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Contents

Contributor contact details

Editor

Professor R. W. Sarsby
Section Leader for Civil Engineering (SEBE)
University of Wolverhampton
Wulfruna Street
Wolverhampton WV1 1SB
UK

email: r.sarsby@wlv.ac.uk

Chapter 1

Dr Robert M. Koerner
H. L. Bowman Professor of Civil Engineering and Director
Geosynthetic Institute
Drexel University
Civil, Architectural and Environmental Engineering Department
3141 Chestnut Street
Philadelphia
PA 19104
USA

email: robert.koerner@coe.drexel.edu

Chapter 2

Dr Steven W. Perkins
Department of Civil Engineering
Montana State University
219 Cobleigh Hall
Bozeman
MT 59717
USA

email: stevep@ce.montana.edu

Chapter 3

G. R. Koerner, Y. G. Hsuan and R. M. Koerner
Drexel University
Civil, Architectural and Environmental Engineering Department
3141 Chestnut Street
Philadelphia
PA 19104
USA

email: gkoerner@dca.net

Chapter 4

L. David Suits
Executive Director
North American Geosynthetics Society
PO Box 12063
Albany
NY 12212-2063
USA

email: ldavidalbany@aol.com

Chapter 5

Emeritus Professor C. J. F. P. Jones
School of Civil Engineering and Geosciences
Drummond Building
Newcastle University
Newcastle upon Tyne
NE1 7RU
UK

email: c.j.f.p.jones@ncl.ac.uk

Chapter 6

Dr Jonathan Fannin
Department of Civil Engineering
University of British Columbia
6250 Applied Science Lane
Vancouver, BC
V6T 1Z4
Canada

email: jonathan.fannin@ubc.ca

Chapter 7

Dr Wilhelm Wilmers
Berliner Ring 72
D 35576 Wetzlar
Germany
email: wilhelm.wilmers@t-online.de

Chapter 8

Dr T. Meggyes
Europe Editor
Land Contamination and Reclamation
Federal Institute for Materials Research and Testing
Unter den Eichen 87
D12205 Berlin
Germany

email: tamas.meggyes@gmail.com

Chapter 9

Helmut Zanzinger
SKZ-TeConA GmbH
Testing, Quality Assurance, Certification
Friedrich-Bergius-Ring 22
97076 Würzburg
Germany

email: h.zanzinger@skz.de

Chapter 10

Dr James G. Collin, P. E.
The Collin Group, Ltd
7445 Arlington Road
Bethesda
MD 20814
USA

email: jim@thecollingroup.com

Chapter 11

Dr Derek Smith
EDGE Consultants UK Ltd
Berkshire House
252–256 Kings Road
Reading
RG1 4HP
UK

email: dms@edgeconsultants.co.uk

Chapter 12

Professor R. W. Sarsby
Section Leader for Civil Engineering (SEBE)
University of Wolverhampton
Wulfruna Street
Wolverhampton
WV1 1SB
UK

email: r.sarsby@wlv.ac.uk

Part I

General issues

1
The design principles of geosynthetics

R. M. KOERNER
Drexel University, USA

1.1 Introduction

As with all new materials design, one adopts and adapts earlier approaches from other and/or similar materials. With geosynthetic materials design, the closest allied fields are construction materials and, in particular, soil materials as encompassed within the discipline of geotechnical engineering. As is to be expected with the gradual maturing of the geosynthetic area, the design methods have advanced from very simplistic to quite detailed and still emerging.

In this regard, the chapter will describe the following:

1 'Design by cost' (exemplifying past practice).
2 'Design by specification' *and* 'Design by function' (exemplifying present practice).
3 'Design using probability' *and* 'Load and reduction factor design' (exemplifying possible future practice).

1.2 Past practice in geosynthetic design

Manufacturers' specifications appeared almost simultaneously with the development and introduction of each geosynthetic product's entry into a particular application. Geotextile and geomembrane manufacturers led the way with product specifications accompanying each product throughout the 1960s and 1970s. The downside of such specifications was that, either overtly or by using subtle test methods, the net result was to use that particular product, thereby excluding all others. Of course, the designer was at liberty to 'cut and paste', thereby forming a project-specific specification but this was difficult owing to rapid changes in the emerging technology and the general lack of field performance and designers experience. Thus, which tests to include, which minimum or maximum values to select, which test procedures to evoke and which testing frequencies to require were all very subjective issues. As a result, the method often used by the designer could be described as 'design by cost'.

Design by cost is quite simple. The funds available are divided by the area to be covered, and a maximum available unit price that can be allocated for the geosynthetic product is calculated. The geosynthetic product with the best properties for the site-specific application is then selected within this unit price limit and according to its availability. The method is obviously weak technically but is one that has been practised and very often resulted in adequate performance. It perhaps typified the situation in the early days of geosynthetics, but it is very outmoded by the current standard of practice.

1.3 Present practice in geosynthetic design

A defining point in geosynthetics was the first international conference on the subject in Paris in 1977. This conference spurred the first books on the topic (Koerner and Welsh, 1980; Rankilor, 1981) both of which collected more advanced and generic specifications and laid the groundwork for designing by function. Thus, from 1980 to the present, geosynthetic design has taken two parallel routes, 'design by specification', and 'design by function'. In general, design by specification is used for ordinary and non-critical applications, while design by function is used for site-specific and generally critical applications. Each will be explained.

1.3.1 Design by specification

Design by specification is very common and is used extensively when dealing with public agencies and many private owners as well. In this method, several application categories are listed in association with various physical, mechanical, hydraulic and/or endurance properties. The application areas are usually related to the intended primary function.

A federal agency that has formulated a unified approach in the USA for geotextiles is the American Association of State Highway and Transportation Officials (AASHTO). In its M288 geotextile specifications, AASHTO provides for three different strength classifications (Table 1.1). The classifications are essentially a list of minimum strength properties meant to withstand varying degrees of installation survivability stresses. It is the first step in the process.

- Class 1. For severe or harsh survivability conditions where there is a greater potential for geosynthetic damage.
- Class 2. For typical survivability conditions; this is the default classification to be used in the absence of site-specific information.
- Class 3. For mild survivability conditions where there is little or no potential for geosynthetic damage.

The second step is to select one of several different tables according to the specific function. These functions follow the intended application. They are filtration,

Table 1.1 AASHTO M288 geotextile strength property requirements

	Test method	Units	Geotextile Classification[a]					
			Class 1		Class 2		Class 3	
			Elongation <50%[b]	Elongation >50%[b]	Elongation <50%[b]	Elongation >50%[b]	Elongation <50%[b]	Elongation >50%[b]
Grab strength	ASTM D4632	N	1400	900	1100	700	800	500
Sewn seam strength[c]	ASTM D4632	N	1200	810	990	630	720	450
Tear strength	ASTM D4533	N	500	350	400[d]	250	300	180
Puncture strength[e]	ASTM D4833	N	500	350	400	250	300	180
Burst strength[f]	ASTM D3786	kPa	3500	1700	2700	1300	2100	950
Permittivity	ASTM D4491	s^{-1}	Minimum property requirements for permittivity, apparent opening size and ultraviolet stability are based on geotextile application. Refer to separate tables for subsurface filtration, separation, stabilization or permanent erosion control					
Apparent opening size	ASTM D4751	mm						
Ultraviolet stability	ASTMD4355	%						

[a]Required geotextile classification is designated in accompanying tables for the indicated application. The severity of installation conditions for the application generally dictate the required geotextile class. Class 1 is specified for more severe or harsh installation conditions where there is a greater potential for geotextile damage, and Class 2 and Class 3 are specified for less severe conditions.
[b]As measured in accordance with ASTM D4632. Note that woven geotextiles fail at elongations (strains) less than 50%, while non-woven geotextiles fail at elongation (strains) greater than 50%.
[c]When sewn seams are required. Overlap seam requirements are application specific.
[d]The required MARV tear strength for woven monofilament geotextiles is 250 N.
[e]Puncture strength will probably change from ASTM D4833 to ASTM D6241 with higher values.
[f]Burst strength will probably be omitted in the near future.

Table 1.2 AASHTO M288 subsurface filtration (called 'drainage' in the actual specification) geotextile requirements

Property	Test method	Units	Requirements for the following amounts of *in situ* soil passing 0.075 mm[a]		
			<15%	15–50%	>50%
Geotextile class			Class 2 from Table 1.1[b]		
Permittivity[c,d]	ASTM D4491	s^{-1}	0.5	0.2	0.1
Apparent opening size[c,d]	ASTM D4751	mm	0.43 maximum average roll value	0.25 maximum average roll value	0.22 maximum average roll value
Ultraviolet stability (retained strength)	ASTM D4355	%	50% after exposure for 500 h		

[a]Based on the grain size analysis of *in situ* soil in accordance with AASHTO T88.
[b]Default geotextile selection. The engineer may specify a Class 3 geotextile from Table 1.1 for trench drain applications based on one or more of the following.

1. The engineer has found Class 3 geotextiles to have sufficient survivability based on field experience.
2. The engineer has found Class 3 geotextiles to have sufficient survivability based on laboratory testing and visual inspection of a geotextile sample removed from a field test section constructed under anticipated field conditions.
3. The subsurface drain depth is less than 2 m, the drain aggregate diameter is less than 30 mm and the compaction requirement is equal to or less than 95% of the value AASHTO specified in T99.

[c]These default filtration property values are based on the predominant particle sizes of the *in situ* soil. In addition to the default permittivity value, the engineer may require geotextile permeability and/or performance testing based on engineering design for drainage systems in problematic soil environments.
[d]Site-specific geotextile design should be performed especially if one or more of the following problematic soil environments are encountered: unstable or highly erodable soils such as non-cohesive silts; gap-graded soils; alternating sand–silt laminated soils; dispersive clays; rock flour.
[e]For cohesive soils with a plasticity index greater than 7, the geotextile maximum average roll value for the apparent opening size is 0.30 mm.

separation, stabilization, erosion control, temporary silt fences and prevention of reflective cracking. Table 1.2 presents the appropriate table for filtration applications. See Koerner (2005b) for the remaining tables and a more complete description together with example problems. It should be mentioned that many federal agencies worldwide have similar generic specifications.

There are additional non-federal and non-proprietary specifications that have been developed throughout the 1990s up to the present, most notable among which are the generic specifications of the Geosynthetic Research Institute (GRI). At present, they are as follows.

GRI-GCL3 *Test Methods, Required Properties, and Testing Frequencies of Geosynthetic Clay Liners (GCLs).*

GRI-GM13 *Test Properties, Required Properties, and Testing Frequency and Recommended Warranty for High Density Polyethylene (HDPE) Smooth and Textured Geomembranes.*

GRI-GM17 *Test Properties, Required Properties, and Testing Frequency and Recommended Warranty for Linear Low Density Polyethylene (LLDPE) Smooth and Textured Geomembranes.*

GRI-GM18 *Test Properties, Required Properties, and Testing Frequency and Recommended Warranty for Flexible Polypropylene (fPP and fPP-R) Nonreinforced and Reinforced Geomembranes.* This has been temporarily suspended pending additional testing.

GRI-GM19 *Seam Strength and Related Properties of Thermally Bonded Polyolefin Geomembranes.*

GRI-GM21 *Test Methods, Required Properties, and Testing Frequency and Recommended Warranty for Ethylene Propylene Diene Terpolymer (EPDM) Nonreinforced and Scrim Reinforced Geomembranes.*

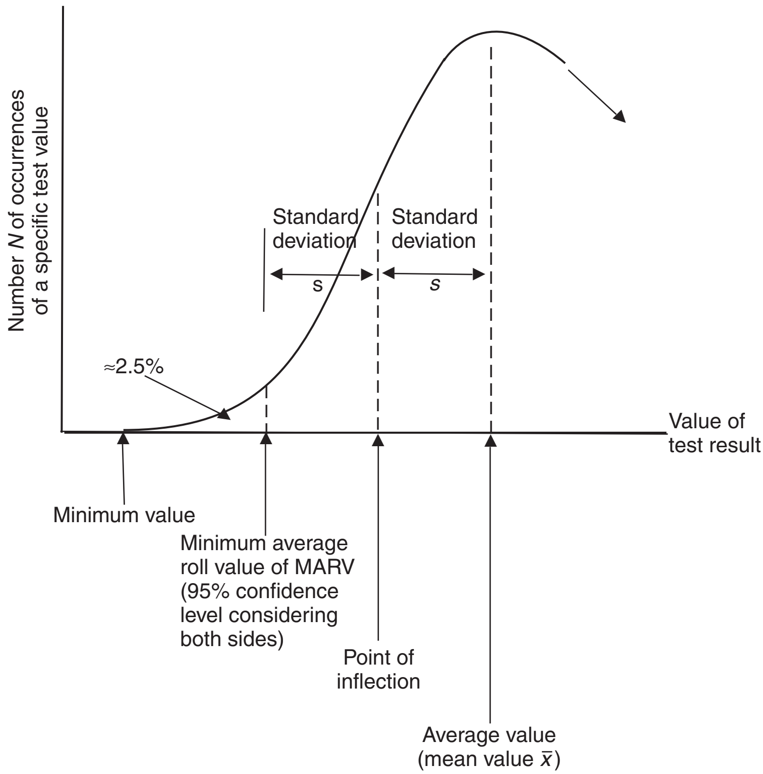
GRI-GT10 *Test Methods, Properties and Frequencies for High Strength Geotextile Tubes used as Coastal and Riverine Structures.*

GRI-GT12 *Test Methods and Properties for Nonwoven Geotextiles Used as Protection (or Cushioning) Materials.*

GRI-GT13 *Test Methods and Properties for Geotextiles Used as Separation Between Subgrade Soil and Aggregate.*

All these specifications are available free on the Geosynthetic Institute's web site at http:/www.geosynthetic-institute.org. There are others available by different groups, but this gives a sampling of generic specifications in the geosynthetics industry.

It must be cautioned that, when using a design-by-specification method, the specifications sometimes list *minimum* required properties, whereas some manufacturers' literature may list either *average lot* or *minimum average roll* property values. By comparing such a specification value with the manufacturer's listed values, one may be comparing different sets of numbers. This is because average lot value is the mean value for the particular property in question from all the tests made on that lot of material. This may be the compilation of thousands of tests made over many months or even years of production of that particular product style. Thus, the average lot value is considerably higher than the minimum value (Fig. 1.1). An intermediate value between these two extremes is the minimum average roll value (MARV). The MARV is the average of a representative number of tests made on selected rolls of the lot in question, which is limited in area to the particular site in question. This value is numerically equivalent to two standard



1.1 Relative relationships of different statistical values used in geosynthetic specifications and manufacturers' literature.

deviations lower than the mean, or average, lot value. Thus, it is seen that the MARV is the minimum of a limited series of average roll values. These different values are shown schematically in Fig. 1.1.

Clearly, the design-by-specification method must compare like sets of numbers. If the intent of the specification is to list MARVs (as it is with Tables 1.1 and 1.2), then the manufacturer's listed mean or average values must be decreased by two standard deviations (approximately 5–20%) if average lot values are given. Only if MARVs are given by the manufacturer can they be directly compared with a MARV-based specification value on a like-set-of-number basis.

It is important to note that only in geotextile design do we use the concepts of MARV and (also the maximum coverage roll value) (MaxARV). This is due to the greater statistical variation in geotextile properties versus other geosynthetics.

1.3.2 Design by function

Design by function consists of assessing the primary in-service function to be performed by the geosynthetic and then calculating the required numerical value of a particular property for that function. By dividing this value into the candidate material's allowable property value, a factor of safety (FS) results:

$$FS = \frac{\text{allowable (test) property}}{\text{required (design) property}} \tag{1.1}$$

where

allowable property = numerical value based on a laboratory test that models the actual situation or is adjusted accordingly by reduction factors
required property = numerical value obtained from a design method that models the actual situation
FS = factor of safety against unknown loads and/or uncertainties in the analytical or testing process; sometimes called a global factor of safety

If the FS is sufficiently greater than 1.0, the candidate geosynthetic is acceptable. The above process can be repeated for a number of available products, and, if others are acceptable, then the final choice becomes one of availability and least cost. The individual steps in this process are as follows.

1 Assess the particular application, considering not only the candidate geosynthetic but also the material system on both sides of it.
2 Depending on the criticality of the situation (i.e. 'if it fails, what are the consequences?'), decide on a minimum FS value. Note that this value may be regulatory suggested or even imposed.
3 Decide on the material's primary function, the choices being separation, reinforcement, filtration, drainage or containment.
4 Calculate numerically the required property value in question on the basis of its primary function.
5 Test for, or otherwise obtain, the candidate geosynthetic's allowable value of this particular property (recall the previous discussion on the recommended use of MARVs).
6 Calculate the FS on the basis of the allowable property (Step 5) divided by required property (Step 4) per Equation [1.1].
7 Compare this FS with the required value decided upon in Step 2.
8 If not acceptable, repeat the process with a product with more appropriate properties.
9 If it is then acceptable, check whether any secondary function of the material is more critical.
10 Repeat the process for other available products and if more than one satisfy the FS requirement, select the product on the basis of least cost and availability.

Note that the design-by-function process can also be used to solve for the required property value:

$$\text{required (design) property} = \frac{\text{allowable (test) property}}{\text{FS}} \qquad [1.2]$$

The design-by-function approach obviously necessitates identifying the primary function that the geosynthetic is to serve; thus an overt awareness of the site-specific situation is necessary on the part of the designer.

It is important to recognize that the test property must be modified to account for field considerations which are not simulated in the laboratory test. For example, one takes a laboratory value of the candidate material and then reduces it for aspects of the testing which did not simulate the anticipated field situation to arrive at an allowable value. Koerner (2005c) presents reduction factor values for reinforcement and flow rate applications insofar as installation damage, creep, degradation, intrusion, clogging, etc., are concerned. See Table 1.3 for strength related and Table 1.4 for flow rate applications. These values are multiplied together (assuming worst-case synergy) and then divided into the laboratory (or ultimate) measured value. This methodology for strength and flow problems is given by

$$T_{\text{allow}} = \frac{T_{\text{ult}}}{RF_{\text{ID}} \times RF_{\text{CR}} \times RF_{\text{CBD}}} \qquad [1.3]$$

where

T_{allow} = allowable tensile strength
T_{ult} = ultimate tensile strength
RF_{ID} = reduction factor for installation damage
RF_{CR} = reduction factor for creep
RF_{CBD} = reduction factor for chemical and biological degradation

and for flow problems by

$$q_{\text{allow}} = \frac{q_{\text{ult}}}{RF_{\text{CR}} \times RF_{\text{IN}} \times RF_{\text{CC}} \times RF_{\text{BC}}} \qquad [1.4]$$

where

q_{allow} = allowable flow rate
q_{ult} = ultimate flow rate
RF_{CR} = reduction factor for creep reduction of void space
RF_{IN} = reduction factor for adjacent materials intruding into the geocomposite's void space
RF_{CC} = reduction factor for chemical clogging
RF_{BC} = reduction factor for biological clogging

1.4 Possible future practice in geosynthetic design

On the horizon there appear to be two possible extensions of the present status of geosynthetic materials design. One is the use of risk assessment via probability

Table 1.3 Recommended strength reduction factor values [after Koerner (2005c)]

Area	Range of reduction factor values		
	Installation damage	Creep[a]	Chemical–Biological degradation[b]
Separation	1.1–2.5	1.5–2.5	1.0–1.5
Cushioning	1.1–2.0	1.2–1.5	1.0–2.0
Unpaved roads	1.1–2.0	1.5–2.5	1.0–1.5
Walls	1.1–2.0	2.0–4.0	1.0–1.5
Embankments	1.1–2.0	2.0–3.5	1.0–1.5
Bearing and foundations	1.1–2.0	2.0–4.0	1.0–1.5
Slope stabilization	1.1–1.5	2.0–3.0	1.0–1.5
Pavement overlays	1.1–1.5	1.0–2.0	1.0–1.5
Railroads	1.5–3.0	1.0–1.5	1.5–2.0
Flexible forms	1.1–1.5	1.5–3.0	1.0–1.5
Silt fences	1.1–1.5	1.5–2.5	1.0–1.5

[a]The low end of the range refers to applications which have relatively short service lifetimes and/or situations where creep deformations are not critical to the overall system performance.
[b]Some authors have listed biological degradation as a separate reduction factor. There is no evidence, however, of such degradation for the typical polymers used to manufacture geotextiles. Thus, it is currently included with chemical degradation as a combined reduction factor.

Table 1.4 Recommended flow rate reduction factors [after Koerner (2005c)]

Application area	Range of reduction factor values			
	RF_{CR}[a]	RF_{IN}	RF_{CC}	RF_{BC}
Sport fields	1.0–1.5	1.0–1.2	1.0–1.2	1.1–1.3
Capillary breaks	1.0–1.2	1.1–1.3	1.1–1.5	1.1–1.3
Roof and plaza decks	1.0–1.2	1.2–1.4	1.0–1.2	1.1–1.3
Retaining walls, seeping rock, and soil slopes	1.2–1.4	1.3–1.5	1.1–1.5	1.0–1.5
Drainage blankets	1.2–1.4	1.3–1.5	1.0–1.2	1.0–1.2
Infiltrating water drainage for landfill covers	1.1–1.4	1.3–1.5	1.0–1.2	1.5–2.0
Secondary leachate collection (landfills)	1.4–2.0	1.5–2.0	1.5–2.0	1.5–2.0
Primary leachate collection (landfills)	1.4–2.0	1.5–2.0	1.5–2.0	1.5–2.0
Wick drains (prefabricated vertical drains)	1.0–2.5	1.5–2.5	1.0–1.2	1.0–1.2
Highway edge drains	1.5–3.0	1.2–1.8	1.1–5.0	1.0–1.2

[a]Creep values are sensitive to the core structure and to the density of the resin used. Creep of the covering geotextile(s) is a product-specific issue. The magnitude of the applied load is of major importance in both situations.

theory such that the FS value is accompanied by an associated probability of failure (P_f); the other is a technique known as load and resistance factor design (LRFD). Each will be briefly described, as well as their interrelationship to one another.

1.4.1 Probability of failure in geosynthetic design

As indicated in the FS equation given previously (Equation [1.1]), the *allowable value* invariably comes from testing of laboratory specimens for the product under consideration. The statistics (mean and standard deviation) of such testing are at present available through the GAI-LAP proficiency test program (Koerner, 1996, 2005a). Allen (2002) gave additional insight in this regard from the perspective of an individual laboratory. The *required value* consists of both geometric and load values. In general, the geometric values are well defined. The load values, however, are very subjective. Live loads including hydraulic and seismic loads are perhaps the variables with the greatest statistical variation of all required input variables. (For this reason consultants sometimes use upper-bound values for use in designs that are particularly sensitive and critical. However, with probability analysis this approach is not needed, nor is it appropriate.)

Upon having the allowable and required values for a particular problem, the calculation process for FS values is exactly as previously described. Computer codes are available for a number of strength- and hydraulic-related applications. Such computer codes, and the theories upon which they are based, usually have great accuracy in comparison with the input variables. Nevertheless, the result of this entire process is to generate a FS value greater than unity. How much greater depends upon the designer's confidence in the input variables versus the implications of failure or, at the minimum, unsatisfactory performance. Yet, the traditional FS value can be nicely counterpointed with a P_f value, which is a form of risk assessment.

Risk assessment in the form of probability of failure, P_f, is not new. From a geotechnical perspective the book by Harr (1987) was probably the first complete treatise on the subject. Adding to this information base was the work of Christian *et al.* (1994). More recently, the US Army Corps of Engineers (1997, 1998) has been involved as well as the recent appearance of the book by Baecher and Christian (2003). Indeed, the effort is at present worldwide in its scope with many excellent references in addition to those noted.

Two situations have recently coalesced to make the probability-of-failure approach practical. The first is the database of the GAI-LAP program mentioned earlier. The second is the appearance of an article by Duncan (2000). The latter methodology will be briefly described since it is recommended in this regard.

Step 1 Assemble the mean value and standard deviations of all the major variables that are to be used in the design method.

Step 2 Calculate the most likely value of the FS, namely FS_{MLV}, using the mean values. (This is, of course, standard design practice with the exception that values should not be artificially inflated as is sometimes done in practice, i.e. they should be actual mean values.)

Step 3 Calculate the standard deviation and coefficient of variation of the FS_{MLV} using plus and minus one standard deviation of all the test and design variables:

$$\sigma_{MLV} = \left[\left(\frac{\Delta FS_1}{2}\right)^2 + \left(\frac{\Delta FS_2}{2}\right)^2 + \left(\frac{\Delta FS_3}{2}\right)^2 + \cdots \right]^{½} \quad [1.5]$$

$$V_{MLV} = \frac{\sigma_{MLV}}{FS_{MLV}} \quad [1.6]$$

where

FS_{MLV} = most likely (or traditional) value of the FS
σ_{MLV} = standard deviation of the FS_{MLV}
V_{MLF} = coefficient of variation of the FS_{MLV}
ΔFS_i = $FS_i^+ - FS_i^-$ (for each variable)
FS_i^+ = FS calculated with the mean value of specific variable increased by one standard deviation
FS_i^- = FS calculated with mean value of the specific variable decreased by one standard deviation

Note that, in calculating each FS_i^+ and FS_i^- value, all the other ΔFS_i variables are kept at their most likely mean values.

Step 4 With both FS_{MLV} and V_{MLF} known, the probability of failure, P_f, can be determined using tables, or by using an analytical approach given by Duncan (2000). The P_f value represents the reliability of the FS_{MLV}. For example, a value of $P_f = 0.04\%$ suggests that the situation will experience four failures in 10 000 similar circumstances.

Step 5 Assess the FS value in the light of the accompanying P_f value. This assessment is currently quite subjective. Obviously, the lower the P_f value, the better, with the situation approaching zero being no likelihood of failure. Relatively high values of P_f can be accepted depending on the duration and criticality of the site-specific application.

