compressibility of the clay deposit, and c_h, which is characterized by the compressibility and the hydraulic conductivity of the clay as well as the performance of the PVD, the larger is the required Q_r. It should be noted that reducing D leads to the increase of Q_r, contrary to the common belief, due mainly to the rapid rate of required discharge as $t \rightarrow 0$ when other conditions are unchanged.

The design should therefore be based on the PVD characteristics to yield the 'flow capacity' at the prevailing lateral pressure and the hydraulic gradient, to be matched with the 'required capacity' from the viewpoint of the rate of volumetric compression of the soil, which is based on the soil compressibility characteristics and PVD spacing in the soil.

Therefore, a reliable PVD, approved with realistic PVD testing, incorporating pertinent soil properties and the applied loading conditions will yield the initial settlement rates that would indicate the maximum required axial discharge capacity for a given project. If at $t \rightarrow 0$ the PVD satisfies the above criteria then long term effects will not be critical unless additional surcharging is contemplated. PVD core or the filter damage at the time of installation might be quite important because any impediment may restrict the capacity when the maximum flow at $t \rightarrow 0$ is required to operate. Buckling, durability of PVD components and the chemical and biological clogging through the filter cover may be important under relevant special circumstances. Factors such as deformation and mechanical clogging of PVDs core and filter during the consolidation process, some of which are not captured adequately in many designs, underline the PVD field behavior. In addition, PVDs should possess adequate tensile strength to withstand stresses generated during installation in thick clay deposits.

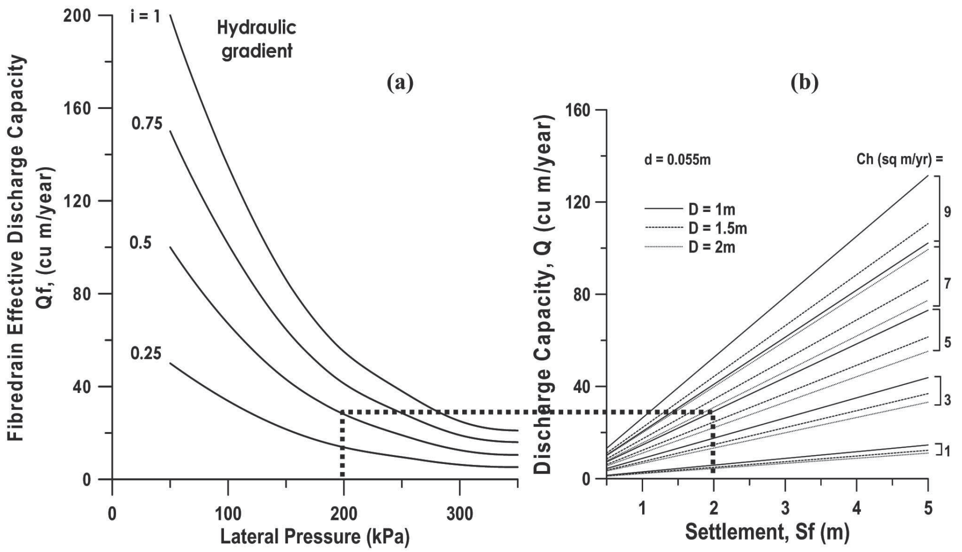

Figure 7.5. Effective discharge capacity of a PVD complying with the ground settlement, geometry of installation and PVD-clay system characteristics.

The effective discharge capacity of a PVD and the effective coefficient of consolidation C_h of a clay soil are functions of hydraulic conductivity and compressibility of the soil, as well as the field performance of the PVD, from storage and installation stages to subsequent field behavior. The design of soil improvement projects involving PVD should hence be based on the effective C_h, the final imposed settlement, the drain spacing as well as the lateral pressure on the drain.

8 ELECTRICAL VERTICAL DRAINS (EVD)

8.1 *Electo-osmosis with vertical drains*

Electro-osmosis (EO) causes movement of pore water in soft clay when an electrical potential difference is applied. Dewatering of such colloidal materials (of the order of few microns or less) by hydraulic means is difficult due to their extremely small hydraulic conductivity. Conductive polymer electrodes supplying DC at voltage gradients of 0.1 to 1 V/cm were used in the shape of channelized sheets to enforce an electric field. Resulting ionic and other electrical phenomena (Mitchell, 1991) cause the shear strength of the clay to increase at a much faster rate than prefabricated vertical drains (PVDs) (Karunaratne et al., 2002; Chew et al., 2004).

Using laboratory electro-osmotic cells, simple inter-relationships amongst the applied voltage, the current passing through the system resistance, ionic influence in the soils etc. can be studied. For instance, when a relatively small constant voltage gradient is applied across a soft clay bed, with parallel plate electrodes in a rectangular tank, simulating a rectilinear flow of current through the clay, the current flow seems to decrease with time. If the voltage gradient is increased incrementally, at some critical value the current flow through the clay appears to increase displaying an effective electro-osmosis process. At a subsequent point in time, the current begins to decrease due to the build-up of the system resistance comprising the resistance at the electrodes, soil-electrode interfaces and the soil itself subject to osmosis. Higher voltage gradients sometimes do not seem to contribute to the effective electro-osmosis as the current begins to fall. Figure 8.1 shows how the incremental voltages from 5V to 30V (Kuma, 2005) build up current in soft marine clay. Conductive polymer electrodes made up from electric vertical drains (EVDs) are used in these tests. Three different EVD versions referred to in the figure have varying conductivities enhanced by embedded copper strips.

Figure 8.2 shows the passage of current during the 2nd, 3rd and 4th voltage increment stages. The current in each EVD type responds to a 5V raise in Stage 2 with a sharp rise followed by a continuous drop. However, in Stage 3, the voltage raise

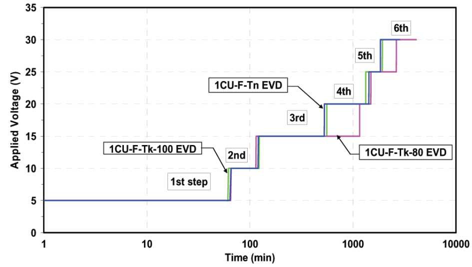

Figure 8.1. Voltage and current through the soil (After Kuma, 2005).

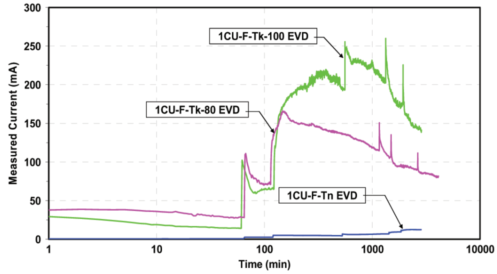

Figure 8.2. Current flow with time at constant voltage (After Kuma, 2005).

accompanied a sharp rise in current followed by a gradual rise to a peak in 1CU-F-Tk-100 EVD, while the other two EVDs continued to fall in current after the initially sharp rise. The electro-osmosis is effective where there is gradual rise in current. It would therefore be useful to elicit whether the voltage applied across the electrodes is of the correct magnitude.

A large percentage of system resistance occurs, as shown in Figure 8.3, at the soil-electrode interface partly due to the drying of the clay at the anode as the moisture moves towards the cathode. The most effective dewatering occurred at voltage gradients of 0.3 to 0.5 V/cm. Figure 8.4 shows the extent of dewatering in laboratory scale models.

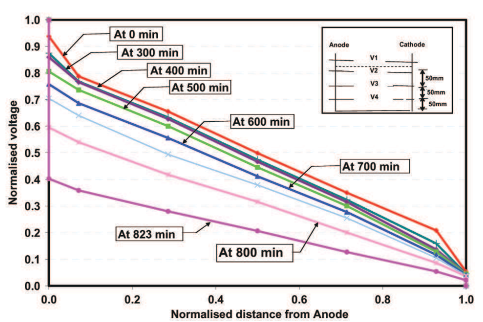

Figure 8.3. Voltage drop at the electrodes in a laboratory model After Kuma, 2005.

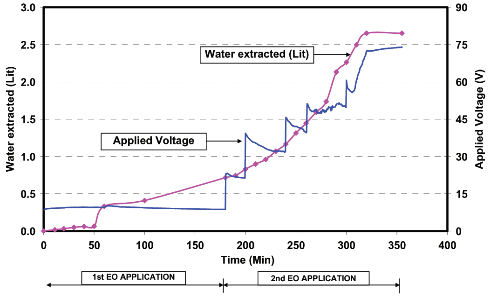

Figure 8.4. Dewatering rates in a laboratory scale model (After Kuma, 2005).

Currently this research is extended towards the development of innovative Electro Osmotic Dewatering (EOD) methods coupled with the mechanical pressure application so that sludge that contains high degree of moisture with a small to medium range hydraulic conductivity could be dewatered rapidly. Figure 8.5 shows a small scale laboratory model built with a central cathode and a peripheral anode both composed of conductive polymer fabric. Central cathode is further assisted by a perforated shaft that is also rotatable.

A negative (suction) or positive pressure can also be applied forcing the excess water to drain out under suction or surcharge effect making the electro osmosis process to be coupled with mechanical pressure application. Based on the preliminary tests conducted with soft clay this apparatus looks quite promising. Accordingly, a large scale pressurized chamber is currently being designed for above application. The potential of such research is very encouraging due to continual production of large volumes of dredged material from marine environment and sludge from high-tech industry that needs safe, efficient and cost-effective methods for reutilization of the waste products.

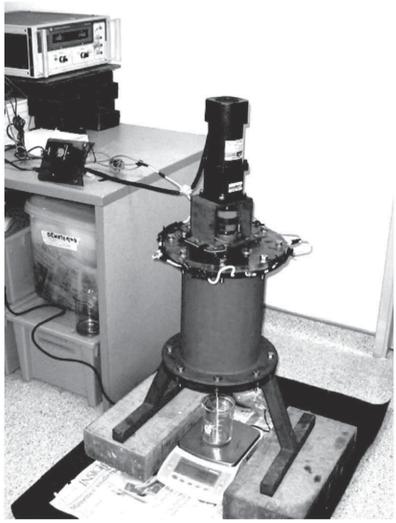

Figure 8.5. Laboratory dewatering model based on EO.

Different EVDs were fabricated with varying degrees of conductivity for laboratory testing as detailed in Table 8.1. These were then installed in soft marine clay of twice liquid limit placed in a rectangular glass tank with appropriate instrumentation for electrical property, pore pressure and temperature measurement.

If EVDs shown in Table 8.1 respond to EO at a particular voltage, as discussed in Figure 8.1, the

Table 8.1. EVDs of different conductivities for laboratory tests.

EVD ID	Core Type	Filter Type	Embedded material Type, Shape & Number	EVD width (mm) and Inclusion dimensions	Resistivity (Ω.m)
Plain	Conductive polymer	NW filter fabric	Nil	100	32.82
1 SS			1SS wire	100	29.91
2 SS			2SS wires	100	26.88
1 CU			1 Copper wire	100	25.74
2 CU			2 Copper wires	100	17.90
3 CU			3 Copper wires	100	10.26
1 CU F-Tn			1 Copper foil	100, 100 μm thick 10 mm wide foil	19.91
1 CU-F-Tk-80			1 Copper foil	80, 260 μm thick 10 mm wide foil	10.02
1 CU-F-Tk-100			1 Copper foil	100, 260 μm thick 10 mm wide foil	9.83
Plain-Q		Nil	Nil	100, w/o filter	2.90
2 CU-Q		Nil	2 Copper foils	100, 20 mm wide 2 mm thick foils w/o filter	1.96
8 Cu-Q		Nil	8 Copper foils	300, 20 mm wide 2 mm thick foils w/o filter	0.92
Aluminam plate	Metal	Nil	Nil	160, 2 mm thick w/o filter	2 E-06
Carbon Fabric	Nil	Conductive carbon fabric	Nil	100 w/o filter	2.6 E-04

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cumulative electrical energy per unit volume passed through the clay can be computed together with the electrical resistance of the system per unit volume, as summarised in Table 8.2.

Table 8.2. Laboratory EVD tests.

Serial No and EVD Type	At peak EO		Cumulative Energy (kWh/m^3)	Reduction in water content	
	Voltage (V)	Current (mA)		Anode	Cathode
1. Plain	30	2.5	0.09	9.88	8.78
2. 1SS	30	39	1.93	8.24	6.74
3. 2SS	30	104	3.02	14.89	14.56
4. 1CU	20	63	2.59	11.25	6.22
5. 2CU	15	166	0.07	12.61	10.85
6. 3CU	15	13	0.30	14.11	8.46
7. 1CU F-Tn	30	166	0.73	4.40	2.67
8. 1CU-F-Tk-80	15	220	0.20	12.59	3.44
9. 1CU-F-Tk-100	15	393	1.20	15.78	9.23
10. 1CU-F-Tk-100+Surcharge	15	400	4.29	19.47	15.76
11. 3 CU + Surcharge	20		14.29	18.71	10.12

2 SS: Two stainless steel wires; 2 Cu: Two copper wires; 3 Cu: Three copper wires

Reference to Table 8.1 and Table 8.2, there should exist a unique relation between the voltage applied (V), current (I) passing through and the resistance (R) of the soil-electrode system. In addition, the effective current which would move the ions in the soil would depend on a minimum voltage gradient applied. This current and hence, the applied electric energy, will enable to energize the ions to move towards the electrodes. When the effective current is applied it would not only enable maximum ionic movement but also alter the energy levels and the concentration of the ions.

Within a given time (t) the electrical energy (E) passing through the system can be formulated by

$$E = V \cdot I \cdot t \qquad (8.1)$$

Using Ohm's law in a DC circuit

$$V = I \cdot R \qquad (8.2)$$

leads Eq 1 to

$$E = V^2 \cdot t/R \qquad (8.3)$$

Taking the logarithm of the components, Eq. 3 becomes

$$\ln E = 2 \ln V + \ln t - \ln R \qquad (8.4)$$

When V is kept constant, as followed in Table 8.1 for most tests, Eq. 8.4 simplifies to

$$\ln E = \ln t - \ln R + \text{Constant} \qquad (8.5)$$

Figure 8.6 shows the relationship of ln(E) vs. ln(R) for the cases considered in Table 8.2. The three straight lines represent the relationship between E and R for voltages 15, 20 and 30V, which were kept constant

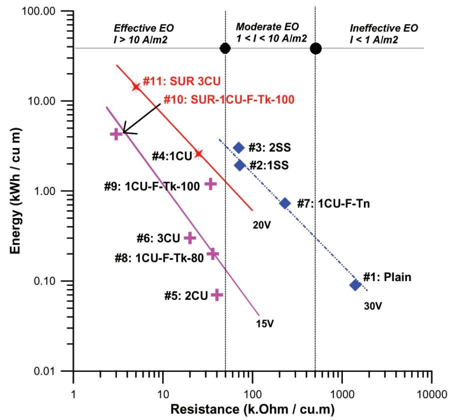

Figure 8.6. Energy vs System resistance (Adopted from Kuma, 2005).

allowing the current to vary as the system was subjected to EO and change in system resistance.

Higher voltages reflect large system resistances while the lower voltages are associated with small system resistances part of which comes from electrodes. It is also interesting to find that EVD with embedded steel wires yielded more resistance, at operating at 30V, than EVD with embedded copper wires and thick foils, which operated at relatively smaller voltages: 15V and 20V.