Having this information on P_f values, we are now in a position to compare them with acceptable values. Unfortunately, there is no consensus of acceptable values at this point in time. Koerner and Koerner (2001) made an initial attempt but the values that they provided were very restrictive and drew a considerable number of negative comments. Their initial table has been modified upwards and is given here as Table 1.5. The table is structured according to the primary function that the geosynthetic is to serve and the sensitivity of the application within that particular

Table 1.5 Suggested limiting probability of failure values compared with results from numerous example problems

Geosynthetic primary function	Consequence of failure %			Average P_f values[a] (%)
	Low	Typical	Serious	
Separation	7.0	1.5	0.5	0.54
Reinforcement	3.0	0.5	0.1	2.1
Filtration	5.0	1.0	0.3	5.1
Drainage	5.0	1.0	0.3	4.8
Containment	3.0	0.5	0.1	2.9

[a]These values were obtained using all the numerical examples in the textbook by Koerner (2005b).

function. Consequences of failure are ranked as being low, typical and serious with approximate definitions as follows:

- 'Low' refers to a remediation cost of US$100 000 or less;
- 'Typical' refers to a remediation cost of US$1 000 000 or less;
- 'Serious' refers to a remediation cost of more than US$1 000 000 and/or loss of life.

The average P_f values taken from the numerical examples in the paper by Koerner (2002) are superimposed on the last column of Table 1.5. It can be seen that there is a correlation with the acceptable values going from a low to serious consequence of failure. Obviously, much more thought and consideration should eventually be included in a table of this type. The paper by D'Hollander (2002) is valuable in this regard.

1.4.2 Load and reduction factor design

If one rearranges Equation [1.1] (together with the elimination of the FS value) in the form of an inequality, one has the basic concept of load and reduction factor design (LRFD). Thus, the required (design) property must be less than or equal to the allowable (test) property. Now, using load factors on the required property (to obtain worst-case conditions) and reduction factors on the ultimate property (to obtain allowable values), we have the desired LRFD formulation

$$\Sigma(\mathrm{LF}_i\, D_{ni}) < (\mathrm{R_n}/\mathrm{RF}) \qquad [1.7]$$

where

LF_i = load factors on each design element (all are 1.0 or greater)
D_{ni} = design value on each element
RF = resistance factors accounting for degradation, creep, etc. (all are 1.0 or greater)
R_n = ultimate resisting value from laboratory tests

As indicated, the left-hand side of the equation is the load side and the right-hand side is the resistance side. Recall from previous discussion in Section 3.2 that the resistance side is already practised in geosynthetics design. Thus, present geosynthetics design practice already accomplishes one half of LRFD. The other half is, however, far less defined or established and is not at present practised in geosynthetics design.

Scott *et al*. (2003) offer load factors in Table 1.6 and Table 1.7 for dead, live, wind and seismic design conditions. It is unfortunate that hydraulic loads are not specifically mentioned since the difference between a 1 h storm and a 100 year

Table 1.6 Load factors for the ultimate limit state, i.e. failure or collapse [after Scott *et al.* (2003)]

	Load factor							
	USA				Canada		Europe	
Load	AASHTO[a] (1998)	ACI[a] (1999)	AISC[a] (1994)	API[a] (1993)	MOT[a] (1992)	NRC[a] (1995)	DGI[a] (1985)	ECS[a] (1995)
Dead	1.25–1.95	1.4	1.2–1–4	1.1–1.3	1.1–1.5	1.25	1.0	1.0–1.35
Live	1.35–1.75	1.7	1.6	1.1–1.5	1.15–1.4	1.5	1.3	1.3–1.5
Wind	1.4	1.3	1.3	1.2–1.35	1.3	1.5	1.3	1.3–1.5
Seismic	1.0	1.4	1.0	0.9	1.3	1.0	1.0	1.0

[a]AASHTO, American Association of State Highway and Transportation Officials; ACI, American Concrete Institute; AISC, American Institute of Steel Construction; API, American Petroleum Institute, MOT, Ministry of Transportation (Canada); NRC, National Research Council of Canada; DGI, Danish Geotechnical Institute; ECS, European Committee for Standardisation.

Table 1.7 Load factors for the serviceability limit state, i.e. excessive deformation [after Scott *et al.* (2003)]

	Load factor						
	USA			Canada		Europe	
Load	AASHTO[a] (1998)	ACI[a] (1999)	AISC[a] (1994)	MOT[a] (1992)	NRC (1995)	DGI[a] (1985)	ECS[a] (1995)
Dead	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Live	1.0	1.0	1.0	0.75	1.0	N/A[b]	1.0
Wind	0.3	1.0	1.0	0.7	1.0	N/A[b]	1.0

[a]AASHTO, American Association of State Highway and Transportation Officials; ACI, American Concrete Institute; AISC, American Institute of Steel Construction; API, American Petroleum Institute, MOT, Ministry of Transportation (Canada); NRC, National Research Council of Canada; DGI, Danish Geotechnical Institute; ECS, European Committee for Standardisation.
[b]N/A, not applicable, values for transient loads are given in the structural code.

storm (or even maximum probable precipitation) is enormous. Nevertheless, Table 1.6 for failure, or collapse, situations is very helpful. LRFD also encompasses serviceability issues and Table 1.7 gives the load factors for such conditions. As an example, a geogrid-reinforced retaining wall would use Table 1.6 to prevent collapse, and Table 1.7 to prevent excessive wall deformation.

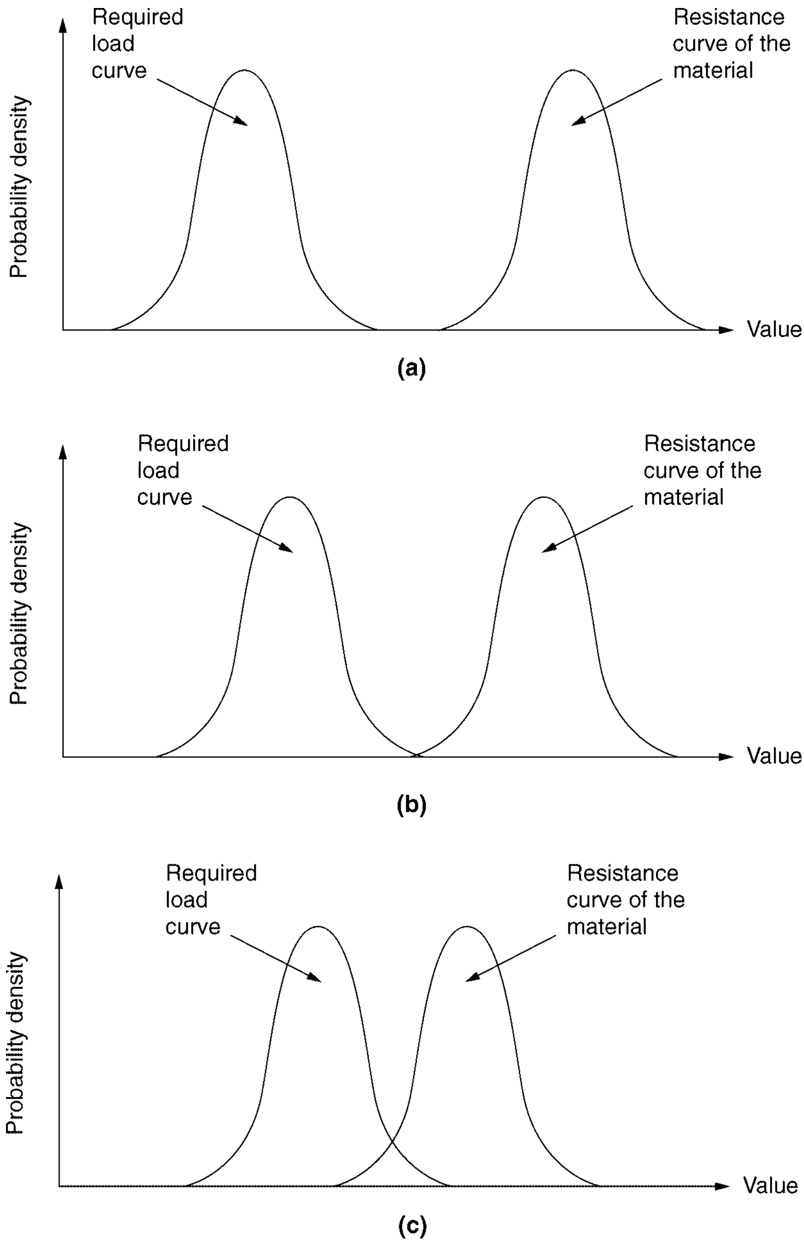
There are many additional aspects of LRFD and the worldwide literature is abundant in this regard. Suffice it to say that structural engineering designers are fully involved in LRFD and it is very possible that geotechnical (and geosynthetics) engineering designers might be encouraged, or even forced, to follow accordingly. Time will tell in this regard.

1.4.3 Interrelationships of probability and load and reduction factor design

Probability and LRFD can be nicely counterpointed against one another, at least on a conceptual basis. Figure 1.2 attempts to do this in the form of probability curves of required load and resistance curves of the material. Figure 1.2(a) shows the case in which the probability curves of required values and resistance values do not overlap at all. This signifies that there is zero risk or likelihood of failure. Figure 1.2(b) shows the case when the curves just touch, which signifies that at the probability extremes there is still no likelihood of failure, but it is a limiting condition. Figure 1.2(c) shows the curves overlapping one another. The greater the overlap, the higher is the risk, or the likelihood of failure. While these concepts are clear, it is quite another matter to obtain the requisite data to draw the curves specifically and to generate numerical information. This area appears to be currently in a developing stage.

1.5 Summary and conclusions

This chapter on geosynthetics design has traced its origin, through the present status, and into possible future methods. Regarding the *past*, design by cost was the original procedure and (paradoxically) served reasonably well. From the earliest days it was tempered with manufacturers' specifications, but that was to be expected. The *present* status sees geosynthetics design taking two pathways: design by specification for customary and non-critical applications; design by function for site-specific and critical applications. Both are quite well positioned and can be considered as the state of the practice. The future of geosynthetics design promises the use of risk assessment via probability theory juxtaposed with LRFD. These techniques can be considered as the state of the art. How quickly this may, or may not, occur is uncertain but a considerable literature base is developing and our structural engineering design colleagues have fully embraced the concept. Clearly, geosynthetics designers should be aware of the details and nuances of the techniques and this brief introduction may help in this regard.



1.2 Contrasting probability curves to assess relative risk, i.e. graphical representation of the LRFD concept: (a) curves not intersecting, no possibility of failure; (b) curves just touching, zero risk of failure; (c) curves intersecting, finite risk of failure.

1.6 Acknowledgements

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2
The material properties of geosynthetics

S. W. PERKINS
Montana State University, USA

2.1 Introduction

This chapter describes the material properties of geosynthetics that are important to their use in various applications and methods for measurement of those properties. Geosynthetic material properties and test methods for the measurement of those properties have arisen first from previously existing materials that resemble geosynthetics, such as textile materials used in the garment trade and plastic materials used in various industrial applications. As functions and applications of geosynthetics have been identified and developed, properties and test methods have followed to aid in proper design and construction. For example, materials used for reinforcement depend heavily on mechanical properties while filtration and drainage functions depend on hydraulic properties. Most applications involve transport and storage of materials, construction in relatively harsh environments and the necessity for a long service life, for which endurance and durability properties are important. Given the relatively young age of geosynthetic materials and their applications, and the time needed for the development and standardization of test methods, many test methods are not standardized. Additionally, the status of these test methods is constantly evolving as tests are developed, standardized and refined.

Geosynthetic materials are time and temperature dependent. This imposes special considerations for testing conditions to ensure consistency between testing laboratories and applicability to field conditions. The properties of geosynthetics are often direction dependent, meaning that the direction in which they are tested will influence the values of certain properties.

The properties of geosynthetics are typically grouped into those used for quality assurance (QA) or quality control (QC) and those used for design. These two groups of properties are sometimes referred to as index and performance properties, respectively. These names have also taken on other meanings, such as index properties being those obtained from tests on the geosynthetic itself as isolated from any surrounding soil, and performance properties being those determined from tests where the geosynthetic is in contact with a subject soil.

In this chapter, properties and test methods are grouped into the categories of physical, mechanical, hydraulic, endurance and degradation properties. The chapter concludes with information given on future trends and sources for obtaining further information.

2.2 Physical properties

Physical properties of geosynthetics are basic properties related to the composition of the materials used to fabricate the geosynthetic and include the type of structure, specific gravity, mass per unit area, thickness and stiffness. The type of structure of a geosynthetic describes the physical make-up of the geosynthetic resulting from the process used to manufacture the material. The structure of the geosynthetic often dictates the application area for which the material is appropriate. For example, a uniaxial geogrid is appropriate for applications where load is expected in one principal direction of the material, such as in a long slope or retaining wall. The geosynthetic structure is most often described for geogrids. The structure of geogrids of greatest importance is that associated with the manufacturing process used to form the junctions of the geogrid, with examples including woven, integral and welded junctions. Structure can also be described for geotextiles where the two main types of structure include woven and non-woven geotextiles.

The specific gravity of a geosynthetic is measured on the basic polymeric material or materials used to form the geosynthetic. The specific gravity is defined conventionally as the ratio of the material's unit volume weight to that of distilled, de-aerated water at a standard temperature. Ranges of values for the specific gravity of commonly used geosynthetic polymers are listed in Table 2.1. The specific gravity of the geosynthetic polymer is important in applications where the geosynthetic will be placed underwater where polymers with values of specific gravity less than one will require weighting in order to sink the material into position.

Mass per unit area describes the mass (usually in units of grams) of a material per unit area (generally in square metres) and should be measured with no tension applied to the material. Typical values for geotextiles lie between 130 and 700 g/m^2 while for geogrids the values range from 200 to 1000 g/m^2.

The thickness of a geosynthetic is measured as the distance between the extreme upper and lower surfaces of the material. For geotextiles, this distance is measured

Table 2.1 Specific gravities of common geosynthetic polymers

Polymer	Specific gravity
Polyamide	1.05–1.14
Polyester	1.22–1.38
Polyethylene	0.90–0.96
Polypropylene	0.91

while a specified pressure is applied to the material. Thicknesses of geotextiles range from 0.25 to 7.5 mm. The thickness of common geomembranes used today is 0.5 mm.

The physical property of stiffness refers to the flexibility of the material and is not a description of the mechanical property of stiffness which describes the material's load–strain modulus. The flexibility of a geosynthetic is determined by allowing the material to bend under its own weight as it is being slid over the edge of a table. The properties of flexural stiffness or rigidity describe the material's capability of providing a suitable working platform during installation and is an important property when installation is performed over soft soil sites.

2.3 Mechanical properties

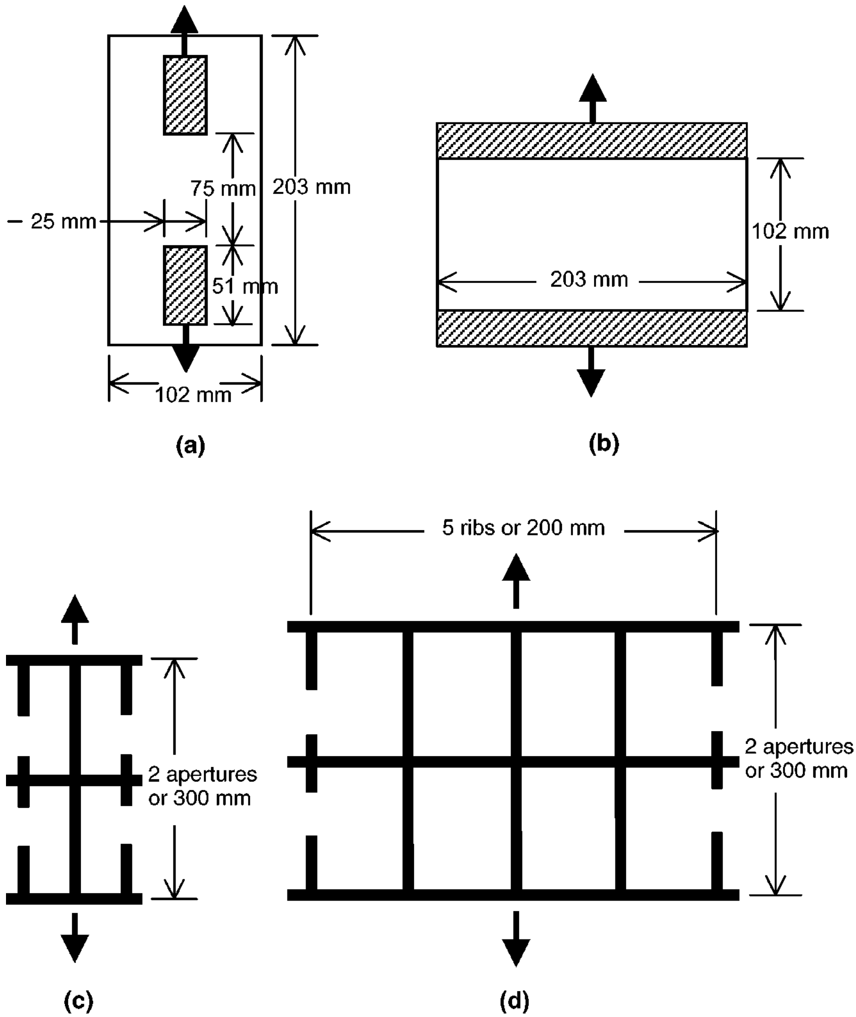
Mechanical properties of geosynthetics relate to applications where the geosynthetic is required to bear a load or to undergo a deformation. During the construction of facilities containing geosynthetics, loads perpendicular to the plane of the geosynthetic can be introduced as the material is placed on irregular surfaces with soil compacted on top. These loads can be significant and can often dictate the mechanical properties specified for the geosynthetic. Failure to specify appropriate mechanical properties for the construction conditions may result in physical damage (i.e. punctures, tears and rips) to the geosynthetic, which then may compromise the mechanical properties needed for proper functioning of the application.

Loading can also be applied in the plane of the geosynthetic resulting in tension of the material. This type of loading is generally associated with the function or operation of the constructed facility and where the mechanical properties of the geosynthetic are typically used in the design of the facility. Mechanical properties pertaining to the shearing resistance between the geosynthetic and the surrounding soil are also important as this resistance is responsible for transferring load from the soil into tensile load in the geosynthetic.

Mechanical properties of geosynthetics are often categorized as either index or performance properties. Index properties refer to those determined on the geosynthetic itself in the absence of any surrounding soil. These properties are sometimes referred to as in-isolation properties. Performance properties involve those determined in the presence of a standard soil or the site-specific soil.

2.3.1 Tensile properties

Tensile properties of geosynthetics are generally the most important set of properties, particularly for applications where reinforcement is the primary function of the geosynthetic. Tensile properties are used for quality QC/QA and as design parameters for various applications. The tests used for QC and QA purposes tend to be simpler and less time consuming to perform and interpret than those used to



2.1 Specimen sizes for various tensile tests: (a) grab; (b) geotextile wide width; (c) geogrid wide width, method A; (d) geogrid wide width, method B and C.

generate design parameters. Figure 2.1 shows a schematic diagram of different types of tensile tests. Figure 2.1(a) is known as a grab tensile test with ASTM test designation ASTM D4632. The grab tensile test is performed on geotextiles and provides QC and QA information that can only be used comparatively between geotextiles with similar structures since each material structure performs in a unique manner in this test. The test is performed by gripping the specimen as shown in Fig. 2.1(a) and applying a continuously increasing load until rupture occurs. The load at rupture and the corresponding elongation are measured and reported. The grab tension test also represents loading that may occur in the field because of the spreading action of two pieces of coarse aggregate in contact with the geosynthetic.

Figure 2.1(b), Fig. 2.1(c) and Fig. 2.1(d) illustrate tension tests used to determine tensile design properties of geotextiles and geogrids. Figure 2.1(b) shows the test specimen size for wide-width tension tests on geotextiles (ASTM D4595). For geogrids, either multirib specimens (Fig. 2.1(c)) or single-rib (Fig. 2.1(d)) may be used according to ASTM D6637. For the tests shown in Fig. 2.1(b), Fig. 2.1(c) and Fig. 2.1(d), the ultimate strength, strain at failure and modulus are typically determined. The strength and modulus are typically expressed in terms of a load per unit width of material rather than a stress since stress requires the definition of material thickness, which is generally difficult to describe for most geosynthetics and does not remain constant during tensile loading. The modulus can be defined as an initial modulus, a secant modulus or an offset tangent modulus. Modulus values are very dependent on how the specimen is conditioned at the beginning of the test and standardized procedures should be followed to ensure comparability of results. Since geosynthetics are rate and temperature dependent, standards should also be followed with respect to these test variables. Geosynthetic materials are typically direction dependent, meaning that tension tests should be performed in both principal material directions.

For tests where elongation or strain is measured, displacement measurement techniques become important. If displacement is measured as movement between the grips, then slippage within the grips should not occur. For geotextiles with strengths less than 90 kN/m, conventional clamping grips are usually sufficient. Wedge grips may be good for materials with strengths between 90 and 180 kN/m. For materials with strengths exceeding 180 kN/m, roller grips are typically used.

The tension tests described above are performed without any soil covering the geosynthetic and are therefore known as in-isolation or in-air tests. Soil covering the geosynthetic provides confinement to the material, which in general has the effect of increasing the material's modulus and strength. Increases in modulus and strength are most significant for non-woven geotextiles, but also noticeable for woven geotextiles and geogrids (Elias *et al*., 1998) This results from internal friction between fibres or yarns, alignment of curved fibres or yarns, and interlocking of soil within openings or apertures of geosynthetics (Elias *et al*., 1998).

The first two factors imply an effect on the intrinsic load–strain properties of the geosynthetic, while the third factor reflects interaction between the geosynthetic and the soil with the measured load–strain properties thereby reflecting the coupled responses of the material itself and its interaction with surrounding soil. For analyses where the geosynthetic and surrounding soil is treated as a coupled system, use of load–strain properties from confined tension tests are appropriate. For analyses where the intrinsic properties of the geosynthetic are uncoupled from the interaction between the geosynthetic and the soil, then the use of load–strain properties from in-air tests are more appropriate. An example of an uncoupled analysis is one using the finite element method where the load–strain properties of the geosynthetic are specified together with interface shear properties (confinement-dependent shear stiffness and friction angle) describing interaction between the geosynthetic and the surrounding soil.

Biaxial tension tests have been performed to assess material load–strain response for applications where load is applied simultaneously in two directions of the material (McGown and Kupec, 2004). Biaxial tests are performed by forming specimens in the shape of a cross and applying load simultaneously in the machine and cross-machine directions. The results can be used to assess strength and modulus under conditions of biaxial loading or to compute Poisson's ratio for use in advanced analyses (Perkins *et al*., 2004).

2.3.2 Compressibility

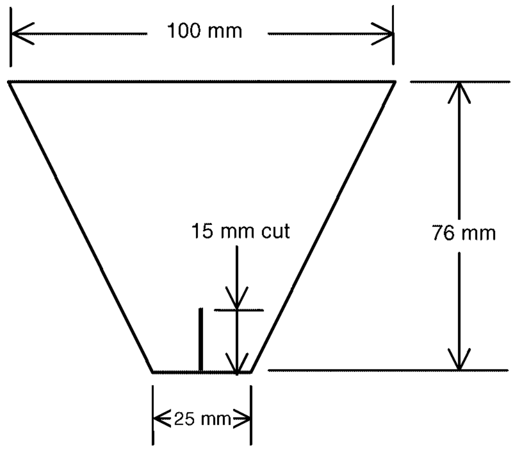
The compressibility of geosynthetics is defined as the relationship between the material thickness as a function of applied normal stress and is a test most appropriate for geotextiles and geonets that need to maintain a certain thickness to ensure water transmissivity. For geotextiles, non-woven needle-punched materials tend to be the most compressible, while woven and non-woven heat bonded materials show small levels of compressibility. Some materials, especially geonets, tend to experience small levels of compression prior to collapse. For these materials, the compression strength is of most importance.

2.3.3 Seam strength

Geosynthetics are generally manufactured in rolls of a given width and length. Particular jobs may require a coverage area that exceeds the size of the manufactured roll and where adjoining rolls may be mechanically or chemically jointed either in the field or in the manufacturing plant. Geosynthetics may be jointed by sewing, stapling, glueing or melting, or by the use of bodkin rods extending through the apertures of geogrids. Tensile tests are performed typically on wide-width specimens to assess the tensile strength of seams. The strength of the seam is compared with the tensile strength of the geosynthetic itself to arrive at a seam strength efficiency. Efficiencies of 100% are possible for geotextiles with tensile strengths less than 44 kN/m and drop to 50% for materials with strengths greater than 440 kN/m (Koerner, 2006).

2.3.4 Burst strength

Burst strength tests are performed on geotextiles and geomembranes by causing a circular piece of material clamped around its perimeter to stretch into the shape of a hemisphere by the application of pressure on one side of the material. The material stretches in tension until rupture occurs. In the field, geotextiles may



2.2 Specimen shape for the trapezoid tear test (ASTM D4533).

experience this type of loading when used as a separator between soft subgrade and coarse aggregate. As subgrade is squeezed upwards between voids of the coarse aggregate, the geotextile takes on a hemispherical shape similar to that experienced in the burst strength test. Geomembranes may experience this kind of loading in landfill applications where a void might open up beneath the geomembrane layer.

2.3.5 Tear strength

During the installation of geotextiles, stresses may be imposed which cause tears to initiate and propagate. Several types of tests have been developed to describe the tearing resistance of geotextiles. The most common test is the trapezoidal tear test (ASTM D4533). In this test, the specimen is formed in the shape of a trapezoid, as shown in Figure 2.2, and a 15-mm cut is made along one end of the specimen. The two non-parallel sides of the specimen are gripped in parallel grips of a tension load frame with the two grips aligned parallel to the cut made in the material and separated by a distance of 25 mm. This is accomplished by allowing folds to occur in the material greater than 25 mm in width. Tension is then applied and the cut in the material propagates across the specimen as individual strands of the geotextile are torn. Minimum values of tear strength are generally specified to control installation damage of geotextiles.