It can also be compared from Figure 8.6 that the system resistance obviously reduces due to densification under surcharge as the #11:SUR 3CU and #10:SUR-1CU-f-Tk-100 have moved further to the left in the more effective region from the original positions without surcharge.

The value of current density for each and every test series is also indicated in Figure 8.6. Three regions can be easily identified based on the effectiveness of EO:

- Effective EO treatment region where I/Area > 10 A/m^2; resistance < 50 $k\Omega/m^3$.
- Moderate EO treatment region where I/Area = 1 to 10 A/m^2; resistance between 50 and 500 $k\Omega/m^3$.
- In-effective EO treatment region where I/Area < 1 A/m^2; resistance > 500 $k\Omega/m^3$.

It can therefore be stated that an effective system would be characterised by a system resistance of less than 50 $k\Omega/m^3$ which entails the use of carbon fabric or copper embedded EVD. Suitable spacing and geometric orientations will be necessary to accommodate the above in the field.

8.2 *Field Trial*

An extensive field trail was conducted on Tuas View reclamation site in Jurong, Singapore, to study the performance of electrically conductive vertical drains (EVDs) in improving the soft soil in-situ (Chew, Karunaratne, Kuma et al., 2003). The site consisted of 12 m to 18.7 m of sand fill underlain by 8 m to 10 m of soft marine clay. Special attention was paid on the conductivity of different types of installed EVDs using conventional PVD installation rigs. EVDs ranged from conductive polymer drains with similar cross-sections to PVDs to those whose conductivity was enhanced with steel and copper wires embedded in the EVD, as well as copper wires attached externally to the EVD.

The electro-osmotic treatment of soft clay was conducted in two plots each of 50 m × 50 m in extent. In both plots, the average vane shear strength of soft clay prior to electrification was determined as 20 to 30 kPa being depth dependent. In Plot 1, EVDs consisting of a conductive polymer channelized strip within a filter sleeve were installed in a triangular pattern at a spacing of 1.6 m. In Plot 2, where EVDs were installed on a 1.2 m square grid, the area was subdivided as shown in Figure 8.7 for installation of EVDs with different conductivities. In specific subplots, voltage, current and voltage reversal patterns were varied as part of the investigation. Ground was monitored with especially designed probes for voltageage distribution in the clay with depth. The current in the wiring system was also noted against voltage and time in different circuits, following

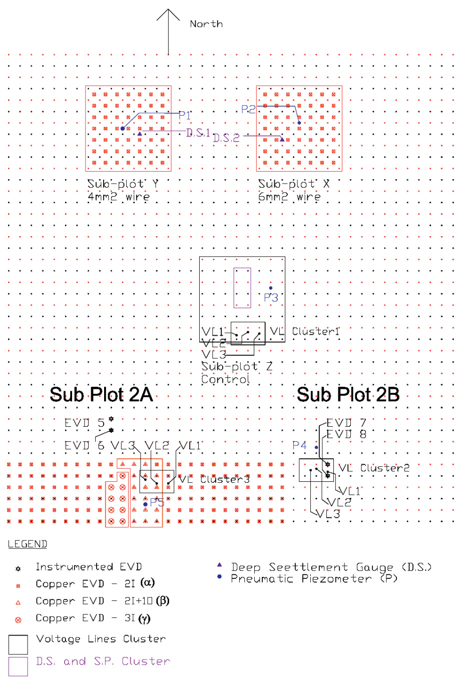

Figure 8.7. Details of Plot 2 and sub-plots.

Kirchoff's rules. Pore pressure was measured at a few selected points; in-situ field vane was used for identifying the change in shear strength in the soft clay.

This EVD field trial presented an opportunity for evaluating the field performance of electro-osmotic treatment of deep seated soft clay of the order of 20 m to 30 m with conductive polymers, compared with successful field work reported in the past by Casagrande (1949), Chappell and Burton (1975), and Lo et al. (1991b) who used copper rods, tubes or sheets and even expendable steel as electrodes.

Many modified versions of EVD with respect to the electrical conductivity were installed so as to study a variety of conditions such as: high water table, surcharge load, magnitude of applied current and voltage, salt water in the pore water, effect of the shoe on the conductivity. Instrumentation used for the project included the standard settlement and pore pressure measurement devices; but they were complimented by electrical measurement devices such as voltages, current and resistance in depth of the clay. Table 8.3 summarizes the details of subplots in the field trial. Each subplot was designed mainly to study variable parameters which included current, voltage gradient, conductive polymer effectiveness in EVD in respect of conductivity, enhancement mechanisms of EVD with copper wires, attempts to inject current in the clay bypassing sand layer and the elimination of metallic shoe effect.

8.2.1 *Subplot 2A*

EVDs in Subplot 2A were composed of embedded 2 and 3-copper wires in conductive polymer. A cluster of three voltage probes were installed vertically at three different locations, 400 mm, and two at 800 m but in two different orientations from the cathode EVD, with provision for voltage measurement in the soil at depths of 6 m, 12 m and 21 m from the ground surface as shown in Figure 8.7. The 21 m depth probe was located in the middle of the soft marine clay. Figure 8.8 illustrates the voltages measured by the Voltage Probe, as a profile of the ground potential.

Figure 8.9 shows the development of voltage build up around an operating cathode as the voltage reversal was effected on 28 June (at 12.30 pm) and on 1 July at 11.20 am. With progress of electro-osmosis, the voltage potential appears to collapse but rejuvenates with reversed polarity.

8.2.2 *Vane shear strength*

The results of the field vane shear test and percentage change in undrained shear strength, as shown in Fig. 8.10 indicate a large shear strength increase especially in the upper half of clay layer in Sub-plot 2A. Below 27.0 m the closer proximity to the stiff clayey silt, which had lower water content, is apparent.

In addition, 300 kPa overburden pressure from 18.7 m of reclamation sand fill on the soft clay may

Table 8.3. Details of sub-plots.

Subplot (Area)	Electrode type	Gross Voltage (V)	Gross current (A)	Maximum Power (kWh/m^3)	Gross Resistance (Ω)	Rate of pore pressure drop (kPa/day)
X (10 m × 10 m) at 0.6 m spacing	2SS + 6 mm^2 external Cu wire exposed in clay only	14-80	46-868 170-220 184-243	0.736	0.29	2.4
Y (10 m × 10 m) at 0.6 m spacing	2 SS + 4 mm^2 external Cu wire exposed in clay and sand	2-42	68-340	0.222	0.2	0.3
2A (25 m × 8 m) on 1.2 m square grid	2 Cu & 3Cu wire embedded	15-74	23-457	1.1	2.02	18
2B (10 m × 10 m) on 1.2 m square grid	2SS embedded	74-77	51-121	0.139	0.5-1.5	0.25
Plot 1 (50 m × 50 m)	2SS	23-71	110-283	0.98	0.22-0.69	1.3

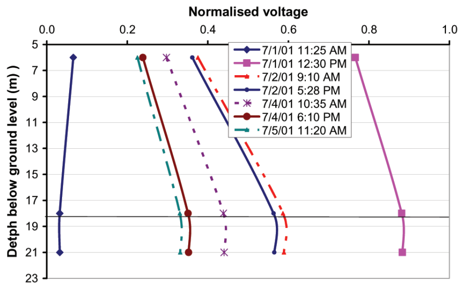

Figure 8.8. Voltage Line 1 Readings with Depth, after reversal of voltage at 11:20 am on 7/1/01.

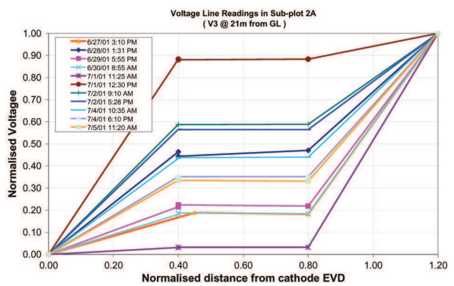

Figure 8.9. Voltage Line 3 Readings at 21 m depth showing the build up of the voltage potential with distance from the cathode at varying time, before and after reversal of voltage at 12.20 pm on June 28 and at 11:20 am on 7/1/01.

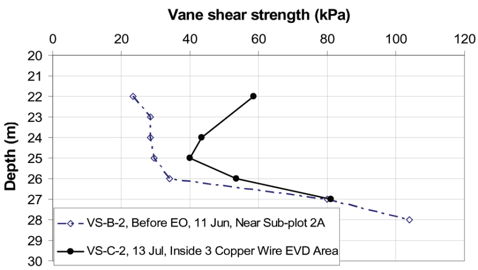

Figure 8.10. Field vane shear strength test results of sub-plot 2A.

have a slight "depth effect" as suggested by Lo et al. (1991). The substantial difference between EO treated and untreated clay indicates that EO caused greater improvement in the shear strength than conventional treatment by PVD action.

The increase in undrained shear strength of soft marine clay with consolidation for Singapore marine clay follows a Cu/P′ of 0.26–0.28, where Cu is the undrained shear strength and P′ is the effective consolidation pressure. If EVDs were to perform as a hydraulic PVD installed at the same 1.2 m spacing, the undrained shear strength would increase with time as shown with the broken line in Fig. 8.11. The vane shear strength before and after EO treatment yielded a Cu variation as shown by the solid curve, which was achieved in 13 days. The time taken by the PVD for the equivalent gain in strength would have been about 130 days illustrating a ten-fold reduction in time to achieve the same strength in soft marine clay. It should be noted that combined EO and PVD effect have been taken following Carillo's (1942) concept and a conservative parabolic variation was assumed between the cathode and the vane point, which was at mid-point between the anode and cathode.

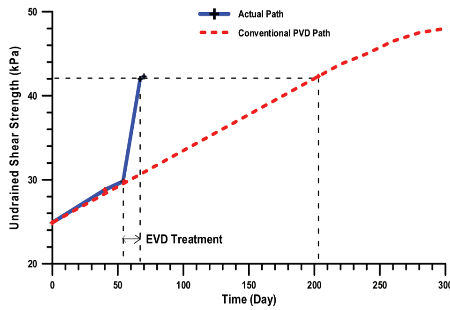

Figure 8.11. Undrained strength gain with PVD and EVD.

It can be observed that conductive polymers brought in to the market by the micro-electronic industry needed only a small power carrying capacity. Extension of the conductive polymer has recently been enhanced with modified polymer types and inclusion of additives. Conductive polymers are generally classified as those materials with surface resistivities around 100 ohms/m^2. For highly conducting thermoplastics carbon or stainless steel fibers are often necessary. EVD is a natural extension of this technology to the ground improvement work. If EVDs with capabilities of passing current > 10 A/m^2, voltages of up to 70V can be installed with conventional PVD installation equipment, instead of surcharge, DC power can be supplied to improve the soft clay much faster. Field trial has indicated a 10 fold increase in the rate of undrained shear strength.

8.3 *Dewatering of soil contaminated with heavy metals*

Industrial sludge usually contains fines with high water content. They also contain heavy metal components mixed with the fines which are also ionisable within the soil. Application of electro-osmosis brings in its potential of attracting these ions to the electrodes. A series of tests done with dredged soils having a high percentage of bentonite and heavy metals permitted attraction of Fe^{2+}, Al^{3+} and Pb^{2+} to the electrodes (Kuma, 2005). Higher quantum of anionic Ferrous and Aluminum migrated towards the cathode. From majority of the soil mass, except near the cathode, the percentage of ferrous reduction was about 40% to 55%. Within a very narrow width along cathode, migrated Ferrous accumulated by 45% to 95%. The percentage reduction in Aluminum content was found to be about 40% to 60%. However, anionic lead was precipitated at anode as the dissolved lead complex migrated under the electric field. As a result there was an accumulation of lead complex (about 30% to 70%) at anode electrode. Similar accumulation of heavy metals at the anode was reported by Wong et al. (1997) and Reddy/Chinthamredddy (2004). Potential of electro-osmotic technology is high in this type of industrial remediation work with EVD electrodes. The EO application in the circular chamber, shown in Figure 8.5, enabled reduction in water content by 40% and simultaneous extraction of heavy metals towards the electrodes. Application of negative or positive pressure coupled with DC electricity provides effective means of remediation of dredged clay soils.

8.4 *Conclusions*

Prefabricated drains effectiveness in soft ground improvement depends on the properties of the PVD, installation stresses, soil-PVD interactive Ch effective in the system. The consolidation rates of soft clay with PVD can be surpassed many folds by electric vertical drains (EVDs). The effectiveness of the latter depends on the EVD-soil system resistivity. Cumulative electric energy is strongly correlated with the system resistance. EVDs additionally provide electro-kinetic attraction of ions to the electrodes enabling effective remediation work and dewatering of industrial waste.

8.5 *Acknowledgment*

The cooperation and assistance given in the laboratory and the field by Lim Leong Hong, Toh Mei Ling and Joshua V M Kuma, while working for higher degrees under the author's co-supervision at the National University of Singapore are greatly appreciated. Jurong Town Corporation, Singapore and RasWill Representative Pte Ltd, Singapore for the much needed cooperation and support in the successful completion of the field trial reported in this paper are also gratefully acknowledged. Thanks are also due to Dr. Chew Soon Hoe for close working relationship on several innovative projects.

9 GEOSYNTHETIC IMPERVIOUS LININGS

9.1 *Geomembrane lining systems*

9.1.1 *Introduction*

More than 40 years after they were first used, geomembranes have now become solutions in their own right for the watertightness of most hydraulic structures: dams, ponds, canals… in which they can replace older solutions such as compacted clay, cement concrete, asphalt concrete or grouting.

The use of geomembranes in hydraulic structures is increasingly widespread and we can imagine, given growing demand for water worldwide, that this increase is likely to continue.

In this paper, we propose to present, through a number of examples, some lessons that have been learned from experience on the different types of structures concerned at the successive stages in projects: design, testing, construction, inspection. Recommendations will be given and regulatory issues will also be addressed.

9.1.2 *Geomembranes in dams*

Among the different types of structures, dams have a particular status due to their impact on the environment and on safety, meaning that they must be considered with the greatest care. The publications of the International Commission on Large Dams (ICOLD) on geomembranes are a demonstration of this with a first Bulletin published in 1981 (n° 38) and a second one in 1991 (n° 78).

The use of geomembranes in dams first developed in Europe, where the first case mentioned in international references concerns the Contrada Sabetta dam in Italy, lined with a polyisobutylene geomembrane covered with concrete slabs in 1959.

Tests conducted after 39 years in service showed satisfactory behaviour of the geomembrane itself (Cazzuffi, 1999).

In 1993, at the first European Large Dams Conference (Chambéry, France), it was decided to create a European working group on the use of geomembranes in dams to collect feedback on experience through a survey and the creation of a database, and to improve knowledge of this technique.

In 1999, the writing-up of a new bulletin was officially entrusted to the group, extended on this occasion, by the ICOLD Committee on Materials for Fill Dams (the committee that had been in charge of the two previous bulletins: 38 and 78). A more detailed presentation of "the new ICOLD Bulletin on impervious geomembranes for dams" is being made at this Conference by A. Scuero et al. (2006), the coordinator of the group.