2.3.6 Puncture strength

In addition to the possibility of tears during installation, geotextiles and geomembranes can experience punctures from rocks, roots, sticks or other debris. A test has been developed to measure the puncture resistance of these materials

(ASTM D4833) where a steel rod of 8 mm diameter is used to puncture a geosynthetic stretched and clamped firmly over a cylinder of 45 mm inside diameter. The force necessary to cause the rod to puncture through the material is known as the puncture resistance.

2.3.7 Friction

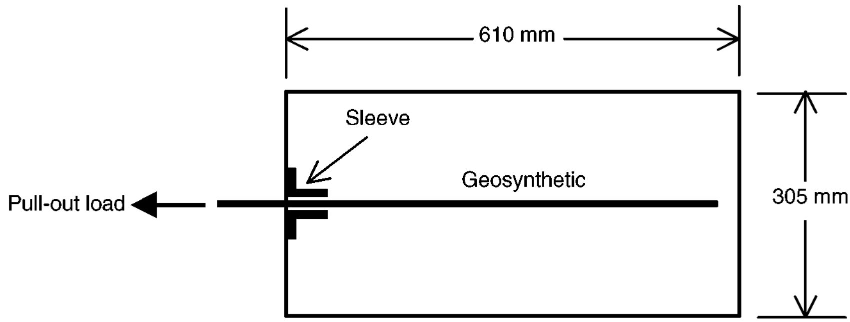
An adaptation of the direct shear test for soils is used to measure the shearing resistance or friction between geosynthetics and soils and between two layers of geosynthetics. Test method ASTM D5321 calls for a shear box measuring 300 mm by 300 mm. The shear box is configured to contain soil in the bottom half and the geosynthetic clamped to the top half of the box. Within the confines of the top half of the box and above the clamped geosynthetic, soil or a textured block may be used to transfer shear load evenly across the face of the geosynthetic.

The test is performed similarly to a direct shear test on soil with normal confinement being applied to the box prior to applying a horizontal shear displacement that causes the two halves of the box to displace and shear relative to each other. The shear load is measured and divided by the area of shear and plotted against the horizontal shear displacement. The ultimate shearing resistance is plotted against the normal stress confinement for several tests at different levels of confinement. A Mohr–Coulomb failure criterion is then obtained from these data. Values of cohesion and friction angle are compared with those obtained for the soil itself to arrive at shear strength parameter efficiencies. Similar procedures are followed for tests performed between two layers of geosynthetics. Results from these tests have applications in landfills where geosynthetic sheets are in contact with soil materials or other geosynthetic sheets on slopes where sliding can occur.

2.3.8 Pull-out resistance

Pull-out tests are typically performed to assess the anchorage or pull-out capacity of geosynthetics. This capacity is important in situations such as retaining walls, slopes and bridging over voids, where the geosynthetic is anchored into stable ground that is outside the zone of failure. The test can also be used to assess interface shear resistance and stiffness properties for applications where soil is moving relative to the geosynthetic, such as in reinforced roadways.

The test is performed in an apparatus described by ASTM D6706 and shown in Fig. 2.3, where the dimensions shown are minimum dimensions that may need to be increased depending on the structure of the geosynthetic, particle size of the soil, and provisions for reducing side-wall friction. Normal stress confinement is provided by an air bag placed between the top of the soil and a reaction frame. A sleeve is fitted to the front of the box where the geosynthetic enters and extends a minimum of 150 mm into the box. The purpose of the sleeve is to reduce the amount of normal stress generated along the front wall of the box as the geosynthetic



2.3 Apparatus for pull-out testing (ASTM D6706).

is being pulled out. Measurements during testing typically consist of applied pull-out load, horizontal displacement of the front of the geosynthetic and horizontal displacement of the geosynthetic at several locations along the material's length. The later is accomplished with the use of a telltale, which consists of a protected wire attached to the measurement point on the geosynthetic and extending out from the back of the box where it is attached to a displacement-sensing device.

The pull-out resistance or anchorage capacity is calculated as a line load taken as the force necessary to cause pull-out divided by the width of the specimen. This force is typically used to compute an interaction coefficient, which is essentially the ratio of the friction angle of the geosynthetic–soil interface to that of the soil itself. To make the calculations described above, it is important that sufficiently large displacement occurs along the entire embedment length of the geosynthetic such that the ultimate shearing resistance is fully mobilized. For long embedment lengths and large normal stress confinement, this may not be the case and the test must then be interpreted as a boundary-value problem where several methods have been proposed (Juran and Chen, 1988; Yuan and Chua, 1991; Perkins and Cuelho, 1999).

2.4 Hydraulic properties

Hydraulic properties of geosynthetics are important in applications where the material is used to convey or prevent the flow of liquids and gases. Geotextiles, geomembranes, geonets, geosynthetic clay liners and drainage composites are all materials that are called upon to perform these functions. Applications include drainage materials behind walls and within slopes, roadways and landfills, filtration materials within roads and around drainage trenches, and liquid and gas containment for ponds, for canals and within landfills.

2.4.1 Porosity

The porosity is a convenient property in that it has the same definition (the ratio of the void volume to the total volume) as that used for soils. The void volume, however, is difficult to measure, so the porosity has to be calculated from other physical properties (mass per unit area, density and thickness). As a result, other measures, including the percentage open area and apparent opening size (AOS), related to the porosity but more easily measured and more directly related to particular applications have been developed.

2.4.2 Percentage open area

The percentage open area is a property that is specified and measured for woven geotextiles and is a property that describes the ratio of the open area to the total area. The open area is typically measured by shining light through the material and projecting this light on to a screen that can be used to measure and sum the open areas. This test is not appropriate for non-woven geotextiles since the overlap of the weaves prevents most light from shining through even though liquid transmission is still very possible.

2.4.3 Apparent opening size

The AOS test was first developed for woven geotextiles but is now also used for non-woven materials. The test is described by ASTM D4751 and consists of passing glass beads of successively larger diameter through the material until only 5% of the beads pass through. The size of the beads in millimetres at which 5% passes is known as O_{95}. The corresponding size in the US sieve size is the AOS. The AOS or O_{95} represents the largest particle that would effectively pass through the geotextile. The equivalent opening size (EOS) has the same meaning as the AOS but can be specified for other percentage passing values, such as O_{50} or O_{90}. The AOS is typically specified in conjunction with requirements for filtration, with proper specification providing for soil retention without pore space clogging.

2.4.4 Permittivity

The permittivity describes the ability for fluid flow across the plane of the geosynthetic. It is formally defined as the cross-plane permeability divided by the thickness of the geosynthetic. ASTM D4491 describes a constant-head and a falling-head permeability test that is used to define permittivity under zero-normal-stress confinement. These tests are conducted like similar tests on soils only with the apparatus sized to accommodate the flows associated with geotextiles. Values of cross-plane permeability for geotextiles range from 0.0008 to 0.23 cm/s with a corresponding range of permittivities ranging from 0.02 to 2.1 s^{-1}. Non-woven needle-punched geotextiles experience a slight to moderate decrease in permittivity as the normal stress confinement on the material is increased. Geonets have values of permeability of the order of 1–10 cm/s. Geomembranes have a value of 10^{-11} cm/s while geosynthetic clay liners have saturated values ranging from 5.0×10^{-9} to 1.0×10^{-10} cm/s.

2.4.5 Transmissivity

Transmissivity describes the ability for fluid flow within the plane of the material and is defined as the in-plane permeability multiplied by the material thickness. The test method ASTM D4716 describes a constant-head test that can be conducted under varying normal stress confinement. Fluid is caused to flow one-dimensionally in the plane of the material from one end to another under constant-head conditions. Values of in-plane permeability of geotextiles range from 0.0006 to 0.04 cm/s with corresponding transmissivity values ranging from 3.0×10^{-9} to 2.0×10^{-6} m^2/s.

2.4.6 Soil retention

Geotextiles are often used as fences to retain fines as turbid water flows from disturbed areas to streams, ponds or lakes. The ability of the geotextile to allow water flow while retaining soil particles is determined by ASTM D5141. In this test, the site-specific soil is mixed with water to form a slurry and is poured into a flume box set on a 8% slope with the downstream end covered by the candidate geotextile. The flow rate of the soil–water mixture passing through the geotextile is measured together with the amount of fines. These measurements allow for the slurry flow rate and retention efficiency to be determined. The process is repeated at least three times to determine the degree of clogging that occurs.

2.5 Endurance properties

Endurance properties of geosynthetics focus on how short-term properties are affected by time during the service life of the facility. Issues of endurance arise as the material is installed, while the load is sustained, and while fluid flow is experienced.

2.5.1 Installation damage

The deformations and stresses experienced by geosynthetics during installation can be more severe than the actual design stresses for the intended application and arise from the placement and compaction of overlying fill. Damage may occur in the form of holes, tears and ruptures, which influences the mechanical and hydraulic properties of the material.

Criteria for survivability of geosynthetics have been developed by AASHTO M288-96. These criteria consider the construction conditions of the subgrade, the contact pressure provided by the construction equipment and the compacted base course thickness to be used. Based on the combination of these conditions, the survivability level of the geosynthetic is assessed. The survivability level is then expressed in terms of certain geosynthetic index properties. Field trials can also be performed using the site-specific ground conditions, construction equipment and procedures with the installed material exhumed immediately after placement to assess damage.

2.5.2 Creep and stress relaxation

Creep is defined as the elongation of a material under a constant load. Stress relaxation is the reduction in (relaxation of) stress when a material is loaded and then held at a constant level of strain. Creep is an important consideration in design as large levels of creep can lead to excessive deformation of reinforced structures or possible creep rupture of geosynthetics. Stress relaxation can result in more load being taken up by the soil, which may produce unsafe conditions for situations where the soil is close to failure.

Creep and stress relaxation are interrelated and are dependent on the viscous properties of the geosynthetic. Viscous properties of geosynthetics are dependent on the type of polymer. Creep and stress relaxation are most significant for geosynthetics composed of polypropylene and polyethylene and less significant for polyester and polyamide geosynthetics. The magnitude of creep and stress relaxation increase as the temperature, magnitude of load and time increase (Greenwood and Myles, 1986). Cyclic loading can also produce creep and stress relaxation since cyclic loading is another form of sustained loading.

ASTM D5262 describes a test method for determining elongation due to creep. The test is relatively simple to conduct and involves placing hanging weights on a geosynthetic specimen and making periodic measurements of elongation. A stress relaxation test is more difficult to conduct in that a fixed displacement must be applied and the load over time must be monitored. This implies the use of a displacement controlled device typically with electronic load-sensing devices.

2.5.3 Abrasion

The abrasion of geosynthetics is defined as the wearing away of any part of a material by rubbing against another surface. Excessive abrasion can lead to a loss of properties, e.g. strength, that are needed for proper functioning. The most pertinent ASTM specification for abrasion testing is ASTM D4886 and is used for geotextiles. In this test, the specimen is mounted on a stationary horizontal platform and is rubbed by an abradant (typically sandpaper) mounted on a flat block. The vertical pressure is controlled while the block containing the abradant is moved back and forth along a uniaxial path. Resistance to abrasion is expressed as a percentage of the original strength of the material. While this test is technically valid for geogrids and geomembranes, it has only been evaluated for geotextiles and a larger database of results are needed before it can be used for other materials.

2.5.4 Clogging

Clogging is an endurance property most pertinent to geotextiles. Clogging can occur over the long term as fluid flows through the geotextile carrying with it suspended particles that become lodged within the material. Physical tests have been devised and evaluated to match these long-term conditions and using site specific soils. These tests suffer from the large amount of time that it takes to conduct the test.

The gradient ratio test (ASTM D5101) has been adopted to reduce the amount of testing time associated with other more direct physical tests. The test is set-up within a vertical column with a layer of soil placed on top of a geotextile. Vertical flow is maintained through the soil–geotextile system. The hydraulic gradient is measured as the head loss divided by the flow length for two regions of the system. The first region contains the geotextile and 1 in of soil above the geotextile. The second region contains 2 in of soil and extends over a length of 1 in above the geotextile to 3 in above the geotextile. The ratio of these gradients is used to assess the clogging potential of the system, with values of three or greater indicating the potential for clogging.

The structure of the geotextile influences the possibility for the formation of a soil cake on the upstream side of the material. If gaps exist between the geotextile and the soil, soil fines tend to collect within these gaps and form a soil cake. This leads to clogging of the surface of the geotextile and is referred to as blinding. Materials with a tortuous surface, such as non-woven needle-punched materials, tend to conform more to the irregular surface of a soil, form less gaps and show less blinding (Giroud, 1994).

2.6 Degradation

Degradation of a geosynthetic results from fundamental changes of the polymer at the molecular level from its as-fabricated state. Degradation processes leading to ageing of the polymer include molecular chain scission, bond breaking, cross-linking and the extraction of components. Chemical fingerprinting methods are available that detect polymer changes: however, these methods are expensive to perform. Common and less expensive tests such as tensile strength and elongation are therefore conducted to assess the impact of these changes.

2.6.1 Temperature

Increasing the temperature has the principal effect of accelerating other degradation mechanisms. When viewed as a degradation mechanism, temperature is therefore generally associated with other mechanisms such as those involving oxidation, hydrolysis, chemical, radioactive, biological and ultraviolet (UV) light processes. High temperatures approaching the melting point of the polymer (165 °C for polypropylene and 125 °C for polyethylene) are an obvious consideration and should be avoided. Low temperatures can influence the brittleness and impact strength of geosynthetics, which influences their workability and potential for damage during installation.

2.6.2 Oxidation

Oxidation is a reactive process by which the elements of a material lose electrons when exposed to oxygen and its valence is correspondingly increased. In geosynthetics, this reaction leads to a fundamental change in the polymer and a degradation of the properties of the material. Polypropylene and polyethylene are generally the most susceptible polymers to the oxidation process. A test method used for exposing geosynthetics to the oxidation process is ASTM D794 specified for plastics. This test method uses an oven to apply heat with a continuous fresh-air flow. The test is carried out to a point where there is an appreciable change in appearance, weight, dimension or other specified properties pertinent to the application in question.

2.6.3 Hydrolysis

Hydrolysis is a process by which a chemical compound decomposes by its reaction with water. Geotextiles can experience hydrolysis degradation by internal or external yarn degradation (Hsuan *et al.*, 1993), which becomes more significant for polyester materials and for liquids with a high alkalinity. Polyamides can be affected by liquids with very low pH values. To evaluate the effect of hydrolysis, simple tests are conducted where a material is immersed in a liquid having a pH level of interest and at temperatures of 20 °C and 50 °C. The strength of the material is determined after a certain amount of immersion time and compared with initial values to detect degradation levels.

2.6.4 Chemical degradation

Chemical degradation involves the change in material properties when the geosynthetic is immersed in various chemicals of interest. ASTM D5322 describes a laboratory test procedure for immersing geosynthetics in chemical liquids. Provisions are given for controlling the temperature, the pressure and the circulation of the solution. ASTM D5496 describes a procedure for immersion of field specimens. These tests are most often used in association with geosynthetics used in landfills and as liners in reservoirs, ponds and impoundments.

2.6.5 Ultraviolet light

UV light is the component of light from the sun with wavelengths shorter than 400 nm. Photons of UV light can break down the chemical bonds (bond scission) of the polymer and lead to degradation of properties. Polyethylene is the most susceptible to UV degradation and can show a 50% strength loss within 4–24 weeks of exposure. Carbon black and other stabilizers are used to provide the polymer with UV protection, which generally means that geosynthetics that are light in colour are more susceptible. Since most geosynthetics are buried in the ground, the issue of UV degradation is important only during transport, storage and construction. During transport and storage, precautions are taken to wrap geosynthetic rolls in a protective cover to prevent UV damage.

For situations where it is important to assess the degradation of geosynthetics to long-term UV exposure, tests can be carried out by exposing geosynthetics to natural or artificial radiation. Sources of artificial radiation includes xenon arc lighting and fluorescent lighting.

2.7 Sources of further information

Information in this chapter has drawn upon several sources of material that serve as founding material for this subject. The textbook by Koerner (2006) was first published in 1986 and is generally regarded as the principal textbook on designing with geosynthetics. This textbook is an excellent reference book on geosynthetic functions, material properties, applications and design methods. Several other excellent reference sources include the books by Holtz *et al*. (1995), Ingold and Miller (1990) and Shukla (2002).

Journals devoted to geosynthetics include *Geotextiles and Geomembranes* (an official journal of the International Geosynthetics Society (IGS) published by Elsevier) and *Geosynthetics International* (an official journal of the IGS published by Thomas Telford). The Industrial Fabrics Association International annually publishes the *Geotechnical Fabrics Report (GFR)* specifier's guide, which is a directory of the property specifications of more than 500 geosynthetics from 50 international producers and is an excellent resource for designers.

International and national organizations setting and publishing testing standards include the European Committee for Standardization (Comité Européen de Normalisation, CEN) (2006), International Organization for Standardization (ISO) (2006), International Union of Laboratories and experts in Construction Materials, Systems and Structures (RILEM) (2006), Netherlands Normalisatie-instituut (NEN) (2006), Association Française de Normalisation (AFNOR) (2006), Deutches Institut für Normung (DIN) (2006), British Standards Institution (BSI) (2006), Ente Nazionale Italiano di Unificazione (UNI) (2006) and American Society of Testing and Materials (ASTM) (2006). Professional societies devoted to geosynthetics include the IGS and its associated chapters, and the Geo-Institute Committee on Geosynthetics of the American Society of Civil Engineers.

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3
The durability of geosynthetics

G. R. KOERNER, Y. G. HSUAN and R. M. KOERNER
Drexel University, USA

3.1 Introduction

Durability is a major issue for all polymeric materials, including geosynthetics, when long design lifetimes are required. Geosynthetics used in critical applications have service lifetime requirements of 30 years to hundreds of years. In some cases, failure may be dramatic and result in high cost and even loss of life. Over the last few decades, lessons have been learned from case histories and considerable research. This chapter of the book will try to answer the question: 'How long will a particular geosynthetic last?' The complicated answer is largely dependent on the polymeric type and its specific formulation as well as its *in situ* exposed environment over time.

3.1.1 Common geosynthetics

Geosynthetics are formulated materials consisting of, at the minimum, the following.

1 The resin from which the name derives.
2 Carbon black or colorants,
3 Short-term processing stabilizers.
4 Long-term antioxidants.

If the formulation changes (particularly the additives), the predicted lifetime will also change. See Table 3.1 for the most common types of geosynthetics and their approximate weight formulations. A description of the commonly used geosynthetic polymers follows.

Polyethylene

Polyethylene (PE) is a common thermoplastic polymer used throughout the world. Its name originates from the monomer ethene used to create the polymer. It can be

Table 3.1 Types of commonly used geosynthetic resins and their approximate weight percentage formulations

Type	Resin (%)	Plasticizer (%)	Fillers (%)	Carbon black or pigment	Additives (%)
Polyethylene	95–98	0	0	2–3	0.25–1
Polypropylene (flexible)	85–98	0	0–13	2–4	0.25–2
Poly(vinyl chloride)	50–70	25–35	0–10	2–5	2–5
Poly(ethylene terephthalate)	98–99	0	0	0.5–1	0.5–1
Polyamide	98–99	0	0	0.5–1	0.5–1
Polystyrene	98–99	0	0	0	1–2

produced through radical polymerization, anionic polymerization and cationic polymerization. This is because ethene does not have any extraneous groups which influence the stability of the propagation head of the polymer. Each of these methods results in a different type of PE.

PE is classified into several different categories based mostly on its physical (mainly density) and mechanical properties. The mechanical properties of PE depend significantly on variables such as the extent and type of branching, the crystal structure and the molecular weight. The following resins are commonly used in geosynthetics and classified according to ASTM D883 (ASTM International, 2000) and ASTM F412 (ASTM International, 2001) by density.

1 High-density polyethylene (HDPE), greater than 0.940 g/cm^3
2 Medium -density polyethylene (MDPE), 0.940–0.926 g/cm^3
3 Linear low-density polyethylene (LLDPE), 0.925–0.919 g/cm^3
4 Low-density polyethylene (LDPE), 0.925–0.910 g/cm^3

High-density polyethylene

HDPE has a low degree of branching and thus stronger intermolecular forces and tensile strength. The lack of branching is ensured by an appropriate choice of catalyst (e.g. Ziegler–Natta catalysts) and reaction conditions.

Medium-density polyethylene

MDPE is a branched polyethylene having a slightly lower density than HDPE. It is the most commonly used polyethylene resin in geosynthetics. It is interesting to note that only after the addition of carbon black and the additive package does the compound cross into the density threshold into HDPE.

Linear low-density polyethylene

LLDPE is a substantially linear polymer, with significant numbers of short branches, commonly made by copolymerization of ethylene with longer-chain olefins. Depending on the crystallinity and molecular weight, a melting point and glass transition may or may not be observable. The temperature at which these occur varies strongly with the type of PE.

Low-density polyethylene
LDPE has a high degree of branching, which means that the chains pack into the crystal structure as well. It has therefore less strong intermolecular forces as the instantaneous-dipole–induced-dipole attraction is less. This results in a lower tensile strength and increased ductility. LDPE is created by free-radical polymerization.

Polypropylene

Polypropylene (PP) is a common thermoplastic used throughout geosynthetics because in large part of its cost-effectiveness. PP is created through polymerization of propylene gas. It is obtained from high-temperature cracking of petroleum hydrocarbons and propane. It is not surprising that PP and PE (known collectively as polyolefins, or simply olefins) have many of the same properties. However, they differ in the following respects.

1 PP has a lower density.
2 The service temperature of PP is higher.
3 PP is harder and more rigid.
4 PP is more resistant to environmental stress cracking.
5 PP is more susceptible to oxidation and chemical attack than PE is.

There are three basic structural stereostatic arrangements of PP. They are isotactic, atactic and syndiotactic. Commercially available PP is 95% isotactic and is exclusively used in geosynthetics.

Poly(vinyl chloride)

Poly(vinyl chloride) (PVC) is a widely used polymer. In terms of revenue generated, it is one of the most valuable products of the chemical industry. Globally, over 50% of PVC manufactured is used in construction for house siding, piping, etc. As a building material, PVC is inexpensive and easy to assemble. In recent years, PVC has been replacing traditional building materials such as wood and concrete. Despite appearing to be an ideal building material, concerns have been raised about the environmental and human health costs of PVC.

PVC is produced from its monomer, vinyl chloride. PVC is a hard plastic that is made softer and more flexible by the addition of plasticizers, the most widely used being phthalates. When used as a geomembrane, plasticizer additions of 25–35% are common (Table 3.1).

Polyester

Polyester is a category of polymers or, more specifically, condensation polymers which contain the ester functional group in their main chain. Such compounds are formed by reaction of alcohols with acids via a chemical bonding known as an ester linkage. There are literally thousands of known esters which appear in many different forms. The chemical name of the polyester formed from the alcohol ethylene glycol and the acid terephthalic acid, or its derivative dimethyl terephthalate is poly(ethylene terephthalate) (PET). Although polyesters do exist in nature, polyesters generally refer to the large family of synthetic polymers.

Polyamide

A polyamide (PA) is a polymer containing monomers joined by peptide bonds. They can occur both naturally, examples being proteins such as wool and silk, and can be made artificially, examples being nylon and Kevlar. Nylon is the polymer sometimes used in geosynthetics.

Production of the monomer is accomplished when an amide link is obtained from the condensation reaction of an amino group and a carboxylic acid or acid chloride group. A small molecule, usually water, ammonia or hydrogen chloride, is eliminated.

The amino group and the carboxylic acid group can be on the same monomer, or the polymer can be constituted of two different bifunctional monomers, one with two amino groups, and the other with two carboxylic acid or acid chloride groups.

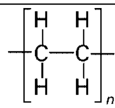
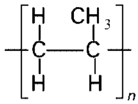
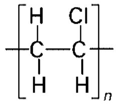
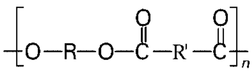
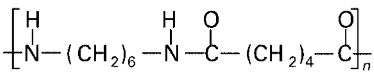
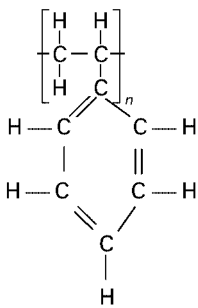
Polystyrene

Polystyrene (PS) is a polymer made from the monomer styrene, a liquid hydrocarbon that is commercially manufactured from petroleum. At room temperature, PS is normally a solid thermoplastic but can be melted at higher temperature for moulding or extrusion, then resolidified. Styrene is an aromatic monomer and PS is an aromatic polymer.

PS was first manufactured by BASF in the 1930s and is used in numerous plastic products. The most common use of PS in geosynthetics is as expanded PS, which is a mixture of about 5% PS and 95% air. This is the lightweight material in which the voids filled with trapped air give expanded PS a low thermal conductivity. It is also used as insulation in building structures. PS for architectural and engineering applications can also be extruded into forms of standard cross-sections or into sheets with various patterns. Expanded PS used to contain chloroflurocarbons but other, more environmentally safe, blowing agents are now used.

Table 3.1, from the book by Koerner (2005), illustrates the approximate formulations of commonly used geosynthetic polymers. Table 3.2, also from the

Table 3.2 Repeating units of polymers used in the manufacture of geosynthetics

Polymer	Repeating unit	Types of geosynthetic
Polyethylene		Geotextiles, geomembranes, geogrids, geopipe, geonets, geocomposites
Polypropylene		Geotextiles, geomembranes, geogrids, geocomposites
Poly(vinyl chloride)		Geomembranes, geocomposites, geopipe
Polyester [poly (ethylene tere-phthalate)]		Geotextiles, geogrids
Polyamide (Nylon 6/6)		Geotextiles, geocomposites, geogrids
Polystyrene		Geocomposites, geofoam

book by Koerner (2005), shows the repeating molecular unit from which the resins generate their names and the types of polymer from which the geosynthetics are made.

3.1.2 Degradation

Under the right set of circumstances, all materials, including the polymers listed in the above tables, will degrade over time. Various degradation mechanisms affecting polymeric materials can act in isolation or synergistically. They are as follows.

In-isolation effects

1 *Ultraviolet (UV) light degradation* occurs only when the geosynthetic is exposed to UV light. Its intensity varies depending upon the location on the globe, atmosphere and time of year.
2 *Radiation degradation* is not a factor unless the polymer is exposed to radioactive materials of sufficiently high intensity to cause chain scission, e.g. high-level radioactive waste materials.
3 *Chemical degradation* can occur in all polymers and varies from water (least aggressive) to organic solvents (most aggressive).
4 *Hydrolysis degradation* is the primary degradation mechanism for polyesters and PAs. Technically, hydrolysis is a reaction with water. That is what happens when esters are hydrolysed by aqueous solutions of various pH values. Jailloux *et al*. (1992) suggested that the alkaline hydrolysis of esters actually involves reaction with hydroxide ions.
5 *Swelling* refers to a growth in bulk as a result of the uptake or adsorption of liquids.
6 *Extraction* refers to a pull-out or withdrawal of components by means of a non-equilibrium driving force such as heat, pressure, diffusion, dispersion or convection.
7 *Delamination r*efers to a splitting apart or separation into layers.
8 *Oxidative degradation* occurs in all polymers and is the major degradation mechanism in PE and PP (Comer *et al.*, 1998).
9 *Biological degradation* is generally not a factor unless biologically sensitive additives (such as low-molecular-weight plasticizers) are included in the polymer formulation.

Synergistic effects

1 *An elevated temperature* is an enabling variable for all the previously mentioned mechanisms. The higher the temperature, the more rapid is the degradation.
2 *Applied stresses* are a complicating factor which is site specific and should be appropriately modelled in the incubation process.
3 *Multiple and/or changing mechanisms over time* need to be considered. It is unlikely that any of the previously described degradation mechanisms are acting alone and in isolation from all others. When considering long service lifetimes of 100 years and beyond (Rollin, 2004), this is almost a certainty in most applications. It is, however, an extremely difficult situation to model owing to the unpredictability of future events and actions.

Each of these mechanisms will be discussed in the subsequent section of this chapter.

3.2 Mechanisms of degradation

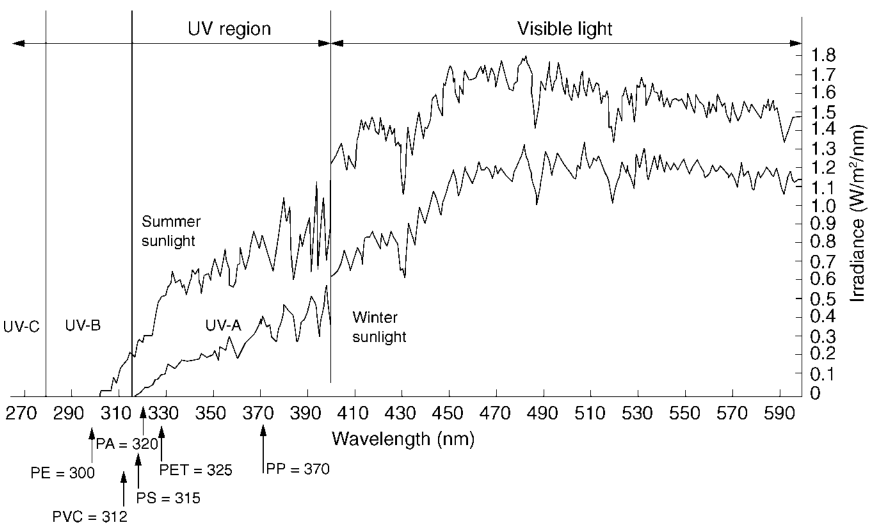
This section describes various polymer degradation processes. Each process is discussed as if it were acting in isolation. This is not indicative of general field conditions. However, it is necessary to describe the isolated events before synergistic effects can be considered and discussed in the subsequent section.

3.2.1 Ultraviolet light degradation

As shown in Fig. 3.1, the spectrum of natural light is broken into two major regions (visible and UV) according to the wavelength of solar radiation. It is well established in the polymer literature that certain wavelengths within the UV portion are particularly degrading to polymeric materials. Van Zaten (1986) mentioned the following commonly used polymers and their most sensitive wavelengths, all of which are in the UV region and are noted on Fig. 3.1.

1 PE, most sensitive wavelength = 300 nm
2 PP, most sensitive wavelength = 370 nm
3 PVC, most sensitive wavelength = 312 nm
4 Polyester, most sensitive wavelength = 325 nm
5 PA, most sensitive wavelength = 320 nm
6 PS, most sensitive wavelength = 315 nm

Furthermore, the mechanism of degradation is well understood. The light with the most sensitive wavelength enters into the molecular structure of the polymer, liberating free radicals which cause bond scission in the primary bonding of the



3.1 The wavelength spectra of visible and UV solar radiation.

polymer's backbone. This mechanism, in direct proportion to the intensity, causes a reduction in mechanical properties until eventually the polymer becomes brittle and cracks to unacceptable levels.

The above type of degradation is greatly reduced by the use of carbon black or chemically based light stabilizers. Carbon black is a finely dispersed powder of approximately micrometre size which acts as a blocking (or screening) agent to prevent the UV light from entering into the polymer structure. It also absorbs some of the energy. Its effectiveness decreases uniformly with time of exposure so that the amount and dispersion of the carbon black are important (Apse, 1989). The maximum amount, however, is limited to the amount that interferes with the growth and strength of the polymer structure. Hindered amine light stabilizers (HALS) are chemicals added to the polymer compound which react with the free radicals liberated by the UV light, preventing the propagation of degradation. When such additives are consumed, however, continued UV exposure will cause rapid degradation of the polymer. A combination of carbon black and chemical absorbers has been shown to be very effective in avoiding UV-induced degradation of polymers (Grassie and Scott, 1985).

For geosynthetic applications, a soil backfill or other covering eliminates the problem of UV degradation entirely. Only exposed geosynthetics are subjected to UV degradation and as little as 15 cm of soil cover is sufficient to prevent its occurrence. Obviously, this cover soil must be placed in a timely fashion which can be achieved in all applications except for the following.

1 Surface impoundments above the liquid level and along their horizontal run-out length.
2 Canal liners above the liquid level and along their horizontal run-out length.
3 Covers of surface impoundments, i.e. floating covers.
4 Landfill liners on side slopes which have had their surfaces exposed by erosion of cover soil and are inaccessible.
5 Exposed geomembranes on masonry, concrete and roller-compacted dams.

3.2.2 Radiation degradation

There are a number of reviews on the effects of radiation on polymer properties (Charlesby, 1960; Phillips, 1988). An extremely brief summary will be given here. The effects of γ-rays, neutrons and β-rays are essentially equivalent when their different penetrating powers are considered. β-rays (electrons) penetrate about a millimetre into a polymer, whereas γ-rays and neutrons penetrate much further. α-rays (helium nuclei) penetrate only micrometres and hence are only involved with very-near-surface damage.

The basic mechanical short-term properties of a typical polymer start to change at a total radiation dose of between 10^6 and 10^7 rad (Phillips, 1988). A rad is equivalent to 100 erg of absorbed energy per gram of material. For reference purposes, the dose of radiation lethal to a human is about 100–200 rad. Therefore it would appear that, if a geosynthetic is containing low-level nuclear waste of even lower radiation than the lethal human dose, the time before significant damage occurs to its short-term mechanical properties will be quite long. Other, more subtle changes may occur. For example, even very small amounts of local surface damage in a semicrystalline geosynthetic might cause reduction in the stress crack resistance of the material. The effects of radiation on the additives may be a more severe problem than the effect on the polymer itself. It is possible, that after a certain irradiation, the material will be more susceptible to other degradation processes.

While no test protocol exists for evaluation, some form of incubation test method can be used with suitable modifications. Whyatt and Fansworth (1990) have evaluated a number of different geomembranes in simulated short-term tests in a high-pH (about 14 wt% NaOH) inorganic solution at 90 °C and subjected them to radiation dose of up to 39×10^6 rad. It was found that only polyolefin geomembranes were unaffected by the radiation. Furthermore, the radiation did not have a significant effect on other chemical degradation rates.

3.2.3 Chemical degradation

The reaction of various geosynthetics to chemicals has probably been studied more than any other degradation mechanism. Most of the work is laboratory orientated via simple immersion tests but the body of knowledge is so great that a reasonable confidence level can be associated with manufacturers' listings and recommendations.

Polymer chains are linked together by weak interchain interactions. In order to avoid alteration, interactions between the chains must be stronger than between the solvent and the polymer. Such polymer–solvent interactions are generally based on polarity. The higher the polarity of the solvent and polymer, the stronger are the possible interactions. This explains why polymers with low polarity such as PE and PP are resistant to a vast array of chemicals. When solvents penetrate polymers, they begin to break the interaction between polymer chains, increasing the distance between them and reducing their attraction, which increases their mobility. This typically leads to swell and softening of the material, opening it up for further attack.

Neat chemicals yield insight into possible chemical interaction but they are far from real-world performance. Complex waste streams such as leachate need to be evaluated and are usually addressed on a site-specific basis. For this reason, the US Environmental Protection Agency (1982, 1984, 1987) developed procedures which are now embodied in ASTM D5322 (ASTM International, 1998b). In this method, samples of the candidate geosynthetic are exposed at 23 and 50 °C and removed at 30, 60, 90 and 120 days. Various physical and mechanical tests are performed and then compared with the unexposed material, e.g. ASTM D5747

(ASTM International, 1995) for geomembranes. A percentage change in this behaviour is determined. When plotted for the various exposure times, trends can be established and a decision made as to the nature and degree of chemical degradation.

Depending on the type of leachate *vis-à-vis* the polymeric compound from which the geosynthetic is made, a number of variations may occur.

1 No change may occur, which indicates that the material is resistant to the leachate at least for the time periods and temperature evaluated.
2 Swelling of the geosynthetic may occur, which in itself may not be significant. Many polymers can accommodate liquid in their amorphous regions without a sacrifice of physical or mechanical properties. Swelling, however, is often the first stage of subsequent degradation and small losses in modulus and strength may occur. The effect is often reversible when the liquid is removed.
3 A nominal, but statistically significant, change in a physical or mechanical property, of course, signifies some type of chemical reaction. The variations are enormous. Quite often the elongation at break in a tensile test will be the first property to show signs of change. It will first occur with the 50 °C incubation data, since this can be considered to be an accelerated temperature test over the 23 °C incubation data.
5 A large change in a physical or mechanical property signifies an unacceptable performance of the material. Limits of acceptability are, however, very subjective. O'Toole (1985–1986), Little (1985) and Koerner *et al.* (1990) suggested recommendations and there are also several expert computer codes available to aid in the decision.

3.2.4 Hydrolysis degradation

Polyester (PET) fibres have been extensively studied to understand the effects of various chemicals with regard to hydrolysis (Risseeuw and Schmidt, 1990; Schmidt *et al*., 1994; Salman *et al*., 1997). Polyester is a polymer where the individual units are held together by ester linkages. In condensation polymerization, when the monomers join together a small molecule is lost. This is different from addition polymerization which produces polymers such as PE in which nothing is lost when the monomers join together.

Hydrolysis is a chemical reaction in which a substance reacts with water and becomes a changed substance. This involves the ionization of the water molecule, as well as splitting of the compound hydrolysed. The chemical process in which scission of a chemical bond occurs via reaction with water is accelerated by extreme acidic or alkali environments.

The polyester-chemical-bond polymers are subject to hydrolysis, thereby producing alcohol and acid end groups. Hydrolysis is a reversible reaction, meaning that the alcohol and acid groups can react with each other to produce a polyester bond and water as a by-product. In practice, however, a degraded polyester fibre will never fully reconstruct back to its original integrity if left in its *in situ* environment.

Factors affecting the rate and amount of hydrolysis depend upon the following.

1 Textile geometry and structure (Halse *et al.*, 1987a, 1987b).
2 Yarn and/or filament cross-section (Collins *et al.*, 1991).
3 Heat setting temperature (Solbrig and Obendorf, 1991).
4 Additives in the fibre (Sanders and Zeronian, 1982).
5 Presence of co-monomers (McIntyre, 1985).
6 Applied stress (Rahman and Alfaro, 2004).

A study sponsored by the US Federal Highway Administration found that polyester geosynthetics are sensitive to hydrolytic degradation at elevated pH levels (Elias, 1998). Elias found that hydrolytic degradation of polyester increased 2.4 times when pH values increased from 7 to 10. Polyester deterioration via hydrolysis will be catalysed by either acids or bases. The reaction rate is also sensitive to temperature. Polyester can be made hydrolysis resistant by increasing its molecular weight above a minimum number molecular weight Mn of 25 000 (Geosynthetic Research Institute, 1999) and reducing the number of carboxyl end groups below a threshold level of 30 (Geosynthetic Research Institute, 1998).

3.2.5 Swelling

One indication of a geosynthetic's durability is the amount of swelling that occurs owing to liquid absorption. It should be emphasized that swelling *per se* does not necessarily mean chain scission nor a failed system. It is, however, slightly disconcerting and usually results in changes in physical and mechanical properties, at least on a temporary basis.

The test for water absorption, which can be modified for any liquid, has been given in ASTM D570 (ASTM International, 1998a). The test is directed at a quantitative determination of the amount of water absorbed, but it is also used as a quality control test on the uniformity of the finished product. The test procedure cautions that the liquid absorption may be significantly different through the edge or through the surface, particularly with laminated products. (This fact alone suggests that in seaming of laminated geomembranes, the upper overlap must be protected against moisture uptake.) Test specimens of 75 mm by 25 mm are used and immersed in a number of possible ways.

1 Under constant immersion for 2 h, 24 h or 2 weeks in 23 °C water.
2 Under cyclic (repeated) immersion.
3 Under constant immersion for 0.5 h or 2 h in 50 °C water.
4 Under constant immersion for 0.5 h or 2 h in boiling water.

The resulting test data are reported as the percentage increase in weight using deionized and distilled water. Some typical values for commonly used geomembranes are as follows (Haxo *et al.*, 1985).

1 PE, negligible.
2 PP, negligible.
3 PVC, 3–4%.
4 PET, 0.5–1%.
5 PA, 0.5–1%.
6 Non-expanded PS, 0.2– 0.7%.

Swelling due to other liquids has been mentioned in the reference cited. It is both liquid and condition dependent.

3.2.6 Extraction

Some polymers exhibit degradation by the long-term extraction of one or more components of the formulation from the polymeric material. These are usually polymers which have been compounded with the use of plasticizers and/or fillers. The as-formulated and compounded mixture of such polymers is very intricate and the bonding mechanic is very complex. When extraction of plasticizers does occur, a sticky surface results with the remaining structure showing signs of increased modulus and strength, and a lowering of the elongation at failure, i.e. the material becomes progressively brittle (Doyle and Baker, 1989). The long-term behaviour, however, is unknown. It is also possible that antidegradation components within the polymer may be extracted and leach out to the surface. This might indicate that the remaining polymer is somewhat more sensitive to long-term degradation.

Over time, plasticizers can migrate from PVC by contact with air, liquid or adsorbent solids. This can result in reduced flexibility, shrinkage and even cracking. The plasticizers used in PVC are either polymers or monomers. Monomeric plasticizers are more commonly used because of their cost-effectiveness; see Miller *et al*. (1991), Hammon *et al*. (1993) and Giroud and Tisinger (1994) for more detail in this regard. To guard against a significant amount of plasticizer migration, a minimum molecular weight of the plasticizer is generally specified (Stark *et al*., 2005). The plasticizer mobility, commonly related to molecular weight, is one of the main factors in the diffusion of plasticizer out of the polymer structure. Nass and Heiberger (1986), Kays (1988) and Wilson (1995) suggested that linearity, polymer morphology, polarity and relative amount of the plasticizer may also be factors that control plasticizer retention. Scuero (1990) and Cazzuffi (1998) showed that there exists many PVC formulations, with quite different durability characteristics.

3.2.7 Delamination

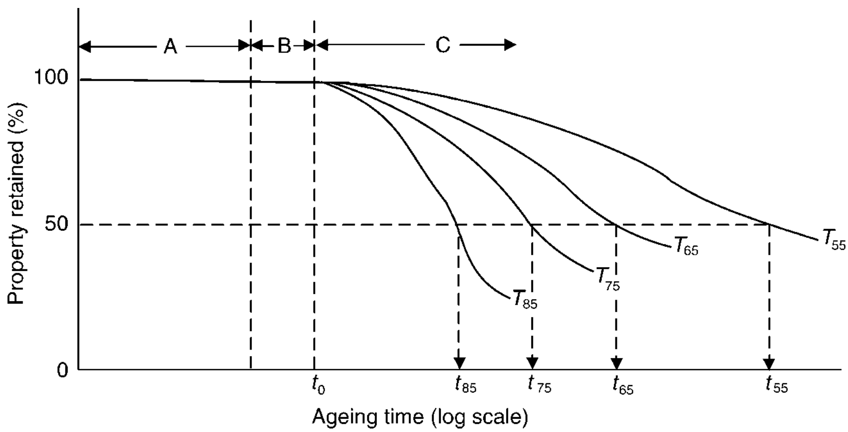
For geosynthetics which are manufactured in individual layers, or plies, delamination is a possibility. Calendering or spreading the coating are the usual production methods. Delamination is observed when liquid enters into the edge of the unprotected material and is drawn into the interface by capillary tension. This can occur between plies, within reinforcing scrim or between the coating of a fabric substrate. When it occurs, the individual components are separated and composite action is lost. This type of wicking action has been problematic in the past but current manufacturing methods and proper construction quality control and construction quality assurance in field operations have almost eliminated the situation.

3.2.8 Oxidative degradation

Whenever a free radical is created, e.g. on a carbon atom in the PE chain, oxygen can create degradation. The oxygen combines with the free radial to form a hydroperoxy radical, which is passed around within the molecular structure. It eventually reacts with another polymer chain creating a new free radical causing chain scission. The reaction generally accelerates once it is initiated.

Antioxidants are added to the compound to scavenge these free radicals in order to halt, or at least to interfere with, the process. These additives, or stabilizers, are specific to each type of resin. This area is quite advanced with all resin manufacturers being involved in a meaningful and positive way. The specific antioxidants are usually proprietary. Removal of oxygen from the geosynthetic's surface, of course, eliminates the concern. Thus, once placed and covered with waste, or liquid, degradation by oxidation is greatly retarded but generally not eliminated. Conversely, exposed material or those covered by non-saturated soil will always be susceptible to the mechanism.

Oxidation as it is related to olefins can further be thought of in distinct lifetime stages (Müeller and Jacob, 2003), as shown in Fig. 3.2.



3.2 Three conceptual stages in the chemical ageing of polyolefin geomembranes: A, antioxidant depletion time; B, induction time; C, 50% property degradation time (the half-life).

Stage A is called the antioxidant time. In this regard, the purposes of antioxidants are, firstly, to prevent polymer degradation during processing and, secondly, to prevent oxidation reactions from taking place during service life. Obviously, there can only be a given amount of antioxidants in any formulation. Once the antioxidants are completely depleted, additional oxygen will begin to attack the polymer chains, leading to subsequent stages as shown in Fig. 3.2. The duration of the antioxidant depletion stage depends on both the type and the amount of antioxidants.

The depletion of antioxidants is the consequence of two processes.

1 Chemical reactions with the oxygen diffusing into the geomembrane.
2 Physical loss of antioxidants from the geomembrane.

The chemical process involves two main functions: the scavenging of free radicals converting them into stable molecules, and the reaction with unstable hydroperoxide (ROOH), forming a more stable substance. Regarding physical loss, the process involves the distribution of antioxidants in the geomembrane and their volatility and extractability.

Hence, the rate of depletion of antioxidants is related to the type and amount of antioxidants, the service temperature and the nature of the site specific environment. See Hsuan and Koerner (1998) for additional details.

Stage B is called the induction time. It is relatively short, since in a pure polyolefin resin, i.e. one without carbon black and antioxidants, oxidation occurs extremely slowly at the beginning, often at an immeasurable rate. Eventually, oxidation occurs more rapidly. The reaction eventually decelerates and once again becomes very slow. This progression is illustrated by the S-shaped curve of Fig. 3.3(a). The initial portion of the curve (before measurable degradation takes place) is called the induction period, or induction time, of the polymer. In the induction period, the polymer reacts with oxygen, forming hydroperoxide (ROOH), according to

$$RH \rightarrow R\bullet + H\bullet \qquad [3.1]$$

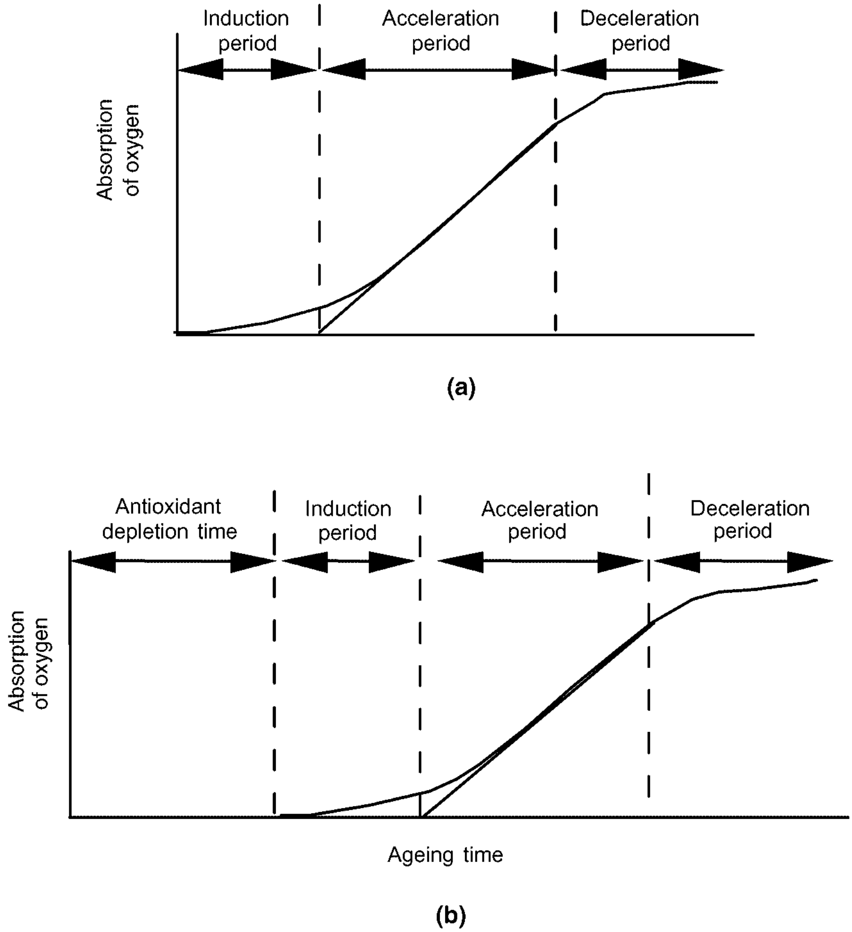
(aided by energy or catalyst residues in the polymer),

$$R\bullet + O_2 \rightarrow ROO\bullet \qquad [3.2]$$

$$ROO\bullet + RH \rightarrow ROOH + R\bullet \qquad [3.3]$$

In the above equations, RH represents the polyethylene polymer chains and the symbol • represents free radicals, which are highly reactive molecules. However, the amount of ROOH in this stage is very small and the hydroperoxide does not further decompose into other free radicals which inhibits the onset of the acceleration stage.

In a stabilized polymer such as a polymer with antioxidants, the accelerated oxidation stage takes an even longer time to be reached. The antioxidants create an additional depletion time stage prior to the onset of the induction time, as shown in Fig. 3.3(b).



3.3 Curves illustrating various stages of oxidation: (a) unstabilized polyethylene; (b) stabilized polyethylene.

In *Stage C*, the physical and mechanical properties begin to degrade as oxidation continues and additional ROOH molecules are being formed. Once the concentration of ROOH reaches a critical level, decomposition of ROOH begins, leading to a substantial increase in the amount of free radicals, according to

$$ROOH \rightarrow RO\bullet\ OH\bullet \text{ (aided by energy)} \quad [3.4]$$
$$RO\bullet + RH \rightarrow ROH + R\bullet \quad [3.5]$$
$$OH\bullet + RH \rightarrow H_2O + R\bullet \quad [3.6]$$

The additional free radicals rapidly attack other polymer chains, resulting in an accelerated chain reaction, signifying the end of the induction period (Rapoport and Zaikov, 1986). This indicates that the concentration of ROOH has a critical control on the duration of the induction period.

The oxidation produces a substantial amount of free radical polymer chains (R•), called alkyl radicals, which can proceed to further reactions, leading to either cross-linking or chain scission in the polymer. As the degradation of polymer continues, the physical and mechanical properties of the polymer start to change. The most noticeable change in physical properties is in the melt index, since it relates to the molecular weight of the polymer. As for the mechanical properties, both tensile break stress (strength) and break strain (elongation) decrease. Ultimately, the degradation becomes so severe that all tensile properties start to change (tear, puncture, burst, etc.) and the engineering performance is jeopardized. This signifies the end of the so-called 'service life' of the geomembrane.

Although quite arbitrary, the limit of service life of polymeric materials is often selected as a 50% reduction in a specific design property. This is commonly referred to as the half-lifetime, or simply the 'half-life'. It should be noted that even, at half-life, the material still exists and can function, albeit at a decreased performance level with a factor of safety lower than the initial design value.

3.2.9 Biological degradation

Within the various plant forms of biological life, i.e. bacteria, antinomycetes, fungi, and algae, polymer degradation is essentially impossible owing to the high molecular weight of the common resins used in geomembranes. In order for such degradation to occur, the chain ends must be accessible and this is highly unlikely for molecular weights greater than 1000, let alone 10 000–30 000, which is common for geomembrane resins. Biological degradation might be possible for plasticizers or additives compounded with the resin, but information is not authoritative on this subject.