The aim here is therefore to give a brief summary of this new bulletin and of technical progress in the field of geomembranes for dams. First of all, it is worth noting that over the period between the 3 bulletins:

- the height of the structures concerned has increased;
- all types of dams have now come to be concerned;
- the number of structures in consideration (and at the same time, the number of pages) has increased regularly.

This trend is a sign of the increasing maturity of the "geomembrane" technique and of the growing confidence owners, designers and builders of these structures have in it.

The new bulletin concerns the different types of dams and deals not only with the rehabilitation of old dams (see Fig. 9.1), but also the construction of new ones. The database contains information on 236 dams: 161 fill dams, 43 concrete and masonry dams and 32 roller compacted concrete (RCC) dams.

Figure 9.1. Example of the rehabilitation of the Chambon dam (Lefranc & al., 2002).

The geomembranes under consideration here are factory prefabricated polymeric and bituminous geomembranes. Sealing operations carried out on site by impregnation of geotextiles or spraying are closer to the family of resins and do not fall within the scope of geomembrane techniques; they are little used and are not dealt with in the bulletin. Geosynthetic Clay Liners (GCL) are another type of geosynthetic barrier; they have not been used on dams (no data from the survey) and are not examined in the bulletin.

The new ICOLD bulletin aims in particular to help the designers of Geomembrane Liner Systems (GLS) for a new dam or a structure to be rehabilitated. When choosing the geomembrane, the reader will find information on geosynthetics in general in the first 3 chapters, on geomembranes in particular and on the stresses to which they are subjected; the 4 chapters dedicated to the different types of dams and to special applications, will provide a large number of examples; finally, the last 2 chapters concern quality control and contracts. The database included in the annexes is also a source of much information for projects.

90% of the geomembranes used are polymeric geomembranes (63% of the total with PVC-P) and 10% are bituminous geomembranes. Bituminous membranes have been used on fill dams, essentially in France (Gautier et al., 2002).

The points to be taken into account by the designer are detailed in the bulletin; here, we will look only at a few points which are discussed and for which the database shows that there is not just one single solution, such as, for example:

- the position of the geomembrane in the case of new fill dams (upstream face or internal position); the use of geomembranes in the internal position is rare at present (10% of fill dams, essentially in China); this position may not have been sufficiently studied to date, notably regarding its implementation;
- whether to cover the geomembrane with a protective layer or not, when it is positioned on the upstream face; this second point will be dealt with on the basis of two examples in the chapter on ponds.

Concerning design, it is worth pointing out:

- on the one hand, that the design must take account of the Geomembrane Lining System (GLS) as a whole (sub-base, geomembrane, cover layer if there is one, anchorage and joints);
- on the other hand, that in the field of dams, although analytical methods and numerical models are to be used, feedback on experience and large-scale trials remain necessary; it is to illustrate this point that two examples of trials, in the laboratory and on site, are given below.

These examples concern the puncturing resistance of the geomembranes which is most certainly, along with durability, one of the main preoccupations of designers.

The first example concerns the study of the BOVILA dam (Sembenelli et al., 1998). Two types of tests were conducted for this fill dam of a height of 80 metres, sealed with a geomembrane over the top 57 metres, to check the resistance of the selected geocomposite (a 3 mm PVC geomembrane coupled to a 700 g/m^2 geotextile) under the hydrostatic stress in service conditions. Large scale hydrostatic pressure tests were carried out on the granular sub-base planned for use on the structure (a 15-25 mm low-cement concrete); the maximum pressure applied was 900 kPa, equivalent to the maximum pressure applied on the structure itself plus 50%; 3 loading-unloading cycles were applied for each test.

The other type of test aimed to simulate a collapse in the granular sub-base. A cylindrical hole with a diameter of 0.3 m and a depth of 0.2 m was made in the sub-base under the geocomposite and the same hydrostatic pressure was applied.

The second example concerns the Selvet dam (Girard et al., 1998); in this case, it is the stress to which the geomembrane is subjected during installation of the cover layer that was stimulated. In fact, when the cover is a granular layer installed using machinery that moves around on top of the layer, the stresses applied during installation are very great. In this case, test areas with the materials being envisaged for the site were used as part of the geosynthetic complex design process. A guide to using such test areas has in fact been written up by the CFG (French Chapter of the IGS, 2001).

The Selvet dam is an 18-metre high rockfill dam sealed with an elastomeric bitumen geomembrane placed on the upstream face and protected, especially against ice, by a layer of rockfill. Six Geomembrane lining systems (GLS) with different support and cover layers including gravels and geotextiles were tested; after installing the 6 GLS in the actual conditions planned for the construction of the dam, two samples of 1 m^2 were dug up for each GLS.

On the basis of visual observation and bi-axial tensile tests on the exhumed samples of geomembranes, the designer chose a GLS that was adapted to conditions on the site (small dam, use of the aggregate available on the site). The results of the bi-axial tensile tests were used to qualify the degree of damage of the geomembrane samples by defining a satisfaction index (ratio of the maximum displacement, measured at failure at the top of the swollen geomembrane, for samples taken from the 6 test areas to the same displacement for virgin samples (Fig. 9.2).

Thanks to the tests areas on site, it was concluded that the materials on the site were too aggressive as a sub-base and that a thick, puncture-resistant geocomposite and a 0-31.5 mm granular layer should be placed between the geomembrane and the upper layer of rockfill. This example illustrates the interest

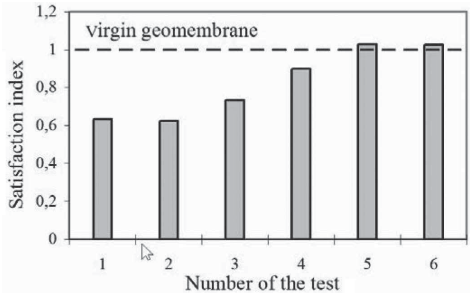

Figure 9.2. Summary of the bi-axial tensile tests for the 6 GLS (Girard & al., 1998).

of such test areas in the absence of laboratory tests that are enough representative of the stresses encountered.

These 2 examples illustrate only cases of fill dams, but it should not be forgotten that all types of dams are concerned (Scuero et al., 2006). To conclude, the very great heights of the different types of dams lined with geomembranes should be mentioned: 196 m for the Karahnjukar fill dam (Iceland, new dam built in 2005), 174 m for the Alpe Gera dam (gravity dam rehabilitated in 1994) and 188 m for the Miel 1 dam (new RCC dam, Colombia, 2002).

Recent publications give detailed descriptions of the studies, construction conditions and results obtained on different types of dams all over the world; among these publications, we will refer to the cases of the Kadamparai masonry dam (India, Sadagopan et al., 2005), and the RCC dams of Olivenhaim (USA, Tarbox et al., 2005) and Burnett (Australia, Neumaier, 2005).

9.1.3 *Geomembranes in ponds*

Geomembranes are very widely used for lining ponds. These ponds have a wide variety of uses: irrigation, drinking water, energy production, aquaculture, leisure activities, fire-fighting, artificial snow production, etc. As in dams, the structures concerned have widely varying sizes, ranging from irrigation ponds for a single farm to very large ponds such as the two examples presented hereafter.