Within the higher forms of biological life, i.e. protozoa, spiders, insects, moles, rats and small mammals, polymers do not contain food and thus are unlikely to be consumed. It is possible, however, that an animal may try to penetrate the synthetic for access to the opposite side. In this case, hardness of the predator's teeth enamel versus the geomembrane's hardness is the key comparison. While such events are possible, authoritative information is not known to the present authors.

Verification of biological resistance is confirmed by a soil, sewage or sludge burial test. It is usually carried out for long exposure times, at a nearly neutral pH and at an elevated temperature. The test specimens are periodically removed from the soil and tested for changes in properties. The extent of the degradation is also examined by way of surface microscopy and various fingerprinting techniques.

3.3 Synergistic effects

While not degradation mechanisms within, or of, themselves, there are several phenomena which can readily work in conjunction with the previously discussed items. They generally have the effect of accelerating the specific degradation process and thus are called 'synergistic effects'. At the outset it should be noted, however, that the quantification of these effects is very complicated and the database is very weak in this regard.

3.3.1 Elevated temperatures

Whenever the temperature at the surface or within the geosynthetic is increased, the material expands, chains deform, the mobility of the polymer (and of its other ingredients) is increased, and degradation is usually accelerated. All can result in internal chain reorganization in the presence of a driving force. The stressed chain segments can move and reorganize themselves in order to reach a more stable state. Such movements may lead to morphological modifications which can lead to material degradation. Of the different degradation mechanisms mentioned earlier, this is the case for all of them with the possible exception of biological degradation, which was seen to be of negligible significance. Clearly, elevated temperatures accelerate UV degradation; this phenomenon is arguably the most important and has the largest database. Thus extreme conservatism is usually taken when testing for UV degradation. As mentioned earlier, chemical resistance incubation is usually carried out at an elevated temperature of 50 °C for comparison with the 23 °C 'standard' temperature. Invariably, the higher temperature produces results having greater changes than the lower temperature does. There is an upper limit for such temperature testing, however, and that value is based upon polymer modification not representative of realistic behaviour. Its value is undoubtedly resin dependent but largely unknown.

For geosynthetics placed in the field, high temperatures can generally be avoided by covering the geosynthetics with soil, liquid or another material. Thus, the buried environment greatly reduces temperatures in most synthetic applications. Notable exceptions are surface impoundment and canal liners (above the liquid surface), floating covers and exposed dam waterproofing. For all these cases, simulated testing is absolutely necessary (Sangam and Rowe, 2002).

For an accelerated simulation of direct sunlight using a laboratory weatherometer, one usually considers a worst-case situation which is the solar maximum condition. This condition consists of global noon sunlight, on the summer solstice, at normal incidence. It should be recognized that UV-A range is the target spectrum for a laboratory device to simulate the naturally occurring phenomenon (Hsuan and Koerner, 1993; Suits and Hsuan, 2001).

The xenon arc weatherometer [ASTM G155 (ASTM International, 2005)] was introduced in Germany in 1954. There are two important features: the type of filters and the irradiance setting. Using a quartz inner and borosilicate outer filter results in excessive low-frequency wavelength degradation. The more common borosilicate inner and borosilicate outer filters shows a good correlation with solar maximum conditions, although there is an excess of energy below 300 mm wavelength. Irradiance settings are important adjustments in shifting the response although they do not eliminate the portion of the spectrum below 300 nm frequency. Nevertheless, the xenon arc weatherometer is a commonly used method for exposed lifetime prediction of all types of geosynthetic.

UV fluorescent lamps (ASTM G154 (ASTM International, 2004b)) are an alternative type of accelerated laboratory test device which became available in the early 1970s. They reproduce the UV portion of the sunlight spectrum but not the full spectrum as in xenon arc weatherometers. Earlier FS-40 and UV-C-313 lamps gave reasonable short wavelength output in comparison with solar maximum. The UV-A-340 lamp was introduced in 1987 and its response is seen to reproduce UV light quite well. This device (as well as other types of weatherometer) can handle elevated temperature and programmed moisture on the test specimens. Such investigations are routinely being conducted around the world and are commonly specified.

3.3.2 Applied stresses

Invariably, the testing for geosynthetic degradation is performed on unstressed laboratory samples. Yet, at the very least, the geosynthetics will have compressive stresses imposed and, quite possibly, tensile stresses as well. What these stresses do to the degradation of the geosynthetics in comparison with testing unstressed samples is largely unknown. The reason for this lack of data is obvious. The cost of experimentation at elevated temperatures is very high in itself and to stress the material in some simulated form of biaxial or triaxial stress would generally be cost prohibitive. Yet, some experimentation with stressed geosynthetics is being initiated, most of which are trying to identify the severity of the effect.

Environmental stress cracking consists of a brittle failure of stressed samples in the presence of a wetting agent. It is of concern when geosynthetics are made of HDPE. This type of failure differs from ductile failure (creep) in that, despite the stress applied on a fairly large area, the deformation only takes place within a thin cross-section and ultimately leads to complete brittle rupture of the material. In addition, the stress level involved in this mechanism is generally less than half the yield stress of the material (Hsuan, 2000). With the advent of ASTM D5397 (American Society for Materials and Testing, 1999) this problem has largely been eliminated from our marketplace. The GRI-GM13 specification (Geosynthetic Research Institute, 1997) requires a 300 h failure time, which requires a quality resin combined with a good additive package.

3.3.3 Multiple and/or changing mechanisms over time

Long exposure results in a multiplicity of effects such as those due to UV, extraction and oxidation, which can result in synergistic effects beyond the previously discussed phenomenon taken individually. For materials as inert as PE, for example, exposure for years at ambient temperature shows no indication of any change in properties. For other polymers, some changes in surface texture or even in macroscropic properties might occur, but their influence on the geosynthetic's behaviour is not clear.

The major authoritative database on long-term ageing is available from Matrecon, Inc. (1988), but the steady development of new polymers and compounds makes the situation elusive, to say the least. It should be mentioned that a few landfill owners are beginning to place geosynthetic samples (coupons) in retrievable locations for annual exhuming and evaluation. Such studies will eventually be helpful in assessing actual degradation and ageing although the coupons are rarely, if ever, in a stressed condition.

3.4 Accelerated testing methods

Clearly, the long time frames involved in evaluating individual degradation mechanisms at field-related temperatures, compounded by the synergistic effects just mentioned, are not providing answers regarding geosynthetic durability behaviour fast enough for the long-term decision-making processes of today. Thus accelerated testing, by high stress, elevated temperature and/or aggressive liquids, is very compelling. Before reviewing these procedures, however, it must be clearly recognized that one is assuming that the high stress, elevated temperature or aggressive liquids used actually simulates extended lifetimes – an assumption which is not readily substantiated. Thus it might be that the test procedures to be described here actually form lower-bound conclusions in predicting degradation, i.e. the results may be minimum values but that is not known with any degree of certainty.

3.4.1 Stress limit testing

Focusing almost exclusively on HDPE pipe for natural gas transmission, the Gas Research Institute, the Plastic Pipe Institute and the American Gas Association are all very active in various aspects of plastic pipe research and development. The three above-mentioned organizations, together with Battelle Columbus Laboratories sponsor the Plastic Fuel Gas Symposia which are held on a biennial basis and the resulting *Proceedings* contain many interesting papers. Stress limit testing in the plastic pipe area has proceeded to a point where there are generally accepted testing methods and standards. ASTM D1598 (ASTM International, 2002) describes a standard experimental procedure, and ASTM D2837 (ASTM International, 2004a) gives guidance on the interpretation of the results of the ASTM D1598 test method.

In ASTM D1598 (ASTM International, 2002), long pieces of unnotched pipe are sealed, capped and placed in a constant-temperature environment. Room temperature of 23 °C is usually used. The pipes are placed under various internal pressures which mobilize different values of hoop stress in the pipe walls, and the pipes are monitored until failure occurs. This is indicated by a sudden loss in pressure. Then the values of hoop stress are plotted versus failure times on a log–log scale. If the plot is reasonably linear, a straight line is extrapolated to the desired, or design, lifetime which is often 10^5 h or 11.4 years. The stress of this failure time reduced by an appropriate factor is called the hydrostatic design basis stress. While of interest for pipelines, the stress state of planar geosynthetics is essentially unknown and is extremely difficult to model. Thus, the technique is not of direct value for synthetic design. It leads, however, to the next method.

3.4.2 Rate process method for pipes

Research at the Gas Institute in Holland (Wolters, 1987) uses the method of pipe ageing that is most prevalent in Europe. It is also an International Standards Organization (ISO) tentative standard currently in committee. Note that other plastic pipe research institutes also are involved in this type of research. The experiments are again performed using sections of unnotched pipe which are tightly capped, but now they are placed in various elevated-temperature environments. So as to accelerate the process, elevated-temperature baths up to 80 °C are used. Different pressures are put in the pipes at each selected temperature so that known hoop stress occurs in the pipe walls. The pipes are monitored until failure occurs, resulting in sudden loss in pressure. Two distinct types of failure are found: ductile and brittle. The failure times corresponding to each applied pressure are recorded. A response curve is presented by plotting hoop stress against failure time on a log–log scale.

The rate process method is then used to predict a failure curve at some temperature other than those tested, i.e. at a lower (field-related) temperature than was evaluated in the high-temperature tests. This method is based on an absolute reaction rate theory as developed by Tobolsky and Eyring (1943) for the viscoelastic phenomenon. Coleman (1956) has applied it to explain the failure of polymeric fibres as used in geotextiles and geogrids. The relationship between failure time and stress is expressed in the following form:

$$\log t_f = A_0 T^{-1} + A_1 T^{-1}\sigma \quad [3.7]$$

where

t_f = time to failure
T = temperature
σ = tensile stress on the fibre
A_0, A_1 = constants

Bragaw (1983) has revised the above model on polymeric fibres and found three additional equations which yield reasonable correlation to the failure data of HDPE pipe. These three equations are

$$\log t_f = A_0 + A_1 T^{-1} + A_2 T^{-1} P \qquad [3.8]$$

$$\log t_f = A_0 + A_1 T^{-1} + A_2 \log P \qquad [3.9]$$

$$\log t_f = A_0 + A_1 T^{-1} + A_2 T^{-1} \log P \qquad [3.10]$$

where

P = internal pipe pressure proportional to the hoop stress in the pipe

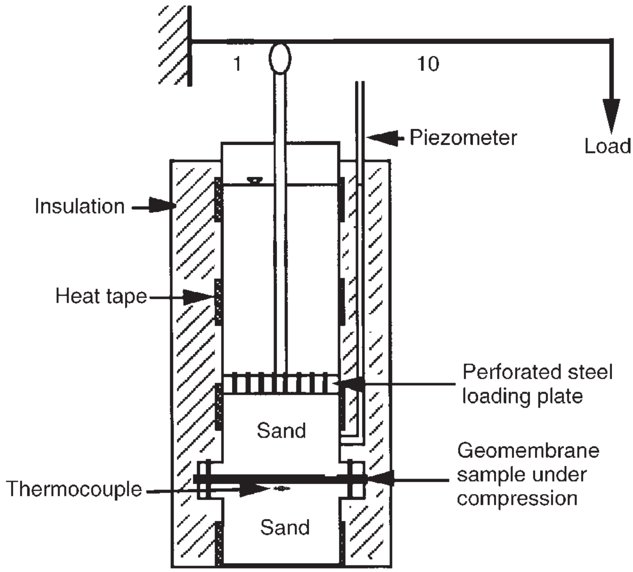
The application of the rate process method requires a minimum of two experimental failure curves at different elevated temperatures well above 40 °C. The equation which yields the best correlation to these curves is then used in the prediction procedure for a response curve at a field-related temperature, e.g. from 10 to 25 °C. Two separate extrapolations are required: one for the ductile response and one for the brittle response. Three representative points are chosen on the ductile regions of the two experimental curves. One curve will be selected for two points, and the other for the remaining point. These data are substituted into the chosen equation, i.e. Equation [3.7], [3.8] or [3.9] to obtain the prediction equation for the ductile response of the curve at the desired (lower) temperature. The process is now repeated for the predicted brittle response curve at the same desired temperature. The intersection of these two lines defines the transition time.

3.4.3 Rate process method for geomembranes

A similar rate process method to that just described for HDPE pipes can be applied to HDPE geomembranes. The major difference is the method of stressing the material. The geomembrane tests are performed using a notched constant-load test which follows ASTM D5397 (ASTM International, 1999). In this test, dumbbell-shaped specimens are taken from the geomembrane sheet. A notch is introduced on one of the surfaces, the notch depth being 20% of the thickness of the sheet. The full description of the notching process has been described by Halse *et al*. (1990). Tensile loads varying from 30 to 70% of the yield stress of the sheet are applied to the notched specimens. The tests are performed in constant-elevated-temperature environments (usually from 40 to 80 °C) and in a surface-active wetting agent. In general, 10% Igepal and 90% tap water are used. The data are presented by plotting percentage yield stress against failure time on a log–log scale. Distinct ductile and brittle regions can be seen, together with a clearly defined transition time (Halse *et al*., 1990).

3.4.4 Elevated-temperature and Arrhenius modelling

Using experimental chambers as shown in Fig. 3.4(a) and Fig. 3.4(b), Mitchell and Spanner (1985) have superimposed a compressive stress, chemical exposure, elevated temperature and long testing time onto a single experimental device. The Geosynthetic Institute has extended their work evaluating HDPE geomembranes



(a)

(b)

3.4 (a) Cross-section of the experimental chamber; (b) photograph of multiple cells maintained at various constant temperatures.

at 85, 75, 65 and 55 °C. At the end of the arbitrarily designated test period, the geomembrane samples are removed. Mechanical tests and chemical analyses are then performed on these incubated samples to monitor whether any changes in the various properties of the geomembrane occurred. The mechanical tests include the following.

1 Tensile strength and elongation.
2 Yield strength and elongation.
3 Stress cracking behaviour.

The chemical analysis tests include the following.

1 Differential scanning calorimetry, for measuring the crystallinity and oxidation induction time.
2 Infrared spectrometry, for measuring the concentration of carbonyl groups.
3 Gel permeation chromatography, for measuring the molecular weight and molecular weight distribution

If there are changes in any of the above properties, e.g. in the concentration of the carbonyl group, the reaction rate K is obtained for each experimental test temperature T. These values were now used with the Arrhenius equation, which is as follows (American National Standards Institute, 1986):

$$K = \exp\left(-\frac{E}{RT}\right) \qquad [3.11]$$

where

K = reaction rate for the process considered
A = constant for the process considered
E = reaction activation energy for the process considered
T = temperature (K = °C + 273)
R = gas constant (=8.314 J/mol K)

By plotting ln K against 1/T, a straight line is obtained. The slope of this line is E/R for the particular property change being monitored. The constant A can also be identified but it drops out of the equation when comparing the responses at two different temperatures.

We can now extrapolate graphically to a lower site-specific temperature. The essential equation for the extrapolation is

$$\frac{r_{T_{\text{test}}}}{r_{T_{\text{site}}}} = \exp\left[-\frac{E_{\text{act}}}{R}\left(\frac{1}{T_{\text{test}}} - \frac{1}{T_{\text{site}}}\right)\right] \qquad [3.12]$$

where

E_{act}/R = slope of Arrhenius plot.
T_{test} = incubated (high) temperature
T_{site} = site-specific (lower) temperature

Using experimental data from Martin and Gardner (1983) for the half-life of the tensile strength of a polybutylene terephthalate plastic, the E_{act}/R value is –12 800 K. The estimated life, extrapolating from the 93 °C actual incubation temperature (which took 300 h to complete) to a site-specific temperature of 20 °C can be solved as follows.

After converting from degrees Celsius to Kelvin,

$$\frac{r_{93\,°C}}{r_{20\,°C}} = \exp\left[-\frac{E_{act}}{R}\left(\frac{1}{93+273} - \frac{1}{20+273}\right)\right]$$

$$= \left[-12\,800\left(\frac{1}{366} - \frac{1}{293}\right)\right]$$

$$= 6083 \qquad [3.13]$$

If the 93 °C reaction takes 300 h to complete, the equivalent 20 °C reaction would take

$$\begin{aligned} r_{20\,°C} &= 6083\ (300) \\ &= 1\,825\,000\ \text{h} \\ &= 208\ \text{years} \end{aligned}$$

Thus, the predicted time for this particular polymer to reach 50% of its original strength at 20 °C is approximately 200 years, its predicted lifetime for Stage C.

Table 3.3 takes the bold step of superimposing lifetime prediction from the three previously discussed stages and makes use of data from Viebke *et al*. (1994) and Martin and Gardner (1983) and much from the Geosynthetic Institute. The table also shows that temperatures higher than 20 °C will cause the lifetime to decrease exponentially. At 40 °C, the predicted lifetime of the same covered geomembrane would be approximately 80% less than at 20 °C. *In situ* temperatures of landfill liners and covers (for both dry and wet landfills) is an ongoing research project at the Geosynthetic Insitute. Koerner and Koerner (2005) gave recent data from facilities that have been monitored for over 13 years.

3.5 Summary and conclusion

This chapter on durability of geosynthetics has attempted to give insight into the long-term performance of these polymeric materials by itemizing those mechanisms that can degrade the resin and/or compound from which they are made. It presents various degradation mechanisms, taken individually, and then describes possible synergistic effects induced by anticipated field conditions. These effects greatly complicate the situation.

Long-term laboratory tests under simulated field conditions are absolutely essential for the future development and improvement of geosynthetics in this

Table 3.3 Lifetime prediction of a backfilled HDPE geomembrane as a function of *in situ* service temperature[a]

In-service temperature (°C)	Stage A (years)		Stage B (years)	Stage C (years)			Total prediction[b] (years)
	Standard oxidation induction time	High-pressure oxidation induction time		Viebke *et al.* (1994)	Martin and Gardner (1983)	Geosynthetic Institute data	
20	200	215	30	740	208	8	555
25	135	144	25	441	100	7	348
30	95	98	20	259	49	6	221
35	65	67	15	154	25	5	142
40	45	47	10	93	13	4	93

[a]Exposed geomembrane lifetimes are considerably less than values in this table.
[b]Total = Stage A (average) + Stage B + Stage C (average)

area. Simulated stress tests under elevated-temperature testing and Arrhenius modelling is clearly the best route in this regard. Answers to the important question 'how long will they last?' may never be known unless such efforts are embraced and the result shared with the community at large.

3.6 Acknowledgements

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4

National and international standards governing geosynthetics

L. D. SUITS
North American Geosynthetics Society, USA

4.1 Why standardization?

In any discussion concerning the development of any of the various types of standard for geosynthetics, the inevitable question is: 'why do we need standards?' There are several reasons that can be given.

1 In relation to testing, standards provide a consistent way to evaluate geosynthetics so that the end user can be assured that, irrespective of the laboratory that performs the testing, the results received will be within the acceptable statistical range for the material. In other words, the same result, within the statistical bounds of the standard, will be obtained irrespective of where tested, as long as the established protocol is followed.
2 The second point is where the controversy starts. Standardizing testing methods, material specifications, standards of practices, etc. allows for continued innovation. By standardizing those areas that can be standardized, it allows concentration on the development of new materials, and new ideas of application because of the first point above. In other words, with standardization, one does not have to concentrate on explaining variations in results of different testing methods for the same characteristic. Of course, this is predicated on the understanding that the standards are being followed as written.

4.2 Types of standard

The *Form and Style for ASTM Standard Manual*, (ASTM International, 2005a) defines four types of standard.

1 Standard test methods.
2 Standards of practice.
3 Standard guides.
4 Standard specifications.

The *ASTM Standard Manual* defines each as follows.

4.2.1 Standard test method

A standard test method provides detailed directions on performing specific tests, which produce a test result, e.g. ASTM D4491 *Standard Test Method for the Water Permeability of Geotextiles by the Permittivity Method*. The standard test method allows results of like tests performed by different laboratories to be compared. If performed as written, they will provide similar results within the statistical bounds of the method. The standard test method provides the basis for laboratory accreditation programmes. When accredited in a test method, not only has the laboratory demonstrated their ability to perform the test as written, but also that they have a quality assurance programme, perform routine maintenance of equipment and keep accurate records regarding testing and maintenance.

4.2.2 Standard of practice

A standard of practice is a compendium of information or series of options that does not recommend a specific course of action. A standard of practice is aimed at increasing the awareness of information and approaches in a given subject area, e.g. ASTM D4354 *Practice for Sampling of Geosynthetics for Testing*.

4.2.3 Standard guide

A guide is different from the previous two standards in that it provides a definitive set of instructions for performing one or more very specific operations that do not produce a test result, e.g. ASTM D4873 *Guide for Identification, Storage and Handling of Geosynthetic Rolls and Samples*.

4.2.4 Standard specification

A standard specification is an explicit set of requirements to be satisfied by a material, product, system or service. As related to geosynthetics, this may be divided into two subcategories.

1 Material property requirements.
2 Material performance.

Material property requirements

Material properties are set for two reasons: firstly, to establish purchasing criteria and, secondly, to provide the means for the purchasing organization to accept materials for use on their construction projects, e.g. ASTM D6817 *Standard Specification for Rigid Cellular Polystyrene Geofoam*.

Table 4.1 Standards development organizations

Country	Organization	Abbreviation
Austria	Österreichisches Normungs institut	ON
Australia	Standards Australia	SA
Belgium	Institut Belge de Normalisation	IBN
Canada	Standards Council of Canada	SCC
Denmark	Dansk Standard	DS
France	Association Française de Normalisation	AFNOR
Germany	Deutsches Institut für Normung	DIN
Italy	Ente Nazionale Italiano di Unificazione	UNI
Japan	Japanese Industrial Standards Committee	JISC
Korea	Korean Agency for Technology and Standards	KATS
Turkey	Turk Standards Iari Enstitusu	TSE
UK	British Standards Institution	BSI
USA	American National Standards Institute	ANSI

Material performance

A material performance specification establishes a level of expected performance of a material to ensure the long term performance of the material. The American Association of State Highway and Transportation Officials (AASHTO) M-288 *Specification for Geotextiles* is aimed at this.

4.3 Standards development organizations

Within each country there may be one or more organizations that undertake the development of any of the types of standards described in Section 4.2. Some examples of such standards development organizations (SDOs) are shown in Table 4.1. A more detailed listing specific to geosynthetics will be described later in the chapter. The organizations shown in Table 4.1 are those SDOs that have been designated as the representative organization to the International Organization for Standardization (ISO). As indicated previously there may be more than one organization that undertakes the development of standards. Some are very specific in the areas that they work in, e.g. the International Electrotechnical Committee (IEC). Examples of some other SDOs within the USA are shown in Table 4.2. The

Table 4.2 Other standards development organizations within the USA

Name	Abbreviation
American Society of Testing and Materials International	ASTM
American Association of State Highway and Transportation Officials	AASHTO
National Institute for Standards and Technology	NIST

author of the chapter chose the USA as he is from the USA and therefore these are the SDOs that he is familiar with. Bodies within other countries of the world are indeed viable and just as crucial to the development of standards.

4.3.1 International standards development organizations

On the international level there are three main standards organizations that operate based on national representation to the areas of interest. These are as follows.

1. International Organization for Standardization (ISO).
2. European Committee for Standardization (Comité Européen de Normalisation, CEN).
3. ASTM International.

ISO is truly a worldwide organization, with representation to it coming from the designated national SDOs for each member country. CEN is also made up of representation from the designated national standards organization, but is limited to Europe. ASTM International is made up of individual members from around the world, has international offices in various parts of the world, and also has a wide distribution of its standards worldwide.

4.3.2 Standards development processes

Generally a standards development organization follows one of three processes in the development of its standards.

1. Consensus.
2. One country, one vote.
3. Weighted country vote.

The descriptions of each of these processes follow.

Consensus

In the consensus process, all negative votes and comments received on a balloted item must be considered and responded to before the document can proceed on to finalization. This does not mean that all negative votes and comments cause changes to be made to the proposed standard. First there has to be a technical reason given for the negative vote or comment. The committee that has jurisdiction over the proposed standard must consider the rationale for the negative vote. They may agree with it, which then does cause a change to be made, and reballoting taking place. If the committee does not agree with the negative vote, there must be a rationale given for not agreeing, and a vote taken to find the negative voter non-persuasive. If the rationale for finding the negative non-persuasive is upheld by the committee, the document proceeds on. If not, it is returned for reconsideration to the working group which is developing it. In regard to comments, these are generally associated with affirmative votes, but the voter may have an idea that they wish to have considered without holding up the progress of the document. In this instance the comment is reviewed and the working group developing it may take one of several actions. It may choose to incorporate the idea of the comment into the standard, which then may require a reballot. It may decide that the comment is appropriate but, so as not to hold up progression of the document, they may agree to progress the document and then to undertake a revision immediately on final approval. The third option is they may not agree with the comment and may move the document on. There does not need to be a committee vote taken for any of the actions described related to comments received on a ballot document. In the consensus process, if it is determined that a negative vote was not addressed in the proper manner, the results of the ballot can be nullified, requiring a reballot.

Simple majority

In the simple majority process, the majority result of the ballot on a document rules. In this process, if there is majority affirmative vote, negative votes do not hold up a document from proceeding but still must be addressed. The risk in this process is that a document may proceed on even if a review of the negative votes determines that one or more of them has merit. In this instance the concern then has to be addressed by proposing a revision to the document, and the ballot process started all over again. A flawed document could be issued as a final approved standard before the flaw is corrected.

4.3.3 Voting structures

The various standards organizations have different ways of structuring their votes. There are three primary ways however, that should be explained.

Balanced membership

In a balanced membership structure, the number of members of voting producer organizations may not exceed the number of user and general-interest voting members. Also, each organization has only one official vote when it comes to determining whether there were a sufficient number of ballots returned to consider it a valid ballot. Every member receives a ballot and is encouraged to vote, but only the ballot of the official voting member of an organization is counted in determining the ballot return statistics. Whether an individual is a voting member or not, if they vote negative, their concern must be addressed as in the section above on the consensus process. An example of an organization which uses this structure is ASTM International. In the voting structure of ASTM International (2005b), the sum of all voting members' ballots returned must equal 60% of the total voting

Table 4.3 Voting requirements of ASTM International

Stage	Return requirement	Affirmative vote requirement
Subcommittee	60% of voting membership	Two thirds of combined affirmative and negative votes to be affirmative
Main committee	60% of voting membership	90% of combined affirmative and negative votes to be affirmative
Society review; concurrent with main committee ballot	Not applicable; comments and negatives must be handled as in other stages	

membership of the committee, or subcommittee in order to be considered a valid ballot. Table 4.3 shows a detailed description of voting requirements at the three stages of balloting for an ASTM International standard. Meeting of the voting requirements shown in Table 4.3 does not give immediate approval to a proposed or revised standard. The negatives votes and comments must be handled as previously described, and a review by the ASTM International Standing Committee on Standards be satisfied that this was done before final approval and publication.

One country, one vote

In the one country, one vote structure, membership on a committee is made up of delegations from each individual member country. There are 'participating' and 'observing' member classifications. Only the 'participating member' countries can vote on a document. The vote is issued through the official SDO representative to the overall committee parent organization. Irrespective of how large the representative delegation is, they have only one vote. An example of this structure can be found with the ISO. Table 4.4 details the voting requirements for the various types and stages of ISO documents (International Organization for Standardization, 2001).

Weighted voting

In the CEN voting system, when voting for a European Standard (EN), each country has been assigned a weighted vote. In order for the proposal to be adopted, 71% of the weighted votes cast (not including abstentions) must be in favour. If the proposal is not adopted, the weighted votes cast by European Economic Area countries shall be counted, with approval if 71% of these votes are in favour (European Committee for Standardization, 2002). Examples of these weightings are given in Table 4.5 (European Committee for Standardization, 2002). The

Table 4.4 Approval requirements of ISO[a]

Stage	International standard	Technical specification	Publicly available specification	Technical report
Proposal stage				
Adoption of new work item	SVAT score >15 Simple majority of P-members of the committee Five P-members participating Five experts named			Not applicable
Adoption of proposal for amendment or revision or transformation of deliverable	SVAT score >9 5 P-members participating Simple majority of P-members of the committee agree to the proposal			Not applicable
Preparatory stage				
Acceptance of WD for circulation as CD	Not defined; determined by the committee secretary in conjunction with the committee			
Committee stage				
Acceptance of CD for submission as DIS	Consensus, or Support from two thirds of the P-members voting	Support from two thirds of the P-members voting	Simple majority of P-members of the committee	
Enquiry stage				
Acceptance for submission as FDIS	Two thirds of P-members positive No more than one quarter of the votes negative		Not applicable	
Approval stage				
Agreement to publish	Two-thirds of P-members positive No more than one quarter of the votes negative		Not applicable	

[a]Abbreviations used in Table 4.4 are as follows: SVAT, standards value assessment tool; P-member, participating member; WD, working draft; CD, committee draft; DIS, draft international standard; FDIS, final draft international standard.

weightings are determined using a formula which takes into account a country's population and its gross national product. There are conditions under which a member country may appeal a vote. The complete listing is found in the CEN/

Table 4.5 Weighted votes of CEN

CEN member country	Weighted votes
UK	29
Poland	27
The Netherlands	13
Belgium	12
Sweden	10
Norway	7
Latvia	4
Malta	3

CENELEC Internal Regulations, Part 2, Clause 7 (European Committee for Standardization, 2002).

4.4 Geosynthetic standards

Based on the information in Section 4.3, one can understand that the development of geosynthetic standards is a lengthy process. However, in the end, irrespective of which SDO you work with, the standards that have been and are currently being developed are of the highest quality and most pertinent to the discipline.

4.4.1 Brief history of geosynthetic standardization

The author's earliest participation in the development of geosynthetic standards was in 1977 when the ASTM Committee D13 on Textiles formed a subcommittee to start work on developing test methods for what was then referred to as filter fabrics. Realizing that the subcommittee was not obtaining the needed input from the geotechnical community, a joint subcommittee was formed between Committee D13 and Committee D18 on Soil and Rock. In 1984, after making no progress towards approving any standards owing to the difficulty of handling negative votes through two different committees, with the approval of Committee D13, Committee D18 and ASTM Headquarters, Committee D35 on Geotextiles and Related Products, later changed to Committee D35 on Geosynthetics, was formed. The standards development within Committee D35 is accomplished through a number of subcommittees. A listing of these subcommittees and the scope of their work may be found on the ASTM International web site under Committee D35 (ASTM International, 2006).

ISO Technical Committee 221 on Geosynthetics

Like the ASTM D35 Committee, the ISO geosynthetics activity was originally a subcommittee under Technical Committee 38 on Textiles, Subcommittee 21 on Geotextiles. ISO Technical Committee 221 on Geosynthetics was approved by the

Technical Management Board of ISO in the year 2000. The work within Technical Committee 221 is accomplished through several working groups. These may be found on the ISO web site under ISO/TC 221 (International Organization for Standardization, 2006).

CEN Technical Committee 189 on Geosynthetics

The CEN Technical Committee 189 was formed in 1989, with its first work programme in 1990. Like ISO Technical Committee 221, the work of CEN Technical Committee 189 is also accomplished within several working groups. They may be found on the CEN web site under CEN/TC 189 (European Committee for Standardization, 2006)

4.4.2 Standards around the world

Table 4.6 is a listing of the reference numbers for the geosynthetic standards available at the time of preparation of this text through the three international standards organizations ASTM International, ISO and CEN. Specific information about the standards listed and any additional new standards is available through the respective SDO (ASTM International, 2006; European Committee for Standardization, 2006; International Organization for Standardization, 2006).

4.5 Future trends

Some may say that, since there are many existing standards within the geosynthetics community, what is left to do? The answer falls into two categories.

1 Work on existing standards.
2 Development of new standards.

4.5.1 Work on existing standards

There is a need to review and revise existing standards continually as experience is gained in their use. There is always new equipment being developed to perform existing methods better. As experience is gained, problems may be discovered that may affect the final results that are reported; maybe this is not a problem, but more efficient ways to perform testing may become evident. In the case of standard specifications, it may be found that material requirements need to be tightened up or improved to ensure the expected and desired performance. In all these cases, it will be necessary to review and revise the existing standard formally and to come to a consensus agreement on the appropriate revisions.

Table 4.6 Geosynthetic standards[a]

Property/subject	Standards organization[5] ASTM International Committee D35	ISO Technical Committee 221	CEN Technical Committee 189
Mechanical	D4354-99(2004) *Standard Practice for Sampling of Geosynthetics for Testing*	ISO 13426-1:2003 *Geotextiles and Geotextile-related Products – Strength of Internal Structural Junctions – Part 1: Geocells*	EN 13249:2000 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in the Construction of Roads and Other Trafficked Areas (excluding railways and asphalt inclusion)*
	D4533-04 *Standard Test Method for Trapezoid Tearing Strength of Geotextiles*	ISO 13426-2:2005 *Geotextiles and Geotextile-related Products – Strength of Internal Structural Junctions – Part 2: Geocomposites*	EN 13249:2000/A1:2005 *Geotextiles and Geotextile-related Products – Required Characteristics for Use in the Construction of Roads and Other Trafficked Areas*
	D4595-86(2001) *Standard Test Method for Tensile Properties of Geotextiles by the Wide-width Strip Method*	ISO 12957-1:2005 *Geosynthetics – Determination of Friction Characteristics – Part 1: Direct Shear Test*	EN 13250:2000 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in the Construction of Railways*
	D4632-91(2003) *Standard Test Method for Grab Breaking Load and Elongation of Geotextiles*	ISO 12957-2:2005 *Geosynthetics – Determination of Friction Characteristics – Part 2: Inclined Plane Test*	EN 13250:2000/A1:2005 *Geotextiles and Geotextile-related Products – Required Characteristics for Use in the Construction of Railways*
	D4759-02 *Standard Practice for Determining the Specification Conformance of Geosynthetics*	ISO 12236:1996 *Geotextiles and Geotextile-related Products – Static Puncture Test (CBR test)*	EN 13251:2000 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in Earthworks, Foundations and Retaining Structures*
	D4833-00e1 *Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products*	ISO 10321:1992 *Geotextiles – Tensile Test for Joints/Seams by Wide-width Method*	EN 13251:2000/A1:2005 *Geotextiles and Geotextile-related Products – Required Characteristics for Use in*
	D4884-96(2003) *Standard Test Method for Strength of Sewn or Thermally Bonded Seams of Geotextiles*	ISO 10320:1999 *Geotextiles and Geotextile-related Products – Identification on Site*	
	D5261-92(2003) *Standard Test Method for Measuring Mass per Unit Area of Geotextiles*	ISO 10319:1993 *Geotextiles – Wide-width Tensile Test*	

Table 4.6 Geosynthetic standards[a] *(cont)*

Property/subject	Standards organization[5] ASTM International Committee D35	ISO Technical Committee 221	CEN Technical Committee 189
Mechanical	D5321-02 *Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosyntheticand Geosynthetic Friction by the Direct Shear Method* D5818-95(2000) *Standard Practice for Obtaining Samples of Geosynthetics from a Test Section for Assessment of Installation Damage* D6241-04 *Standard Test Method for the Static Puncture Strength of Geotextiles and Geotextile-related Products Using a 50-mm Probe* D6244-98(2004) *Standard Test Method for Vertical Compression of Geocomposite Pavement Panel Drains* D6364-99(2004) *Standard Test Method for Determining the Short-term Compression Behavior of Geosynthetics* D6637-01 *Standard Test Method for Determining Tensile Properties of Geogrids by the Single or Multi-rib Tensile Method* D6638-01 *Standard Test Method for*	ISO 9864:2005 *Geosynthetics – Test Method for the Determination of Mass Per Unit Area of Geotextiles and Geotextile-related Products* ISO 9863-1:2005 *Geosynthetics – Determination of Thickness at Specified Pressures – Part 1: Single Layers* ISO 9863-2:1996 *Geotextiles and Geotextile-related Products – Determination of Thickness at Specified Pressures – Part 2: Procedure for Determination of Thickness of Single Layers of Multilayer Products* ISO 9862:2005 *Geosynthetics – Sampling and Preparation of Test Specimens*	*Earthworks, Foundations and Retaining Structures* EN 13256:2000 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in the Construction of Tunnels and Underground Structures* EN 13256:2000/A1:2005 *Geotextiles and Geotextile-related Products – Required Characteristics for Use in the Construction of Tunnels and Underground Structures* EN 13256:2000/AC:2003 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in the Construction of Tunnels and Underground Structures* EN 13257:2000 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in Solid Waste Disposals* EN 13257:2000/A1:2005 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in Solid Waste Disposals* EN 13257:2000/AC:2003 *Geotextiles and Geotextile-related Products –*

Determining Connection Strength Between Geosynthetic Reinforcement and Segmental Concrete Units (Modular Concrete Blocks)
D6706-01 *Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil*
D6916-03 *Standard Test Method for Determining the Shear Strength Between Segmental Concrete Units (Modular Concrete Blocks)*
D7005-03 *Standard Test Method for Determining the Bond Strength (Ply Adhesion) of Geocomposites*
D7179-05 *Standard Test Method for Determining Geonet Breaking Force*

Characteristics Required for Use in Solid Waste Disposals
EN 13265:2000 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in Liquid Waste Containment Projects*
EN 13265:2000/A1:2005 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in Liquid Waste Containment Projects*
EN 13265:2000/AC:2003 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in Liquid Waste Containment Projects*
EN 13738:2004 *Geotextiles and Geotextile-related Products – Determination of Pullout Resistance in Soil*
EN 14574:2004 *Geosynthetics – Determination of the Pyramid Puncture Resistance of Supported Geosynthetics*
EN 918:1995 *Geotextiles and Geotextile-related Products – Dynamic Perforation Test (Cone Drop Test)*
EN ISO 10319:1996 *Geotextiles – Wide-width Tensile Test* (ISO 10319:1993)
EN ISO 10321:1996 *Geotextiles – Tensile Test for Joints/Seams by Wide-width Method* (ISO 10321:1992)

Table 4.6 Geosynthetic standards[a] (*cont.*)

	Standards organization[5]		
Property/subject	ASTM International Committee D35	ISO Technical Committee 221	CEN Technical Committee 189
Mechanical			EN ISO 12236:1996 *Geotextiles and Geotextile-related Products – Static Puncture Test (CBR-Test)* (ISO 12236:1996) EN ISO 12957-1:2005 *Geosynthetics – Determination of Friction Characteristics – Part 1: Direct Shear Test* (ISO 12957-1:2005) EN ISO 12957-2:2005 *Geosynthetics – Determination of Friction Characteristics – Part 2: Inclined Plane Test* (ISO 12957-2:2005) EN ISO 13426-1:2003 *Geotextiles and Geotextile-related Products – Strength of Internal Structural Junctions – Part 1: Geocells* (ISO 13426-1:2003) EN ISO 13426-2:2005 *Geotextiles and Geotextile-related Products – Strength of Internal Structural Junctions – Part 2: Geocomposites* (ISO 13426-2:2005) EN ISO 13427:1998 *Geotextiles and Geotextile-related Products – Abrasion Damage Simulation (Sliding Block Test)* (ISO 13427:1998)

			EN ISO 13428:2005 *Geosynthetics – Determination of the Protection Efficiency of a Geosynthetic Against Impact Damage (*ISO 13428:2005) EN ISO 9862:2005 *Geosynthetics – Sampling and Preparation of Test Specimens* (ISO 9862:2005) EN ISO 9863-1:2005 *Geosynthetics – Determination of Thickness at Specified Pressures – Part 1: Single Layers* (ISO 9863-1:2005) EN ISO 9863-2:1996 *Geotextiles and Geotextile-related Products – Determination of Thickness at specified Pressures – Part 2: Procedure for Determination of Thickness of Single Layers of Multilayer Products* (ISO 9863-2:1996) EN ISO 9864:2005 *Geosynthetics – Test Method for the Determination of Mass Per Unit Area of Geotextiles and Geotextile-related Products* (ISO 9864:2005)
Hydraulic	D4491-99a(2004) *Standard Test Methods for Water Permeability of Geotextiles by Permittivity* *D4716-04 Test Method for Determining the (In-plane) Flow Rate per Unit Width and Hydraulic Transmissivity of a Geosynthetic Using a Constant Head*	ISO 12958:1999 *Geotextiles and Geotextile-related Products – Determination of Water Flow Capacity in their Plane* ISO 12956:1999 *Geotextiles and Geotextile-related Products – Determination of the Characteristic Opening Size*	EN 12447:2001 *Geotextiles and Geotextile-related Products – Screening Test Method for Determining the Resistance to Hydrolysis in Water* EN 13252:2000 *Geotextiles and Geotextile-related Products –*

Table 4.6 Geosynthetic standards[a] (*cont.*)

	Standards organization[5]		
Property/subject	ASTM International Committee D35	ISO Technical Committee 221	CEN Technical Committee 189
Hydraulic	D4751-04 *Standard Test Method for Determining Apparent Opening Size of a Geotextile* D5101-01 *Standard Test Method for Measuring the Soil–Geotextile System Clogging Potential by the Gradient Ratio* D5141-96(2004) *Standard Test Method for Determining Filtering Efficiency and Flow Rate of a Geotextile for Silt Fence Application Using Site-specific Soil* D5199-01 *Standard Test Method for Measuring the Nominal Thickness of Geosynthetics* D5493-93(2003) *Standard Test Method for Permittivity of Geotextiles Under Load* D5567-94(2001) *Standard Test Method for Hydraulic Conductivity Ratio (HCR) Testing of Soil/Geotextile Systems* D6088-97(2002) *Standard Practice for Installation of Geocomposite Pavement Drains* D6140-00 *Standard Test Method to Determine Asphalt Retention of*	ISO 11058:1999 *Geotextiles and Geotextile-related Products – Determination of Water Permeability Characteristics Normal to the Plane, Without Load*	*Characteristics Required for Use in Drainage Systems* EN 13252:2000/A1:2005 *Geotextiles and Geotextile-related Products – Required Characteristics for Use in Drainage Systems* EN 13254:2000 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in the Construction of Reservoirs and Dams* EN 13254:2000/A1:2005 *Geotextiles and Geotextile-related Products – Required characteristics for Use in the Construction of Reservoirs and Dams* EN 13254:2000/AC:2003 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in the Construction of Reservoirs and Dams* EN 13255:2000 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in the Construction of Canals* EN 13255:2000/A1:2005 *Geotextiles and Geotextile-related Products –*

Paving Fabrics Used in Asphalt Paving for Full-width Applications
D6523-00 *Standard Guide for Evaluation and Selection of Alternative Daily Covers (ADCs) for Sanitary Landfills*
D6574-00 *Test Method for Determining the (In-Plane) Hydraulic Transmissivity of a Geosynthetic by Radial Flow*
D6707-01 *Standard Specification for Circular-knit Geotextile for Use in Subsurface Drainage Applications*
D6767-02 *Standard Test Method for Pore Size Characteristics of Geotextiles by Capillary Flow Test*
D6817-04 *Standard Specification for Rigid Cellular Polystyrene Geofoam*
D6826-05 *Standard Specification for Sprayed Slurries, Foams and Indigenous Materials Used As Alternative Daily Cover for Municipal Solid Waste Landfills*
D6917-03 *Standard Guide for Selection of Test Methods for Prefabricated Vertical Drains (PVD)*
D6918-03 *Standard Test Method for Testing Vertical Strip Drains in the Crimped Condition*
D7001-05 *Standard Specification*

Required Characteristics for Use in the Construction of Canals
EN 13255:2000/AC:2003 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in the Construction of Canals*
EN 13562:2000 *Geotextiles and Geotextile-related Products – Determination of Resistance to Penetration by Water (Hydrostatic Pressure Test)*
EN ISO 11058:1999 *Geotextiles and Geotextile-related Products – Determination of Water Permeability Characteristics Normal to the Plane, Without Load* (ISO 11058:1999)
EN ISO 12956:1999 *Geotextiles and Geotextile-related Products – Determination of the Characteristic Opening Size* (ISO 12956:1999)
EN ISO 12958:1999 *Geotextiles and Geotextile-related Products – Determination of Water Flow Capacity in Their Plane* (ISO 12958:1999)

Table 4.6 Geosynthetic standards[a] (*cont.*)

	Standards organization[5]		
Property/subject	ASTM International Committee D35	ISO Technical Committee 221	CEN Technical Committee 189
Hydraulic	f*or Geocomposites for Pavement Edge Drains and Other High-flow Applications* D7008-03 *Standard Specification for Geosynthetic Alternate Daily Covers* D7180-05 *Standard Guide for Use of Expanded Polystyrene (EPS) Geofoam in Geotechnical Projects*		
Durability	D1987-95(2002) *Standard Test Method for Biological Clogging of Geotextile or Soil/Geotextile Filters* D4355-05 *Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus* D4594-96(2003) *Standard Test Method for Effects of Temperature on Stability of Geotextiles* *D4873-02 Standard Guide for Identification, Storage, and Handling of Geosynthetic Rolls and Samples* D4886-88(2002) *Standard Test Method for Abrasion Resistance of*	ISO 13438:2004 *Geotextiles and Geotextile-related Products – Screening Test Method for Determining the Resistance to Oxidation* ISO 13437:1998 *Geotextiles and Geotextile-related Products – Method for Installing and Extracting Samples in Soil, and Testing Specimens in Laboratory* ISO/TR 13434:1998 Geotextiles and Geotextile-related Products – Guidelines on Durability ISO 13431:1999 *Geotextiles and Geotextile-related Products – Determination of Tensile Creep and Creep Rupture Behaviour*	CEN/TR 15019 *Geotextiles and Geotextile-related Products – On-site Quality Control:2005* CR ISO 13434:1998 *Guidelines on Durability of Geotextiles and Geotextile-related Products* EN 12224:2000 *Geotextiles and Geotextile-related Products – Determination of the Resistance to Weathering* EN 12225:2000 *Geotextiles and Geotextile-related Products – Method for Determining the Microbiological Resistance by a Soil Burial Test* EN 12226:2000 *Geotextiles and Geotextile-related Products –*

Geotextiles (Sand Paper/Sliding Block Method)

D5262-04 Standard Test Method for Evaluating the Unconfined Tension Creep Behavior of Geosynthetics

D5322-98(2003) Standard Practice for Immersion Procedures for Evaluating the Chemical Resistance of Geosynthetics to Liquids

D5397-99e1 Standard Test Method for Evaluation of Stress Crack Resistance of Polyolefin Geomembranes Using Notched Constant Tensile Load Test

D5496-98(2003) Standard Practice for In Field Immersion Testing of Geosynthetics

D5596-03 Standard Test Method for Microscopic Evaluation of the Dispersion of Carbon Black in Polyolefin Geosynthetics

D5721-95(2002) Standard Practice for Air–Oven Aging of Polyolefin Geomembranes

D5747-95a(2002) Standard Practice for Tests to Evaluate the Chemical Resistance of Geomembranes to Liquids

D5819-05 Standard Guide for Selecting Test Methods for Experimental Evaluation of Geosynthetic Durability

ISO 13428:2005 *Geosynthetics – Determination of the Protection Efficiency of a Geosynthetic Against Impact Damage*

ISO 13427:1998 *Geotextiles and Geotextile-related Products – Abrasion Damage Simulation (Sliding Block Test)*

ISO/TR 12960:1998 *Geotextiles and Geotextile-related Products – Screening Test Method for Determining the Resistance to Liquids*

ISO/TR 10722-1:1998 *Geotextiles and Geotextile-related Products – Procedure for Simulating Damage During Installation – Part 1: Installation in Granular Materials*

General Tests for Evaluation Following Durability Testing

EN 14030:2001 *Geotextiles and Geotextile-related Products – Screening Test Method for Determining the Resistance to Acid and Alkaline Liquids* (ISO/TR 12960:1998, modified)

EN 14030:2001/A1:2003 *Geotextiles and Geotextile-related Products – Screening Test Method for Determining the Resistance to Acid and Alkaline Liquid*s (ISO/TR 12960:1998, modified)

EN 14414:2004 *Geosynthetics – Screening Test Method for Determining Chemical Resistance for Landfill Applications*

EN 1897:2001 *Geotextiles and Geotextile-related Products – Determination of the Compressive Creep Properties*

EN ISO 10320:1999 *Geotextiles and Geotextile-related Products – Identification on Site* (ISO 10320:1999)

EN ISO 13431:1999 *Geotextiles and Geotextile-related Products – Determination of Tensile Creep and Creep Rupture Behaviour* (ISO 13431:1999)

EN ISO 13437:1998 *Geotextiles and*

Table 4.6 Geosynthetic standards[a] (*cont.*)

	Standards organization[5]		
Property/subject	ASTM International Committee D35	ISO Technical Committee 221	CEN Technical Committee 189
Durability	D5885-04 *Standard Test Method for Oxidative Induction Time of Polyolefin Geosynthetics by High-Pressure Differential Scanning Calorimetry* D5970-96(2002) *Standard Practice for Deterioration of Geotextiles from Outdoor Exposure* D6213-97(2003) *Standard Practice for Tests to Evaluate the Chemical Resistance of Geogrids to Liquids* D6388-99 *Standard Practice for Tests to Evaluate the Chemical Resistance of Geonets to Liquids* D6389-99 *Standard Practice for Tests to Evaluate the Chemical Resistance of Geotextiles to Liquids* *D6992-03 Standard Test Method for Accelerated Tensile Creep and Creep-rupture of Geosynthetic Materials Based on Time–Temperature Superposition Using the Stepped Isothermal Method*		*Geotextile-related Products – Method for Installing and Extracting Samples in Soil, and Testing Specimens in Laboratory* (ISO 13437:1998 EN ISO 13438:2004 *Geotextiles and Geotextile-related Products – Screening Test Method for Determining the Resistance to Oxidation* (ISO 13438:2004) ENV ISO 10722-1:1998 *Geotextiles and Geotextile-related Products – Procedure for Simulating Damage During Installation – Part 1: Installation in Granular Materials* (ISO 10722-1:1998)

Geosynthetic liners; also known as clay geosynthetic barriers	D5887-04 *Standard Test Method for Measurement of Index Flux Through Saturated Geosynthetic Clay Liner Specimens Using a Flexible Wall Permeameter* D5888-95(2002)e1 *Standard Guide for Storage and Handling of Geosynthetic Clay Liners* D5889-97(2003) *Standard Practice for Quality Control of Geosynthetic Clay Liners* D5890-02 *Standard Test Method for Swell Index of Clay Mineral Component of Geosynthetic Clay Liners* D5891-02 *Standard Test Method for Fluid Loss of Clay Component of Geosynthetic Clay Liners* D5993-99(2004) *Standard Test Method for Measuring Mass Per Unit of Geosynthetic Clay Liners* D6072-96(2002) *Standard Guide for Obtaining Samples of Geosynthetic Clay Liners* D6102-04 *Standard Guide for Installation of Geosynthetic Clay Liners* D6141-97(2004) *Standard Guide for Screening Clay Portion of Geosynthetic Clay Liner (GCL) for Chemical Compatibility to Liquids* D6243-98 *Standard Test Method for*		EN 13361:2004 *Geosynthetic Barriers – Characteristics Required for Use in the Construction of Reservoirs and Dams* *EN 13362:2005 Geosynthetic Barriers – Characteristics Required for Use in the Construction of Canals* EN 13491:2004 *Geosynthetic Barriers – Characteristics Required for Use as a Fluid Barrier in the Construction of Tunnels and Underground Structures* EN 13492:2004 *Geosynthetic Barriers – Characteristics Required for Use in the Construction of Liquid Waste Disposal Sites, Transfer Stations or Secondary Containment* EN 13493:2005 *Geosynthetic Barriers – Characteristics Required for Use in the Construction of Solid Waste Storage and Disposal Sites* EN 13719:2002 *Geotextiles and Geotextile-related Products – Determination of the Long Term Protection Efficiency of Geotextiles in Contact with Geosynthetic Barriers* EN 13719:2002/AC:2005 *Geotextiles and Geotextile-related Products – Determination of the Long Term Protection Efficiency of Geotextiles*