The Afourer pumping station presented in detail by Fayoux et al. (2006) in this conference is located in Morocco; it completes an existing facility and is used for both electricity production and for irrigation. In that it offers two basins close to each other at a difference in height of 800 metres, the site is a good one for the installation of a pumped storage power plant. The capacity of each of the basins is 1 260 000 m^3 with a maximum embankment height of 18 metres; the slope of the inner face of the embankments does not exceed 1/3. These structures located in highly permeable limestone zones were sealed using a PVC membrane of a thickness of 1.5 mm; the liner was installed in 2003–2004 over a total surface area of around 330 000 m^2. The designer chose to protect this geomembrane by a covering structure, giving three reasons to justify this choice:

- the lifetime of the geomembrane is considerably increased, even if good-quality geomembranes have a lifetime of 20 to 30 years in the climate in question (Fayoux, 2005); the period during which the facility would be out of use to replace the geomembranes would have an enormous cost;
- protection against the wind would have required large anchorage systems for this type of large basin, thus reducing the economic interest of not installing a cover layer; construction of the anchorage would have had to be particularly careful to avoid altering the condition of the surface of the sub-base (risk of puncture), thus increasing its cost;
- protection against vandalism and passing animals.

Fayoux et al. (2005, 2006) give a detailed description of the test areas used to finalise the GLS (thicknesses and materials adopted for the bottom and slopes, installation procedures). The layer covering the geomembrane on the embankments is the following: non-woven needle-punched geotextile (500 g/m^2), 5/16 mm crushed aggregate (thickness = 0.20 m), 100/200 mm rockfill (thickness = 0.20 m). The installation of the cover structure is also presented in detail. In particular, we will mention:

- the upper geotextile was sewn to guarantee continuity during the installation of the upper granular layers and in service;
- with great care, it proved possible to install the granular layers with a hydraulic shovel working from the top of the embankment; this is generally not recommended, but was made possible by the low slope angle of the embankments (Fig. 9.3).

The second example is presented by Takimoto et al. (2002); they present the installation procedures and the behaviour of an EDPM (Ethylene-propylene Diene Monomer) geomembrane used to line the upper pond of the Yambaru pumped storage power plant (Japan) located on the edge of the Pacific Ocean and using seawater. Like the Afourer plant, this type of facility requires large ponds that are subject to particular stresses, in particular very rapid emptying/filling cycles and the effects of the wind. The Yambaru pond has the following main characteristics: volume stored = 560 000 m^3; embankment slope = 1/2.5; embankment height = 25 m; maximum embankment length = 74 m, with an intermediate berm; first studies in 1987 and facility commissioned in 1999. The pond liner system design process led to the following system being chosen: 0/20 mm gravel sub-base (transition-water and air drainage) of a thickness of 50 cm, non-woven, puncture-resistant geotextile (800 g/m^2) and EDPM geomembrane (2 mm); as the geomembrane is not covered, it is carefully anchored to partly-

Figure 9.3. On slope GLS testing for Afourer pond (Fayoux & al., 2005).

Figure 9.4. Construction of sheet connection structure (Takimoto & al., 2002).

prefabricated concrete blocks installed before the granular sub-base (Fig. 9.4). It should be noted that these choices were made on the basis of extensive specific preliminary studies.

For instance, the pre-selected geomembrane made specifically for this structure underwent exposure tests near the site. The concrete blocks used for anchoring and connecting the strips of geomembrane and for drainage thanks to two built-in conduits, are another essential part of the design of the structure; these are prefabricated, 2.00 m-long, U-shaped blocks placed in the sub-base in the direction of the slope at intervals of 8.50 m on the slopes and 17 m on the bottom. The strips of geomembrane prepared in the factory to these dimensions are assembled onto the blocks on site, with the joints placed inside the "U" which is then filled with concrete; this system is then covered by a strip of geomembrane stuck onto the main geomembranes using adhesive tape. This anchoring system was the subject of prior experimental studies consisting in inflating a geomembrane with its 4 edges fixed down and of a simulation using the finite elements method.

This structure was filled for the first time in 1998, the liner work having been completed in 1996. It should be mentioned that the pond was subjected to typhoons causing lifting of the geomembrane by up to 1.50 m. The monitoring system did not reveal any leaks through the geomembrane.

The authors mention the appearance on the geomembrane of algae and shellfish that are likely to pose problems of displacement of the controllers; they also report on tests on the geomembrane and comparisons with other sites on the basis of which they estimate a lifetime of 40 years for the geomembrane.

These two examples illustrate the two possible choices: covered or exposed geomembrane; on the basis of these two structures, we can review the advantages and drawbacks of the two solutions, either on dams or on ponds.

The "exposed geomembrane" solution presents the following advantages: cost generally lower (although the cost of an elaborate anchorage system should not be underestimated), quicker and easier installation, no risk of damage of the geomembrane by installing a cover layer, easy visual inspection and repairs, shorter intervention times (important in rehabilitation operations); the drawbacks are: lower durability of the geomembrane which is exposed in particular to UV radiation, risk of mechanical damage by vandalism, floating objects, falling objects, ice, mechanical effects of wind and waves, need for a more resistant sub-base.

The main advantage of the "covered geomembrane" solution is that the geomembrane is protected against the external factors mentioned above, thus giving it greater durability. This point has to be taken into account in the economical study of a project. On the other hand, consideration must be given to the risks of puncturing the geomembrane when installing the cover layer, to difficulties of access to the geomembrane in case of a leak (detection and repair) and to the cost that is generally higher. A cover layer is particularly interesting when the cost of suspending service is high, as in the case of Afourer (difficult to stop irrigation and/or electricity production).

This comparison shows that there is not one single solution that is best in all cases; for a given structure, the choice will lie with the designer who will be able to make use of the feedback collected together in the database of the future bulletin on geomembranes for dams. It can be noted that, for the fill dams listed in the bulletin, 39 are lined with an exposed geomembrane and 112 with a covered geomembrane. To help designers, a comparative table of the two solutions is supplied in the bulletin.

9.1.4 *Geomembranes in canals*

Geomembranes are also increasingly widely used in canals, whether for the transport of water (irrigation, drinking water, electricity production) or for navigation. As with dams, the size of the structures can vary enormously, from the metre-wide irrigation canal to enormous projects such as the Kimberley Canals in Australia mentioned by C. Kelsey (2005).

The need to transport water goes back a very long way, as is demonstrated by large structures such as the Roman aqueducts (Koerner, 2005). In this field, too, geomembranes are used for new structures and in the rehabilitation of old structures; they have now often become more interesting, technically and economically, than older solutions (clay, concrete, etc.). In the case of irrigation canals, the objective is clearly one of saving water; in this case, geomembranes are fully justified, as shown by the experiment of Swihart/Haynes (2002); based on a comparison of different liner systems in real conditions, the results of 10 years of observations and tests are presented. The estimation of durability confirms the better long-term behaviour of the "covered geomembrane" solution and the greater efficiency of geomembranes than concrete in reducing water losses in experimental conditions (Table 9.1).

Table 9.1. Comparison of different Geomembrane Lining Systems (Swihart/Haynes, 2002).

Lining	Estimated durability (years)	Effectiveness at seepage reduction	Benefit/cost Ratio
Concrete alone	40-60	70	3.0-3.5
Exposed geomembrane	10-25	90	1.9-3.2
Concrete covered geomembrane	40-60	95	3.5-3.7

It should be noted that the geomembranes concerned in these irrigation canals are less thick than those used on most other structures and that their durability is shorter as a result.

The recent publication of the ICID (International Commission on Irrigation and Drainage, 2004) confirms the growing use of geomembranes in irrigation channels. As for dams, this is an update of a previous manual (n° 108; 1990), taking account of the place that geomembranes have come to take; it shows that they would appear to offer better performance than traditional solutions (concrete, clay); it suggests that covering the geomembrane is desirable because irrigation channels cannot be protected from access by the public or animals, and require maintenance operations (removal of sediment, for example). This work describes the geosynthetics and covering structures, and deals with the issues of choosing the geomembrane, installing it, quality procedures and contracts. It is a general-purpose publication and designers will also need to study recommendations, in particular those from similar applications such as ponds and dams.

Water transport canals for hydroelectric power production are another field in which geomembranes are used. Strobl et al. (2002) and Schaefer et al. (2004) present a comparison between the traditional sealing methods (cement and asphalt concrete) and polymeric and bituminous geomembranes in the rehabilitation of canals lined with cement concrete. The main advantages of geomembranes are pointed out: time savings on installation, lower costs, no decrease in the size of the canal (thickness of the geomembrane compared with that of a layer of concrete), smoother surface allowing greater flow rates. The expected lifetime of the "geomembrane" solution is estimated to be higher than 30 years (experience value observed); the value given for the traditional solution is 50 years. In the cases presented, the geomembranes are exposed; detail is given of a study of the design of anchorage systems to avoid the geomembrane being pulled off in case of tearing caused by falling objects (a car is given as an example), based in particular on an experiment in a canal. Guidelines have just been published by the Technical University of Munich (2006), based in particular on the studies of Strobl et al. (2002) and Schaefer et al. (2004). An example of rehabilitation is given by Dos Santos Magalhaes et al. (2004), who describe the good behaviour of the canal, ten years after the installation of the geomembranes and evaluate the profit due to the greater flow rate allowed by the smoother surface of the geomembranes.

Navigable waterways are a means of transport that has been used for centuries but which has been in decline over the last few decades. The problems of pollution caused by road transport are now likely to restore these waterways to their place. Geosynthetics, and in particular geomembranes, can play an important role in this context; in this field, too, they offer a well-adapted solution that is much appreciated, as is shown in the example in France explained below.

In France, the greater part of the current network was designed in the late 19th century. A few structures of larger dimensions, better adapted to large-scale, more intense traffic, have been built since the 1950s and the construction of some large canals is also scheduled. The embankments of older canals were always built using the materials available on site after more or less careful selection. They are now showing signs of age, in particular leaks, failures and erosion phenomena. The services in charge of operating these relatively old waterways are therefore involved in canal repair and improvement operations. The traditional improvement techniques consist in building a vertical tightness screen inside the body of the canal embankment (sheet piling or grout curtain) or in installing a watertight surface coating (cement concrete or asphalt concrete). Over the last twenty years or so, the use of geomembrane liner systems has emerged and has tended to replace the above mentioned traditional methods.

Ten or so cases of rehabilitation by geomembrane are analysed in the two papers by Flaquet-Lacoux et al. (2004) and Poulain et al. (2000); in all the cases presented, the work was carried out after leaks were noticed on the outer face of the embankments, at their foot or at a certain distance from them. Apart from an increase in water requirements, these leaks give rise to risks of internal instability (backward erosion) and global instability (sliding). In most cases, the canal had an old concrete coating that no longer fulfilled its purpose and improvement work had been done of the embankments by grouting or the installation of sheet piling, but without lasting success. Feedback on the experience of the rehabilitation work described shows that the "geomembrane" technique gives satisfaction to users, with the leaks disappearing; in comparison with the vertical tightness solutions (sheet piling or grouting), the main constraint involved in the "geomembrane" solution concerns the need to drain the canal: this leads to very short installation times due to the needs of navigation.

In the most frequent situation, that of an old concrete lining, this former lining had to be destroyed in most cases given its poor condition (Figure 9.5). For these navigable canals, the geomembrane is covered by a layer protecting it against impacts by boats; in almost all cases, the cover is composed of concrete slabs cast on-site in formwork. In one case, the geomembrane is protected by concrete-coated rockfill.

But a few problems have also been noted; they do not bring into question the effectiveness of lining canals with geomembranes however, because they are due to the instability of the sub-base. It should therefore be remembered that the geomembrane does not have a mechanical role and that it requires a sufficiently strong, stable sub-base: an appropriate geotechnical study must therefore be conducted to check this stability (stability of the embankments, risks of subsidence in karstic zones, sub-surface runoff, groundwater, etc.) or to define the necessary reinforcement work.

Figure 9.5. View of an old concrete lining (Poulain & al., 2000).

This analysis of feedback on experience is part of a wider study of the use of geomembranes in navigable waterways with a view to rehabilitating old structures and building new ones; this study is devoted more particularly to the choice of the structure covering the geomembrane and to the examination of the stability of this structure on the embankments and the design of geosynthetics anchorage systems at the top of the embankments (Poulain et al., 2000).

9.1.5 *Durability*

Although the subject of this chapter is to give an overview of the uses of geomembranes in hydraulic structures, it would seem useful to provide designers with information on what is surely one of the main issues: durability. The references quoted before and others available in the literature, concerning laboratory studies, experiments and feedback, show that the durability of the geomembranes in the structures described previously reach 20 to 30 years or more for exposed geomembranes, as long as their formulation is adequate and the liner system as a whole (including the sub-base, drainage and anchorage) has been well designed, built and monitored. The expected lifetime of covered geomembranes protected from climate and mechanical aggression is much longer.

On this point, we can mention the recent publications of Koerner et al. (2003) and Hsuan et al. (2005), examining ageing and predicting geomembrane lifetimes on dams and hydraulic structures.

The presentation by Giroud (2005) relating 40 years of use of geosynthetics in dams, also provides information on this subject from feedback on experience; such is also the case of the article by Scuero et al. (2004). Royet et al. (2002) analyse the behaviour of 12 dams lined with PVC and bituminous geomembranes (observations and measurements of leakage flow).

Samples taken from dams and then subjected to tests complement visual observations and monitoring measurements and can provide results on the evolution of geomembranes placed in real conditions. The publications of Cazzuffi (1995, 1998) on canals and dams in Italy and of Girard et al. (2002) on ponds and dams with exposed and covered geomembranes illustrate this approach for PVC-P geomembranes. The second publication illustrate the interest of installing control zones to make it possible to take sample and to monitor the evolution of the geomembrane installed. These publications clearly show that if the formulation and the properties of the chosen geomembrane are adapted to the stresses encountered on the structure, its behaviour over time is satisfactory: this is the case of the geomembranes used for large structures; in contrast, cases of very rapid ageing have been observed on poorly formulated geomembranes on small structures.

9.1.6 *Other uses – Monitoring*

Other uses or conditions of use of geomembranes are worth mentioning; they are the following, each one being illustrated by an example:

- installation under water (Scuero et al., 2003),
- use in cold climates (Larson et al.),
- lining concrete reservoirs (Finley et al., 2004),
- floating covers (Sadlier et al., 2002),
- and geotubes (protections against flooding or for dewatering – Kim et al., 2005).

It should also be noted that monitoring of GLS, in particular using optic fibres, is developing (Schäfer et al., 2003).

9.1.7 *Standards*

Among the numerous standards existing all around the world concerning the geomembranes (physical, hydraulic and mechanical properties, durability, …), we want only mentioned two recent European Standards which specifies the relevant characteristics of geomembranes to be used for reservoirs and dams (EN 13361, 2004) and for canals (EN 13362, 2004); these standards specify also the appropriate tests methods to determine these characteristics, but do not give values. In particular, they enables the designers to define the characteristics applicable for a given project.

9.1.8 *Conclusion*

In dams, liner systems using geomembranes present good behaviour, as is shown by the results of the survey mentioned above; pathologies are observed, however, on ponds and canals of modest dimensions. This remark would appear to be linked to the fact that for dams and large structures involving significant safety issues, the designer and owner base themselves on technical criteria, while for modest structures, it is the financial factor, unfortunately, that often becomes dominating. One conclusion of the ICID publication (2004) could be mentioned here regarding the latter fact: *"It is unwise to pay too much, but it is worse to pay too little. When you pay too much, you lose a little money – that is all. When you pay too little, you sometimes lose everything because the thing you bought is incapable of doing the function it was bought to do."* Another difference between small and large works is that often less experienced people are concerned by small projects even though technical questions are similar.