Table 4.6 Geosynthetic standards[a] (*cont.*)

Property/subject	Standards organization[5] ASTM International Committee D35	ISO Technical Committee 221	CEN Technical Committee 189
Geosynthetic liners; also known as clay geosynthetic barriers	*Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liner by the Direct Shear Method* D6495-02 *Standard Guide for Acceptance Testing Requirements for Geosynthetic Clay Liners* D6496-04a *Standard Test Method for Determining Average Bonding Peel Strength Between the Top and Bottom Layers of Needle-Punched Geosynthetic Clay Liners* D6766-02 *Standard Test Method for Evaluation of Hydraulic Properties of Geosynthetic Clay Liners Permeated with Potentially Incompatible Liquids* D6768-04 *Standard Test Method for Tensile Strength of Geosynthetic Clay Liners*		*in Contact with Geosynthetic Barriers* EN 14196:2003 *Geosynthetics – Test Methods for Measuring Mass Per Unit Area of Clay Geosynthetic Barriers* EN 14415:2004 *Geosynthetic Barriers – Test Method for Determining the Resistance to Leaching* EN 14575:2005 *Geosynthetic Barriers – Screening Test Method for Determining the Resistance to Oxidation* EN 14576:2005 *Geosynthetics – Test Method for Determining the Resistance of Polymeric Geosynthetic Barriers to Environmental Stress Cracking*

Geomembranes

D4437-99 *Standard Practice for Determining the Integrity of Field Seams Used in Joining Flexible Polymeric Sheet Geomembranes*

D4545-86(1999) *Standard Practice for Determining the Integrity of Factory Seams Used in Joining Manufactured Flexible Sheet geomembranes*

D4885-01 *Standard Test Method for Determining Performance Strength of Geomembranes by the Wide Strip Tensile Method*

D5323-92(1999) *Standard Practice for Determination of 2% Secant Modulus for Polyethylene Geomembranes*

D5494-93(1999)e1 *Standard Test Method for the Determination of Pyramid Puncture Resistance of Unprotected and Protected Geomembranes*

D5514-94(2001) *Standard Test Method for Large Scale Hydrostatic Puncture Testing of Geosynthetics*

D5617-04 *Standard Test Method for Multi-axial Tension Test for Geosynthetics*

D5641-94(2001)e1 *Standard Practice for Geomembrane Seam Evaluation by Vacuum Chamber*

D5820-95(2001)e1 *Standard*

Table 4.6 Geosynthetic standards[a] (*cont.*)

Property/subject	Standards organization[5] ASTM International Committee D35	ISO Technical Committee 221	CEN Technical Committee 189
Geomembranes	*Practice for Pressurized Air Channel Evaluation of Dual Seamed Geomembranes* D5884-04a *Standard Test Method for Determining Tearing Strength of Internally Reinforced Geomembranes* D5886-95(2001) *Standard Guide for Selection of Test Methods to Determine Rate of Fluid Permeation Through Geomembranes for Specific Applications* D5994-98(2003) *Standard Test Method for Measuring Core Thickness of Textured Geomembrane* D6214-98(2003) *Standard Test Method for Determining the Integrity of Field Seams Used in Joining Geomembranes by Chemical Fusion Methods* D6365-99 *Standard Practice for the Nondestructive Testing of Geomembrane Seams using the Spark Test* D6392-99 *Standard Test Method for Determining the Integrity of*		

Nonreinforced Geomembrane Seams Produced Using Thermo-fusion Methods

D6434-04 *Standard Guide for the Selection of Test Methods for Flexible Polypropylene (fPP) Geomembranes*

D6455-99 *Standard Guide for the Selection of Test Methods for Prefabricated Bituminous Geomembranes (PBGM)*

D6497-02 *Standard Guide for Mechanical Attachment of Geomembrane to Penetrations or Structures*

D6636-01 *Standard Test Method for Determination of Ply Adhesion Strength of Reinforced Geomembranes*

D6693-04 *Standard Test Method for Determining Tensile Properties of Nonreinforced Polyethylene and Nonreinforced Flexible Polypropylene Geomembranes*

D6747-04 *Standard Guide for Selection of Techniques for Electrical Detection of Potential Leak Paths in Geomembrane*

D7002-03 *Standard Practice for Leak Location on Exposed Geomembranes Using the Water Puddle System*

Table 4.6 Geosynthetic standards[a] (*cont.*)

	Standards organization[5]		
Property/subject	ASTM International Committee D35	ISO Technical Committee 221	CEN Technical Committee 189
Geomembranes	D7003-03 *Standard Test Method for Strip Tensile Properties of Reinforced Geomembranes*		
	D7004-03 *Standard Test Method for Grab Tensile Properties of Reinforced Geomembranes*		
	D7006-03 *Standard Practice for Ultrasonic Testing of Geomembranes*		
	D7007-03 *Standard Practices for Electrical Methods for Locating Leaks in Geomembranes Covered with Water or Earth Materials*		
	D7056-04 *Standard Test Method for Determining the Tensile Shear Strength of Pre-fabricated Bituminous Geomembrane Seams*		
	D7106-05 *Standard Guide for Selection of Test Methods for Ethylene Propylene Diene Terpolymer (EPDM) Geomembranes*		
	D7177-05 *Standard Specification for Air Channel Evaluation of Polyvinyl Chloride (PVC) Dual Track Seamed Geomembranes*		

Erosion control	D6454-99 *Standard Test Method for Determining the Short-term Compression Behavior of Turf Reinforcement Mats (TRMs)* D6524-00 *Standard Test Method for Measuring the Resiliency of Turf Reinforcement Mats (TRMs)* D6525-00 *Standard Test Method for Measuring Nominal Thickness of Permanent Rolled Erosion Control Products* D6566-00 *Standard Test Method for Measuring Mass per Unit Area of Turf Reinforcement Mats* D6567-00 *Standard Test Method for Measuring the Light Penetration of a Turf Reinforcement Mat (TRM)* D6575-00 *Standard Test Method for Determining Stiffness of Geosynthetics Used as Turf Reinforcement Mats (TRM's)* D6818-02 *Standard Test Method for Ultimate Tensile Properties of Turf Reinforcement Mats*		EN 13253:2000 *Geotextiles and Geotextile-related Products – Characteristics Required for Use in Erosion Control Works (Coastal Protection, Bank Revetments)* EN 13253:2000/A1:2005 *Geotextiles and Geotextile-related Products – Required Characteristics for Use in External Erosion Control Systems*
Terminology	D4439-04 Standard Terminology for Geosynthetics	ISO 10318:2005 Geosynthetics – Terms and Definitions	EN ISO 10318:2005 Geosynthetics – Terms and Definitions (ISO 10318:2005)

[a]As you move horizontally across the columns of Table 4.6, standards may not necessarily be the corresponding standard between each SDO. You may have to move vertically to find a corresponding standard.

4.5.2 Development of new standards

As new materials and new products are developed, it will become necessary to go through the same sort of test method development as has been done for the existing geosynthetics. It may become evident that even for the existing materials and products, because of a new use, or a problem occurring with their use, that a new better method of evaluation is needed.

4.5.3 Avoiding duplication of efforts

Looking down through Table 4.6 it becomes very evident that, just between the three SDOs listed there, it appears that each SDO has standards that will provide the same basic information. There may be minor or insignificant differences between the methods that will not have an effect on the final information reported for each, or there may be some major differences which will produce some values or information to be reported by each. The geosynthetics industry has truly become a global industry with manufacturers of products in just about all regions of the world. Most of these companies are not confined to doing business in their regions but have and are providing materials worldwide. If they are required to test or provide materials meeting the requirements of basically the same methods but to follow the standards particular to that region, it is very evident that the economic effect on the manufacturer can be great.

4.5.4 Efforts to avoid duplication

There have been efforts on the parts of the three committees shown in Table 4.6 to avoid duplication but, for reasons beyond the scope of this chapter, it has been very difficult to implement plans that had been devised to accomplish this. That said, the committees still are working among themselves to avoid duplicative efforts. Economics is not the only reason that concerted efforts need to be made to avoid duplicative work. It is time wasted if there is already a readily acceptable standard within another SDO for a group of people to spend time 'reinventing the wheel,' when they could be moving on to other areas that need work and where there may not already be a standard in existence.

4.6 Conclusions

It was stated at the beginning of this chapter that there is a definite need for standardization in the field of geosynthetics. It provides the end user with the ability to be comfortable that, at least in the testing area, the results of tests performed by one laboratory can be reproduced within statistical limits by another laboratory, provided that the same testing protocol is followed. For manufacturers, they then do not have to incur the expense of having the same products tested by however many different procedures for the same test there might be. From standardized specifications, the manufacturers would then not have to produce many different products to meet the many different requirements that appear in specifications for the same end use. Standardization also provides the opportunity to focus on innovation and development of new products to meet the needs of the engineering community.

4.7 References

ASTM International (2005a), *Form and Style for ASTM Standards Manual*, ASTM International, West Conshohocken, Pennsylvania.

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Part II

Applications

5 Multifunctional uses of geosynthetics in civil engineering

C. J. F. P. JONES
Newcastle University, UK

5.1 Introduction

Most geosynthetic materials play a passive role, e.g., geosynthetic barriers stop the passage of liquids; geosynthetic reinforcement provides tensile resistance, but only after an initial strain has occurred; and geo-drains provide a passage for water but do not cause the water to flow. New applications for geosynthetics have been developed by the introduction of new multifunctional geosynthetic materials. These can be identified in three forms, as *composite geosynthetics* which provide two or more conventional functions in one material, *smart geosynthetics* which provide critical management information and *active geosynthetics* that create a change in their environment rather than simply acting in a passive role.

Composite geosynthetics associated with filtration and drainage have been available for many years; recent developments relate to combining reinforcing functions with drainage.

Smart geosynthetics provide information on the development on ground movements, which can influence the performance of geotechnical structures such as road and railway embankments. The identification of these movements is important in maintaining serviceability and safety, particularly where the embankment supports high-speed traffic.

Active geosynthetics represent a new generation of geosynthetic materials, which have a wide range of new applications. The key innovation with these materials is that they are electrically conductive and have the ability to initiate electrokinetic processes as well as to retain the established geosynthetic functions. The electrokinetic phenomena used are electro-osmosis and electrophoresis (electro-osmosis causes water movement through low-permeability materials and electrophoresis relates to the movement of particles in materials with very high water contents).

Designing with active geosynthetics involves an understanding of electrokinetic phenomena and consideration of soil properties not usually considered in design.

This chapter identifies these and illustrates their relevance as well as providing information on case histories of the use of electrically conductive geosynthetics.

5.2 Composite geosynthetics

5.2.1 Combined filtration and drainage

Vertical sand drains were proposed as a means of deep soil improvement in 1925 by the American engineer Moran. The first practical sand drain installation was constructed in California. Later the Swedish engineer Kjellman introduced the first prototype prefabricated vertical drain made from cardboard, which were called wick drains. The main features of the prefabricated vertical drain are the combination of an outside filter surrounding an inner drainage core, which can be installed rapidly in the field to reduce the drainage path of poorly draining soils. In 1971, the Kjellman wick drain was improved by the introduction of a grooved plastic polyethylene core as a replacement for the cardboard centre. The use of prefabricated vertical drains has increased significantly and is now established construction practice. Many types have been developed, rapid and simple installation procedures have been introduced and installation depths of about 60 m at a rate of up to 1 m/s are possible.

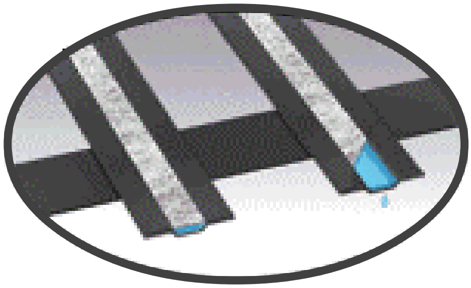
5.2.2 Combined reinforcement and drainage

The design of reinforced slopes is concerned with the provision of an adequate factor of safety on strength and the control of settlements to acceptable limits. Conventional design and construction methods have used granular materials because of their high shear strength and good drainage properties. Research and long-term case histories have indicated that cohesive soils can be used in the construction of reinforced slopes if an adequate drainage system is provided.

When low-permeability fills are loaded, excess pore water pressures can be generated. This can result in a reduction in the available shear strength of the cohesive fill and also a reduction in the soil–reinforcement bond, requiring more reinforcement to provide an adequate bond length. The dissipation of excess pore water pressures results in consolidation and settlement of the reinforced structure, which can result in unacceptable face deflections.

The magnitude of excess pore water pressure present in a slope is a function both of the applied load and of the ability of the drainage system to dissipate the excess pore water pressure. At the base of a slope, with no drainage, large excess pore water pressures can develop. If drainage is provided and complete dissipation of excess pore water pressure occurs before construction of the next layer, the excess pore water pressure in the completed structure would be only a fraction of that otherwise present.

The ideal reinforcing material for cohesive soils requires the drainage charact-



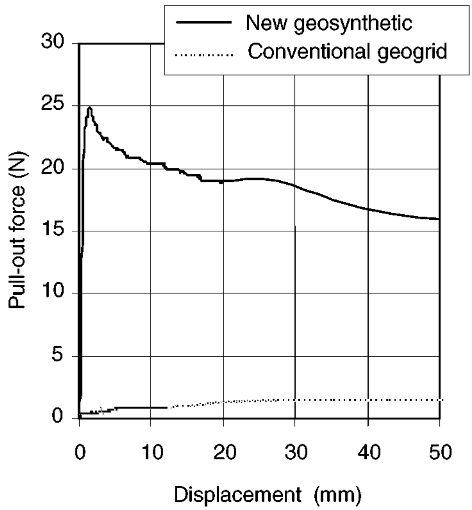
5.1 Integral drainage and reinforcement.

eristics of a non-woven geotextile and the strength of stiffer or stronger reinforcing geosynthetics. Alternatively, it is possible to combine existing materials (e.g. using a non-woven drainage geotextile together with geogrid reinforcement).

Heshmati (1993) studied the effects of combining a drainage material with grid reinforcement in clay soil. He concluded that the drainage and the reinforcement function were equally important in producing a stable and efficient structure. An important observation was that the method used to combine the drainage and reinforcing functions is critical. Simply placing a geotextile drain in conjunction with geogrid reinforcement can result in a reduction in strength as the presence of the drainage layer can lubricate the surface of the reinforcement. An essential requirement is that the combined functions of reinforcement and drainage have to be made integral.

An innovative geosynthetic material has been developed, which conforms with Heshmati's findings relating to a reinforcing material, also providing drainage. The multifunctional geosynthetic material consists of high-tenacity polyester encased in a polyethylene sheath. The sheath both protects the load-carrying elements and maintains the shape of the product which is profiled to provide a drainage channel on one side. The profiled strap has a thermally bonded non-woven geotextile strip bonded on the shoulders of the drainage channel action as a filter. The geotextile allows excess pore water pressure to dissipate while retaining the cohesive soil (Fig. 5.1). Confirmation of the performance of the combined geogrid reinforcement and drainage material has been provided by Kempton *et al.* (2000) who identified the following.

1 The effectiveness in dissipating excess pore water pressures under various confining stresses.
2 The pull-out resistance of the multifunctional material compared with a conventional geogrid of similar construction but with no drainage component.
3 The horizontal flow characteristics of the material under various hydraulic gradients and confining pressures.
4 Suitable parameters for use in design for constructing steep slopes using cohesive fills.



5.2 Pull-out results for a new geosynthetic and conventional geogrid with no drainage component after dissipation of excess pore water pressure for 12 h (after Kempton *et al.,* 2000).

The test results show that the pore water pressure reduces to 20% of the applied pressure in a 36–42 h period at confining pressures of both 50 and 100 kPa. No noticeable difference was observed in pore water pressure values measured above and below the test specimen even though the drainage channel was only on one side of the combined reinforcement and drainage material.

Pull-out testing on the combined material and on conventional geogrid with no drainage component was conducted after partial and full dissipation of excess pore water pressure (Fig. 5.2). The improvement in pull-out resistance is explained by the rapid dissipation of the excess pore water pressure in the immediate vicinity of the composite material, thus allowing early development of bond between the reinforcement and the soil. Full dissipation of excess pore water pressure is assumed when the pore water pressure reaches 10% of the applied overburden pressure in the immediate vicinity of the composite material.

5.3 Smart geosynthetics

The collapse of subsurface voids is a major problem in some counties in Europe, North America and parts of Africa and Asia. These voids are the result of geological conditions associated with the solution of soluble rocks (karst) or the result of underground mining. A feature of the development of surface voids caused by the collapse of old mine workings or evaporite solution of underlying strata is that the time of occurrence and location of any void are unknown. Prediction of void formation can only be made based upon historical evidence or probability theory. From an engineering perspective, the development of randomly occurring voids is a major potential hazard which poses particular structural problems to transport systems.

The logical approach to the development of surface voids is to avoid the problem by relocating the particular structure to an area not affected. This may be possible with some building structures but it is usually impossible to use this strategy with basic infrastructure such as transportation systems. In these cases, design precautions are required. The basic design approach is to consider two limit states, covering ultimate and serviceability conditions. The *ultimate limit state* considers collapse conditions whilst the *serviceability limit state* governs deformation modes of failure, which do not lead to collapse but which render the structure or any system supported by the structure unserviceable.

For small voids it is possible, using engineering techniques, to ensure serviceability although the serviceability limits are sensitive to the engineering problem. High-speed rail systems are the most sensitive to differential settlements and voids of 2–4 m in diameter may be the limiting size for design.

In the case of voids in excess of 10–20 m it may not be possible to design against the ultimate limit case even for highways. With very large voids, collapse of the structure is inevitable and any structural precautions are restricted to providing warning of the collapse in order that loss of life may be avoided and if necessary to permit the mobilization of emergency measures. The design objective in this case is to provide a safe period of 24 h from loss of serviceability to total collapse. In the case of small voids (less than 3–10 m), the design objective is to provide long-term serviceability.

5.3.1 Design solutions

The use of basal reinforcement to prevent collapse of fill following the formation of a void or to support embankments over piles are accepted foundation engineering techniques, as described in BS 8006 (British Standards Institution, 1995). The development of basal reinforcement is an example of the technical and economic benefits which have been provided by the introduction of high-strength polymeric materials for use in reinforced soil (Jones, 1996). The technique was used successfully in the reconstruction of the East Coast Main Railway line between London and Edinburgh in 2002–2003 and on the new rail track at Stansted Airport, UK.

A weakness of the use of basal reinforcement is that it can mask the movement of the subsoil, and monitoring is required. To overcome the difficulties inherent in monitoring potential settlement, a smart geosynthetic material has been developed named 'Geodetect'. Geodetect is a system that combines the reinforcing function with a monitoring system based upon optical technology which can be used over extensive areas.

5.3.2 Conventional monitoring systems

Conventional monitoring systems can be classed in three groups.

1 Usual sensors.
2 Electrical warning system.
3 Ground-penetrating radar.

Usual sensors

Two kinds of usual sensor can be used: sensors fixed to the geosynthetic material (to measure the strain) and sensors positioned in the soil (to measure the settlement). Various kinds of sensor can be used including strain gauges, rod extensometers and inclinometers. These sensors can only be used for discrete measurement during a full-scale test. They cannot be used in a warning system because their lifetime in soil is short and they are difficult to fix to geosynthetic materials. Sensors in soil cannot be used in a general warning system. The use of settlement gauges require great care during installation and can only detect large localized sinkholes.

Electrical warning system

The electrical warning system consists of two non-woven geotextiles fitted together with electric wires at the inner side to form a detection layer. Deformation below the warning layer is indicated by an increase in electrical resistance. The method is effective for the detection of collapsing subsurface cavities and has been used in Germany (Ast and Haberland, 2002; Leitner *et al.*, 2002). However, the electrical aspect of the method may be a disadvantage for railway line application owing to electrical interference with signal systems.

Ground-penetrating radar

Ground-penetrating radar is a non-invasive electromagnetic geophysical technique for subsurface exploration, characterization and monitoring. It is widely used in locating lost utilities, environmental site characterization and monitoring, unexploded ordnance and land mine detection, groundwater, pavement and infrastructure characterization, mining, voids, and cave and tunnel detection. The application of the technique to sinkhole surveys has not proved effective in the case of railways. The method requires daily monitoring and can be expensive.

5.3.3 Geodetect system

The Geodetect system is formed as a composite material from a geosynthetic reinforcement material containing optical fibres. The optical fibres are inserted into the geotextile during manufacture in a flexible sheath. The reinforcement geosynthetic is formed as a non-woven geotextile containing polyester reinforcement strands. The strands are needle punched to the non-woven geotextile in the production (machine) direction.

The optical fibres use the fibre Bragg grating (FBG) technique. FBGs are diffracting elements printed in the photosensitive core of a single-mode optical fibre. The grating reflects a spectral peak based on the grating spacing; thus changes in the length of the fibre due to tension or compression alter the grating spacing and the wavelength of light that is reflected back. Quantitative strain measurements can be made by measuring the centre wavelength of the reflected spectral peak. By using different wavelengths on which the mirrors are reflecting, signals of various FBG sensors can be identified. The wavelengths and wavelength shifts of these so-called mirrors can be measured with a fibre optic unit, allowing them to undergo demultiplexing in the wavelength domain. In this way, the space-distributed sensors are identified and distinguished. As each sensor has its own characteristic wavelength, the sensors can be connected in series on one optical line or a star configuration can be made. Using an optical switch, several hundreds of sensors can be measured with a relatively small low-cost interrogation unit. The Geodetect system has the following features.

1 Corrosion resistant.
2 Free from electromagnetic interference.
3 Radiation resistant.
4 Explosion proof (no risk of sparks).
5 Immune to lightning strikes.

The Geodetect system has been tested in the laboratory and on a full scale (Briancon *et al.,* 2004). Two simulated sinkholes were formed by deflating balloons located under a railway containing the Geodetect membrane. During deflating, the Bragg gratings located the cavities, indicating an instantaneous increase in strain as deflation commenced. Additional strain was recorded during the passage of vehicles above the voids.

The Geodetect system has been used by French Railways over an identified fault located perpendicular to the track. The length of the area treated was 50 m and the width of the track 5 m. The embankment under the track consisted of 250 mm ballast and a base of 500 mm and the Geodetect membrane was located below the base course (Nancy *et al.*, 2004). The design criteria required of the system were set as follows.

1 'Warning' level, surface settlement $s_w = 6$ mm.
2 'Slowdown' level, surface settlement $s_s = 9$ mm.
3 'Intervention' level, surface settlement $s_i = 21$ mm.

The geosynthetic reinforcement layer was designed as basal reinforcement using the following parameters (Villard *et al.*, 2000).

1 Embankment expansion coefficient $C_c = 1.1$.
2 Dead loading applied to the base (formation + ballast) $q_0 = 6$ kPa.
3 Live loading, 25.5 t per axle distributed over three rail sleepers.
4 Assumption of circular cavity 1.2 m in diameter.

With these parameters, the maximum stress q_{max} applied to the reinforcement is 45 kPa. The stiffness J of the reinforcement is 3400 kN/m.

5.4 Active geosynthetics

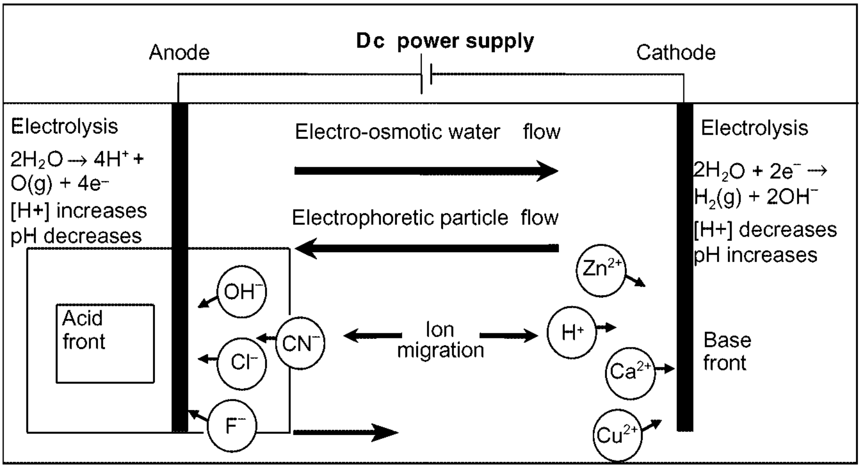
The main applications of geosynthetics are primarily related to the civil engineering and environmental industries and are well established as providing filtration, separation, reinforcement and drainage and acting as barriers. In use, conventional geosynthetic materials have a *passive* role, e.g. as barriers stopping the passage of liquids, and as reinforcement providing tensile resistance but only after an initial strain has occurred and as drains providing a passage for water but not causing water to flow. New applications for geosynthetics can be identified if the geosynthetic can provide an *active* role, initiating chemical or physical change to the soil matrix in which it is installed as well as providing the established functions. This can be achieved by creating electrically conducting geosynthetics and combining electrokinetic phenomena with established geosynthetic functions to give electrokinetic geosynthesis (EKGs).

5.4.1 Electrokinetic phenomena in soils

Electrokinetic techniques have been developed for treatment of clay soils, since their introduction as a construction technique by Casagrande in 1939. Electrokinetics, for these applications, may be defined as the application, or induction, of an electrical potential difference across a soil mass containing fluid, or a high-fluid content slurry or suspension, causing or caused by the motion of electricity, charged soil and/or fluid particles.

Electrokinetic phenomena are the result of the coupling between hydraulic and electrical potential gradients in fine-grained soils (Yeung and Mitchell, 1993; Acar and Alshawabkeh, 1994). These phenomena occur due to the presence of a diffuse double layer around grained soil particles and involve the movement of electricity, charged particles and fluids (Mitchell, 1993). Electrokinetic phenomena may be defined in terms of five categories. Of these, the three most relevant are defined below and illustrated in Fig. 5.3.