As a last remark, it is important to stress that the designer must conceive the geomembrane system as a whole. Good performance depends just as much on the sub-base and covering layers that might be used, as on the geomembrane itself. In particular, the aspects of drainage under the geomembrane, protection against puncturing of the geomembrane and its anchorage to the sub-base and peripherals are key factors in the success of liner systems for water-tightness. A good installation, with a specific Quality Assurance Plan, very important for joints, is of course also necessary.

9.2 *Geosynthetic Clay Liners (GCL)*

9.2.1 *Applications*

"Clay Geosynthetic Barrier" is the official name in the European standardization for an impervious lining with a bentonite clay layer confined in between two sheets of geotextile. But much more often "Geosynthetic Clay Liner (GCL)" is used, therefore this term will be kept in the following.

GCL have been installed in many applications, predominantly in landfill and road construction (including runways). In hydraulic application it can be found as lining (initial or repair) of

- irrigation and retention ponds,
- irrigation canals,
- dykes (or levees).

It is still a very young technique, e.g. the Earth Manual (1998) does not even mention this lining alternative for canals.

With the applications listed above, the GCL is placed in the dry. The requirements to the fabric and the bentonite are similar to the application for landfill covers. This issue is treated among others in the keynote lecture of Kavazanjian et al. and will not be discussed here. The usual installation guidelines require that the bentonite must not have any contact to water until the system of lining and protection is completed.

A special hydraulic application of GCL is the use as an impermeable surface lining for dikes and levees. After several dike failures in Middle Europe, GCL were proposed for repair or improvement. It was proposed to line the water side of the dike with a GCL to hinder or at least reduce the percolation of water through the dike. But installing just a lining on the water side of the dike may not result in the desired effect. The main problem is the contact to an impervious soil layer at the waterside foot of the embankment. If the water is able to flow below the lining into the dike, the effect of the lining will be negligible. The seepage line in the embankment will reach nearly the same level as without the lining. The only advantage is gaining some time until this condition is reached, depending on the permeability of the embankment. If there is no soil layer with low hydraulic conductivity at the foot of the dike, additional measures are necessary, e.g. a seepage screen to a sufficient depth.

When installing a GCL, the overlap needs special treatment. Usually bentonite powder is spread in the overlap and the seam is sealed with bentonite paste. Such after-treatment is not possible when the GCL is to be placed in the wet, e.g. in a canal that cannot be closed and emptied. Then a lot more aspects have to be taken into account.

The first attempt of placing a GCL under water was undertaken in France 1994 in the reach of Niffer, the connection of the Grand Canal d'Alsace and the Rhine-Rhone-Canal (Walter 1996). For installation a completely new placement device had to be designed and to be built. For this application the GCL is covered by a gravel layer immediately after placement to avoid floating. An immediate surcharge is necessary since in the beginning air bubbles remain in between the clay minerals and in between the fibres that causes the GCL to float. To avoid any aftercare of the seam, the overlap is manufactured as a bentonite-filled nonwoven (top fabric of the lower GCL) and a woven (base fabric of the upper GCL). This way, no transmission flow in the overlap is possible. In the French placement process, the overlap is still the weak point because of the risk that the gravel needed to keep the GCL on the bottom may fall or roll in between the overlapping sheets.

A second installation in the wet was done in 1997, namely the lining of a navigation canal near Berlin in Germany. The clay liner had to be placed among waterborne traffic. A similar application started in 2000 in the Dortmund-Ems-Canal. Both installations have been and are still accompanied by extensive monitoring (Fleischer/Heibaum 2002).

The liner used in Germany consisted of a base woven geotextile, a sodium bentonite fill of 4200 g/m^2 and a cover nonwoven. To reduce the risk of leakage due to gravel in the overlap, a new solution for the necessary surcharge was tested. Instead of gravel a second geocomposite – a 'sandmat' with 8000 g/m^2 of sand in between two nonwovens was chosen to protect the GCL. GCL and sandmat are placed in one action. Both the GCL and the sandmat were rolled up together on a steel tube, but staggered 80 cm (Fig. 9.6). The placement has been done by a lattice trolley boom with a vertical lattice mast and a hydraulically driven spreader bar. That device enables installation to a depth of more than 20 m. For the second

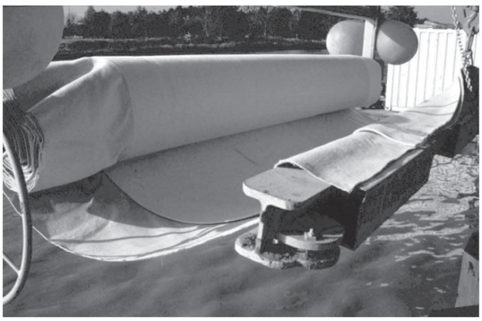

Figure 9.6. Staggered GCL and sandmat to be unrolled under water.

installation in the Dortmund-Ems-Canal sandmat and GCL were combined to one single geocomposite of three geotextiles and a layer of sand in between the upper two sheets and the bentonite layer in between the lower sheets. The system of (bottom up) woven base, bentonite layer, intermediate nonwoven, sand layer and top nonwoven was bonded by needlepunching.

The latest project, the lining of a drinking water canal in the Ukraine, put into action the lessons learned from the two above projects. This canal cannot be closed being the only water supply for the Donez Basin. To repair the leaking concrete lining, a new impervious layer was needed, but with limited thickness only, not to reduce significantly the channel's cross section. The solution chosen was to place a GCL upon the existing surface (under water) and to build a protection layer from prefabricated concrete slabs with spacers to hold them with a gap above the GCL and then to fill the gap with concrete.

9.2.2 *Requirements*

Proposing a new lining system for underwater installation means competing with the well-known systems for waterways, namely clay liners made of conditioned natural clay. As a result of the two pilot projects in Germany general requirements for the GCL have been established (EAO 2002). The main requirements are:

- A sufficiently low permeability. For an impervious lining, the maximum discharge through the layer is $2{,}5 \times 10^{-8}$ $m^3/s/m^2$. So for a GCL considering a water head of 5 m, one has to ask for a permittivity of 5×10^{-9} s^{-1}.
- A flexibility to guarantee that no leakage or weakening will develop due to deformations of the subsoil. Deformations of the subsoil are inevitable, since during the construction process the subsoil will be saturated and is only partially saturated afterwards.
- There has to be a sufficient resistance against impact forces. When riprap is dropped upon the GCL, the liner must not be perforated. It is not a question of damage of the geotextile, but the bentonite will be locally displaced, leading to locally increased permeability. For that reason armour stones have to be laid on the GCL with care. For the placement under water, the stones may be dropped from the water surface. Tests showed that the maximum sinking velocity causes no harm to the liner.
- An additional protection layer is necessary in the zone of fluctuating water level because GCL are not resistant against plant roots.
- High resistance against erosion under the conditions of the waterway is necessary to maintain the imperviousness. Neither high gradients nor dynamic hydraulic loading (unsteady and reversing flow) must erode the bentonite from the GCL.
- Particular care is necessary during the placement of the GCL. Special attention has to be put on the overlaps. No granular material at all must remain in between two overlapping GCL sheets. Otherwise a permanent leakage may be created. Also any folds in the GCL have to be avoided. The overlapping technique must guarantee imperviousness from the very beginning, since no after-treatment like after placing in the dry is possible

9.2.3 *Conclusion*

For GCL all the general statements given in 9.1.8 are valid as well, even though the systems are rather different. There is sufficient experience in hydraulic application of GCL as long as they are placed in the dry. Since in many cases, linings have to be installed under water, special requirements have to be fulfilled by a GCL when used instead of a traditional sealing material. First projects have been executed successfully and relevant recommendations are available.

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