1 *Electromigration or ion migration.* The applied electrical potential difference induces ion migration within the fluid phase of a charged-particle matrix.
2 *Electrophoresis.* The applied electrical potential difference induces movement of suspended colloidal particles within a fluid medium.
3 *Electro-osmosis.* The applied electrical potential difference induces fluid flow in a charged-particle matrix.



5.3 Electrokinetic effects (after Glendinning *et al.*, 2005).

Typically electro-osmotic dewatering of clay soils is of the order of one to four orders of magnitude faster than hydraulic dewatering, with a typical value of electro-osmotic permeability k_e for a clay soil being 10^{-5} cm²/V s, as opposed to hydraulic permeability which ranges from 10^{-9} to 10^{-5} m/s for silts and clays. Actual values of k_e and k_h for a range of soils are shown in Table 5.1. The greatest advantage to be gained from electrokinetic dewatering over hydraulic dewatering is when the ratio of k_e to k_h is high.

5.4.2 Electro-osmosis

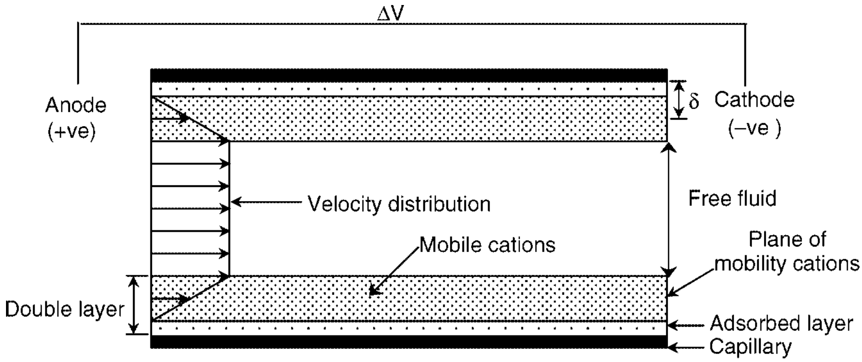
Several theories have been developed to explain electro-osmotic transport of water in clay soils. Of these, the Helmholtz–Smoluchowski theory is one of the earliest and one of the most widely used (Mitchell, 1993). It is based on an electrical condenser analogy that assumes that the soil capillaries have charges of one sign on or near the surface of the wall and countercharges concentrated in a layer in the liquid a small distance from the wall (Fig. 5.4). The mobile shell of counterions is assumed to drag water through the capillary by plug flow, resulting in a high-velocity gradient between the two plates of the condenser. The balance between the electrical force causing water movement and friction between the liquid and the wall controls the rate of water flow.

If v is the flow velocity and δ is the distance between the wall and the centre of the plane of mobile charges, then the velocity gradient between the wall and the centre of the positive charges is v/δ; thus, the drag force per unit area is $\eta\, dv/dx = \eta v/\delta$ where η is the viscosity. The force per unit area from the electric field is $\sigma \Delta E/\Delta L$, where σ *is* the surface charge density and $\Delta E/\Delta L$ is the electrical potential gradient. At equilibrium,

Table 5.1 Coefficients of electro-osmotic permeability and other parameters (after Mitchell, 1993)

Material	Water content (%)	K_e (10^{-5} cm^2/s V)	Conductivity σ (S/m)	Approximate k_h (cm/s)	K_i water per unit charge[a] (m^3/s A)
London clay	52.3	5.8	N/A	10^{-8}	N/A
Boston blue clay	50.8	5.1	N/A	10^{-8}	N/A
Kaolin	67.7	5.7	N/A	10^{-7}	N/A
Clayey silt	31.7	5.0	N/A	10^{-6}	N/A
Rock flour	27.2	4.5	N/A	10^{-7}	N/A
Na-montmorillonite	170	2.0	N/A	10^{-9}	N/A
Na-montmorillonite	2000	12.0	N/A	10^{-8}	N/A
Mica powder	49.7	6.9	N/A	10^{-5}	N/A
Fine sand	26.0	4.1	N/A	10^{-4}	N/A
Quartz powder	23.5	4.3	N/A	10^{-4}	N/A
Ås quick clay	31.0	2.0	N/A	2.0×10^{-8}	N/A
Bootlegger Cove clay	30.0	2.4–5.0	0.02	2.0×10^{-8}	1×10^{-7}
Silty clay, West Branch Dam	32.0	3.0–6.0	0.25	$(1.2–6.5) \times 10^{-8}$	$9.6 \times 10^{-9}–2 \times 10^{-8}$
Clayey silt, Little Pic River, Ontario	26.0	1.5		2×10^{-5}	
Decomposed granite, Silty clay	N/A	30	0.17	$10^{-4}–10^{-5}$	1.8×10^{-7}
Silty clay	23	5.0	0.08–0.12	$(0.5–8) \times 10^{-5}$	$(6.3–4.2) \times 10^{-9}$
Silty clay	10	9.0	0.02	1×10^{-5}	4.5×10^{-8}
Marine silty clay	37	6–9	0.024–0.033	10^{-7}	$(1.8–3.8) \times 10^{-7}$

[a] N/A, not applicable.



5.4 Helmholtz –Smoluchowski model (after Mitchell, 1993).

$$\eta \frac{v}{\delta} = \sigma \frac{\Delta E}{\Delta L} \qquad [5.1]$$

or

$$\sigma\delta = \eta v \frac{\Delta L}{\Delta E} \qquad [5.2]$$

From electrostatics, the potential gradient ζ across a condenser is given by:

$$\zeta = \frac{\sigma\delta}{D} \qquad [5.3]$$

where

D = relative permittivity or dielectric constant of the pore fluid

Substitution of σ and δ into Equation 5.3 gives

$$v = \left(\frac{\zeta D}{\eta}\right) \frac{\Delta E}{\Delta L} \qquad [5.4]$$

The potential ζ is the zeta potential, which is not equal to the surface potential of the double layer. For a single capillary of area a, the flow rate is

$$q_a = va = \frac{\zeta D}{\eta} \frac{\Delta E}{\Delta L} a \qquad [5.5]$$

and for a group of N capillaries with a total cross-sectional area A normal to the flow direction,

$$q_A = Nq_a = \frac{\zeta D}{\eta} \frac{\Delta E}{\Delta L} Na \qquad [5.6]$$

If the porosity is n, then the cross-sectional area of the voids is nA, which must equal Na. Thus,

$$q_A = \frac{\zeta D}{\eta} n \frac{\Delta E}{\Delta L} A \qquad [5.7]$$

By analogy with the Darcy law, this may be rewritten as

$$q_A = k_e i_e A \qquad [5.8]$$

where

$$i_e = \frac{\Delta V}{\Delta L} = \text{electrical potential gradient (V/m)} \qquad [5.9]$$

$$k_e = \frac{\zeta D}{\eta} n = \text{coefficient of electro-osmotic permeability} \qquad [5.10]$$

This is the equation that describes the flow of water under an electrical potential gradient.

5.4.3 Electro-osmotic permeability

Unlike hydraulic permeability, k_e is relatively independent of pore size (Table 5.1). Casagrande (1952) suggested that for most practical applications a value for k_e = 5×10^{-5} cm^2/s V can be accepted. Therefore, it can be seen that electro-osmosis can be effective for water movement in fine-grained soils compared with water flow under hydraulic gradients.

5.4.4 Electro-osmotic efficiency

The efficiency of electro-osmosis relates to the quantity of water moved per unit charge passed (Grey and Mitchell, 1967). According to Mitchell (1993), this factor may vary by several orders of magnitude depending upon the soil type.

The efficiency of electro-osmosis depends upon the electrical conductivity of the soil mass. The electrical conductivity is in turn controlled by the water content, the cation exchange capacity and the free electrolyte concentration of the pore fluid, as indicated by Grey and Mitchell (1967) and validated by Lockheart (1983). The mineralogy of the soil itself also has a major implication upon the soil conductivity.

5.4.5 Energy requirements

The quantity of water moved per unit charge passed (l/h A or mol/F) is a measure of the viability of using electro-osmosis on a particular site. If this quantity is expressed as k_i then

$$q = k_i l \quad [5.11]$$

where k_i can vary over a wide range. The power consumption is defined by

$$P = \Delta E\, I = \frac{\Delta E\, qh}{k_i} \quad [5.12]$$

where ΔE is in volts and I in amperes.

The power consumption per unit volume of flow is

$$\frac{P}{qh} = \frac{\Delta E}{k_i} \times 10^{-3}\ \mathrm{KW\,h} \quad [5.13]$$

5.4.6 Relationship between k_e and k_i

The electro-osmotic flow rate is given by

$$q_h = k_i l = k_e \frac{\Delta E}{\Delta L} A \quad [5.14]$$

However, $\Delta E/I$ is the resistance and $\Delta L/(\text{resistance} \times A)$ is the specific conductivity σ; hence

$$k_i = \frac{ck_e}{\sigma} \quad [5.15]$$

where

k_e = electro-osmotic permeability (cm/s V cm)
k_i = electro-osmotic efficiency (gal/A h)
σ = specific conductivity (S/m)
C = constant = 1.0 using the stated units

Equation [5.15] is useful in that k_e varies within narrow limits as shown in Table 5.1. Hence, the electro-osmotic efficiency measured by k_i is dependent upon the electrical specific conductivity σ of the soil.

5.4.7 Applied voltage

Electro-osmosis may be carried out by applying a constant current or constant voltage. During treatment, the resistivity of the soil changes owing to electrochemical changes and desiccation. When a constant voltage is applied, the magnitude of the current decreases corresponding to the increase in the resistance of the soil mass being treated. Similarly, if a constant current is applied, the corresponding applied voltage will increase. Hamir (1997) carried out a series of basic tests to assess the difference between constant-voltage and constant-current tests. The findings of his investigation were that, in terms of the properties of the treated soil, no distinction between the two methods could be found.

Polarity reversal (the reversal of electrodes or current direction) has a marked effect on the effectiveness of dewatering in producing more uniform conditions. In addition, current intermittence has been shown to increase the efficiency of electro-osmotic dewatering. The principles of the improvement are quantitatively interpreted in terms of the depolarization of the double layer associated with current intermittence. It has been shown that electro-osmotic strengthening of marine sediments is enhanced by the use of current intermittence with optimum conditions being obtained with intermittence intervals as 2 min on and 1 min off (Mohamedelhassan and Shang, 2001). Another benefit is the reduction in anode corrosion (Micic *et al.*, 2001).

5.4 8 Pore water pressures

For certain applications such as soil consolidation, the generation of negative pore water pressure is the principle operator in the strengthening process, and negative pore pressure is proportional to the applied voltage.

Esrig (1968) has presented solutions for the development of pore water pressures under an applied electrical potential field. The solution is based upon the following assumptions.

1 The soil is homogeneous and in a fully saturated state.
2 The physical and physiochemical properties of the soil are uniform throughout and are constant with time.
3 Electrophoresis of fine-grained soil particles does not occur.
4 There is proportionality between the electrically induced velocity v of water flow through the soil and the voltage gradient V. The proportionality factor is the coefficient of electro-osmotic permeability i.e. the Helmholtz–Smoluchowski equation is applicable.

$$v = k_e \frac{\partial V}{\partial x}$$

5 All applied voltages are effective in moving water.
6 The electric field produced throughout the soil mass is constant with time.
7 No reactions occur at the electrodes (e.g. electrolysis).
8 Fluid flows due to an electric field and due to a hydraulic gradient may be superimposed to find the total fluid flow.

Some of these assumptions are inherently incorrect.

1 A natural material is very rarely, if ever, homogeneous.
2 The resistivity of the soil undergoing electro-osmosis has been shown to vary with both position and time because of both desiccation and electrochemical changes.
3 Separation of the electrodes from the soil may also occur owing to gas formation caused by electrolysis.

4 The physical and physiochemical properties of the soil vary with time, as does the electric field.
5 Electrolysis occurs during all electro-osmotic treatments.

The assumption that fluid flow caused by an electric field may be superimposed on the flow caused by a hydraulic gradient is important. Grey and Mitchell (1967) demonstrated the validity of the theory indirectly through the measurement of streaming potentials resulting from hydraulic flows. Wan and Mitchell (1976) showed that the superposition of pore water pressures generated by surcharge loading and electro-osmosis was valid. Accepting the validity of superposition of electrical and hydraulically driven flows through an incompressible soil mass and limiting consideration to one dimensional flow gives

$$\frac{\partial v}{\partial x} + \frac{\partial V_h}{\partial x} = 0 \tag{5.16}$$

where

v = velocity of flow due to the electrical potential gradient $\partial V/\partial x$ (i.e. the Helmholtz–Smoluchowski)
V_h = the velocity of flow due to the hydraulic gradient (i.e. Darcy's equation)

Differentiation of Equation [5.16] with respect to x and inserting the Helmholtz–Smoluchowski and the Darcy equations give

$$k_e \frac{\partial^2 V}{\partial x^2} + \frac{k}{\gamma_w} \frac{\partial^2 u}{\partial x^2} = 0 \tag{5.17}$$

Rearranging gives

$$\frac{\partial^2 u}{\partial x^2} + \frac{k_e \gamma_w}{k} \frac{\partial^2 V}{\partial x^2} = 0 \tag{5.18}$$

and substitution of the variable ξ, where

$$\xi = \frac{k_e \gamma_w}{k} V + u \tag{5.19}$$

gives

$$\frac{\partial^2 \xi}{\partial x^2} = 0 \tag{5.20}$$

which is the Laplace equation in a one-dimensional system. Integrating once gives

$$\frac{\partial \xi}{\partial x} = C_1 \tag{5.21}$$

and integrating again gives

$$\xi = xC_1 + C_2 \tag{5.22}$$

where

C_1, C_2 = constants of integration dependent upon the boundary conditions.

The possible drainage conditions that may exist at the electrodes, which govern these boundary conditions, are discussed in the following section.

5.4.9 Electrode drainage conditions and pore water pressures

The drainage conditions applied to either the anode or the cathode have one of the following two conditions.

1 *Open.* This condition exists when the electrode is open to the atmosphere, such that no excess pore pressure can exist at the electrode, i.e. $u = 0$. This condition will occur where an EKG electrode is utilized with an external filter in place, or where a more conventional metallic-type electrode is used together with an associated drainage path, e.g. sand drain or hollow perforated electrode. An open electrode may also be pumped, recharged or simply allowed to overflow.
2 *Closed.* This condition exists when the electrode is sealed such that no passage of gas or fluid can take place along the length of the electrode.

The configurations used for consolidation and/or dewatering in practice are as follows.

1 Open, anode not recharged; open, cathode with overflow.
2 Closed, anode; open, cathode with overflow.
3 Closed, anode; open, cathode pumped.

5.4.10 Electrolysis effects associated with electro-osmosis

During electro-osmotic treatment of a soil mass, the whole system acts as an electrochemical cell in which cations migrate to the cathode and anions migrate to the anode. Reduction reactions take place at the cathode, with the principal reaction being the reduction of water to hydrogen gas. At the anode, there is the possibility of two distinct oxidation reactions: the oxidation of the anode material itself into its oxide or the oxidation of water to yield oxygen. Hence, the anode will degrade, with the rate of degradation being dependent upon the material from which the anode is manufactured. It is the degradation of the anode that has caused the most problems with the application of electro-osmotic treatment on an industrial scale. Corrosion of the anode is accompanied by a build-up of corrosion products, which degrade the electrical efficiency of the process. The process is also influenced by the production of gas at the electrodes, which must be vented for the process to continue. A schematic diagram of the movement of current and the electrolytic reactions that may occur during electro-osmotic treatment are

$$2H_2O + O_2 + 4e^- \rightarrow 4OH^- \text{ or } 2H^+ + 2e \rightarrow H_2 \qquad [5.23]$$

$$\text{iron anode: } Fe + H_2O \rightarrow FeO + 2H^+ + 2e^- \text{ or } 2H_2O \rightarrow O_2 + 4H^+ + 4e^- \qquad [5.24]$$

$$\text{copper anode: } Cu + H_2O \rightarrow CuO + 2H^+ + 2e^- \text{ or } 2H_2O \rightarrow O_2 + 4H^+ + 4e^- \qquad [5.25]$$

$$\text{carbon anode: } C + 2H_2O \rightarrow CO_2 + 4H^+ + 4e^- \qquad [5.26]$$

The liberation of hydrogen ions at the anode causes a reduction in pH, and an increase in hydroxide ions at the cathode causes an increase in pH. If these ions are not removed or neutralized from the anode and cathode, electromigration and/or ion migration will occur with hydrogen ions moving towards the cathode (acid front) and hydroxide ions moving towards the anode (base front). The advance of acid and base fronts is governed by ion migration as well as by electro-osmotic flow, diffusion and the buffering capacity of the soil medium. Owing to a higher ratio of charge to ionic size, H^+ ions move more rapidly than OH^- ions during electromigration. Additionally, electro-osmotic flow occurs in the same direction as H^+ migration, but it opposes the direction of OH^- migration. As a consequence, the acid front from the anode moves more quickly than the base front from the cathode, so that acidic conditions prevail throughout most of the soil mass.

5.4.11 Evolution of active geosynthetics

The problems relating to the degradation of the electrodes and gas and water production can be overcome by forming them of materials not susceptible to corrosion and also providing inbuilt drainage facilities for the gases produced and the water collecting at the cathode. This can be achieved by using electrically conducting geosynthetic materials to form the electrodes. The resultant materials differ from conventional geosynthetics in that they are active in the sense that they cause change in soils rather than acting in a passive manner as is the case with conventional materials (Jones *et al.*, 2005).

Jones *et al.* (1996) introduced the concept of EKG materials, defining them as a range of geosynthetics, which, in addition to providing filtration, drainage and reinforcement can be enhanced by electrokinetic techniques for the transport of water and chemical species within fine-grained low-permeability soils, which are otherwise difficult or impossible to deal with. In addition, transivity, sorption, wicking and hydrophobic tendencies may also be incorporated in the geosynthetic to enhance other properties. The EKG can take the form of a single material which is electrically conductive, or a composite material, in which at least one element is electrically conductive. They can be of the same basic form as present-day filter, drainage, separator and reinforcement materials but offer sufficient electrical conduction to allow the application of electrokinetic techniques.

Jones *et al.* (1996) undertook a series of laboratory studies to evaluate the use of conductive geotextiles as electrodes in electro-osmotic consolidation and reinforced soil. The types of geosynthetic used included needle-punched geotextiles with copper wire of 1 mm diameter inserted into the geotextile to make it electrically conductive, conductive fibre (carbon) needle-punched material and modified polyester reinforcing tape. The latter was made electrically conductive by the addition of metal stringers aligned parallel to the polyester reinforcing elements. The results of the tests were favourable and indicated that the EKG behaved as well as a conventional copper electrode.

In the reinforced soil tests, the EKG reinforcement was used as an anode, with the cathode formed from a needle-punched EKG fabric. The results of pull-out tests showed an increase in reinforcement bond of up to 211% and increases in shear strength of up to 200% compared with the values obtained when the geosynthetics were not electrically conductive, (Hamir, 1997). Nettleton *et al.* (1998) continued the work presented by Jones *et al.* (1996) and suggested that a band-drain-type electrode would be the most suitable configuration to fulfil all the electrode requirements associated with consolidation, bioremediation and moisture control in embankments.

Current EKG materials are formed using special conductive elements woven knitted or needle punched into conventional geosynthetic products to form drains, tubes, bags, grid or sheet elements (Hamir *et al.*, 2001; Electrokinetic, 2005).

5.4.12 Development of soil acceptability criteria for electrokinetic treatment

Acceptability criteria for the electrokinetic treatment of soils have been developed on the basis of standard and non-standard soil mechanics tests. The basis for advancing these criteria is to permit a relatively rapid assessment of the suitability of a soil for treatment by electro-osmosis based upon standard soil mechanics laboratory tests. This does not mean that electro-osmosis specific tests are not required in order to predict the performance of a particular site installation, but it permits an informed decision to be made at an early stage as to whether to proceed with the more specialized testing.

General classification tests

The liquid and plastic limits of a soil define the range within which a cohesive soil behaves in a plastic state. The value of the Atterberg limits of a cohesive soil depends upon several factors, including the quantity and type of clay mineral and type of absorbed cation. Typical values for the Atterberg limits for different clay mineral types are given in Table 5.2. It can be seen that the different clay mineralogies have a direct bearing upon the values of the Atterberg limits. It has also been shown that the electrical conductivity of a soil is influenced, to some extent, by the surface charge density A_0 and is dependent upon the clay mineralogy. Hence, the relationship between the Atterberg limits and the electrical

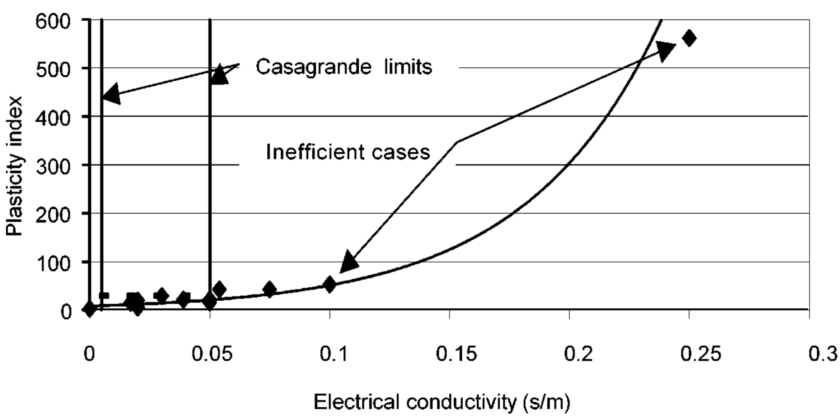
Table 5.2 Atterberg limits and associated parameters for different clay minerals (after Lambe and Whitman, 1969 and Mitchell, 1993)

Clay mineral type	Liquid limit (%)	Plastic limit (%)	Activity	Shrinkage limit (%)
Montmorillonite	100–900	50–100		8.5–1.5
Nontronite	37–72	19–27		
Illite	60–120	35–60	0.5–1.0	15–17
Kaolinite	30–110	24–40	0.5	25–29
Hydrated halloysite	50–70	47–60		
Dehydrated	33–55	30–45		
Halloysite	160–230	100–120	0.5–1.2	7.6
Attapulgite	44–47	36–40		
Chlorite	200–250	130–140		

conductivity of the soil can be linked to soil mineralogy. In addition to the mineralogy, the electrical conductivity of the soil i*n situ* is partially governed by the electrical conductivity of the pore fluid and the water content.

The chemistry of the pore water, which is also reflected in the exchangeable cations present within the clay minerals, influences the range of results for the Atterberg limits presented in Table 5.2. The monovalent cations (e.g. Na^+ and K^+) give higher values of liquid and plastic limits whereas the presence of divalent and trivalent cations (e.g. Mg^{2+}, Fe^{2+} and Al^{3+}) give lower values. If the soil is tested repetitively, the exchangeable cations can be flushed from the clay, and a change in the Atterberg limits takes place. Thus, for samples that are to be tested with a view to utilizing the results for prediction of the viability of electro-osmosis, it is important that the tests are undertaken in accordance with BS 1377: Part 2 (British Standards Institution, 1990) and not repetitively wetted and dried.

The validity of the correlation proposed for the acceptability of soils based upon the Atterberg limits and conductivity is demonstrated in Fig. 5.5. The results



5.5 Conductivity against plasticity index for a range of natural soils (after Pugh, 2002).

presented in Fig. 5.5 relate to a review of published electro-osmotic case studies where both the plasticity indices and the electrical conductivities have been given (Bjerrum *et al.*, 1967; Casagrande, 1952; Casagrande *et al.*, 1961; Hamir, 1997; Pugh, 2002).

The delineation of acceptable electrical conductivities *(σ)* and, hence, plasticity indices is based upon the limits proposed by Casagrande (1952) that an acceptable and economic range for the electrical conductivity is 0.05 – 0.005 S/m. This range gives an associated acceptable range of plasticity index in the range from 5 to 30%. This criterion is adopted for the assessment of soils.

Chemical and electrochemical tests

The standard chemical and electrochemical tests may be carried out in accordance with BS 1377: Part 3 (British Standards Institution, 1990) with the soil samples being taken and prepared in accordance with BS 1377: Part 1 (British Standards Institution, 1990).

Organic content

The presence of organic matter in a soil can have a significant effect upon the cation exchange capacity, especially in high-pH conditions and can therefore have an effect upon the electro-osmotic efficiency of the soil to treatment. Therefore, if the soil may be described as an organic clay or silt, it is recommended that specific electro-osmotic testing be carried out to ascertain the soil's suitability.

Electrical resistivity

The electrical resistivity ρ of the soil may be determined in accordance with BS 1377: Part 3:1990 Part10 (British Standards Institution, 1990). The disc electrode method is the most appropriate. Electrical resistivity may be related to conductivity σ by

$$\sigma = \frac{1}{\rho} \tag{5.27}$$

The range of acceptable and economic values of electrical conductivities σ within the range 0.05–0.005 S/m are acceptable. Values in excess of this range do not indicate that the soil is not susceptible to treatment by electro-osmosis, but that the electro-osmosis installation will draw a high current and may not be economic.

Compressibility and permeability tests

The standard compressibility and permeability tests may be carried out in accordance with BS 1377: Part 5 (British Standards Institution, 1990). The falling-head permeability test, which is the most appropriate for fine-grained soils, is not covered by current British or ASTM standards and the method given by Head (1982) is recommended.

One-dimensional consolidation parameters

Electro-osmotic consolidation is normally two dimensional, in that the electric field and the flow of water and development of pore water pressures are in the same direction, whereas the surface settlements induced are in an orthogonal direction. BS 1377: Part 5 (British Standards Institution, 1990) recommends that the soil specimen should be oriented such that the soil will be loaded in the same direction relative to the stratum as the applied stress *in situ*. Electro-osmotic consolidation via vertical drains or electrodes is equivalent to consolidation using surcharging and vertical sand drains, in the sense that the applied pressure and drainage occur in two dimensions, therefore, the coefficients of radial drainage are appropriate (Johnson, 1970; Das, 1997).

Shear strength–total stress

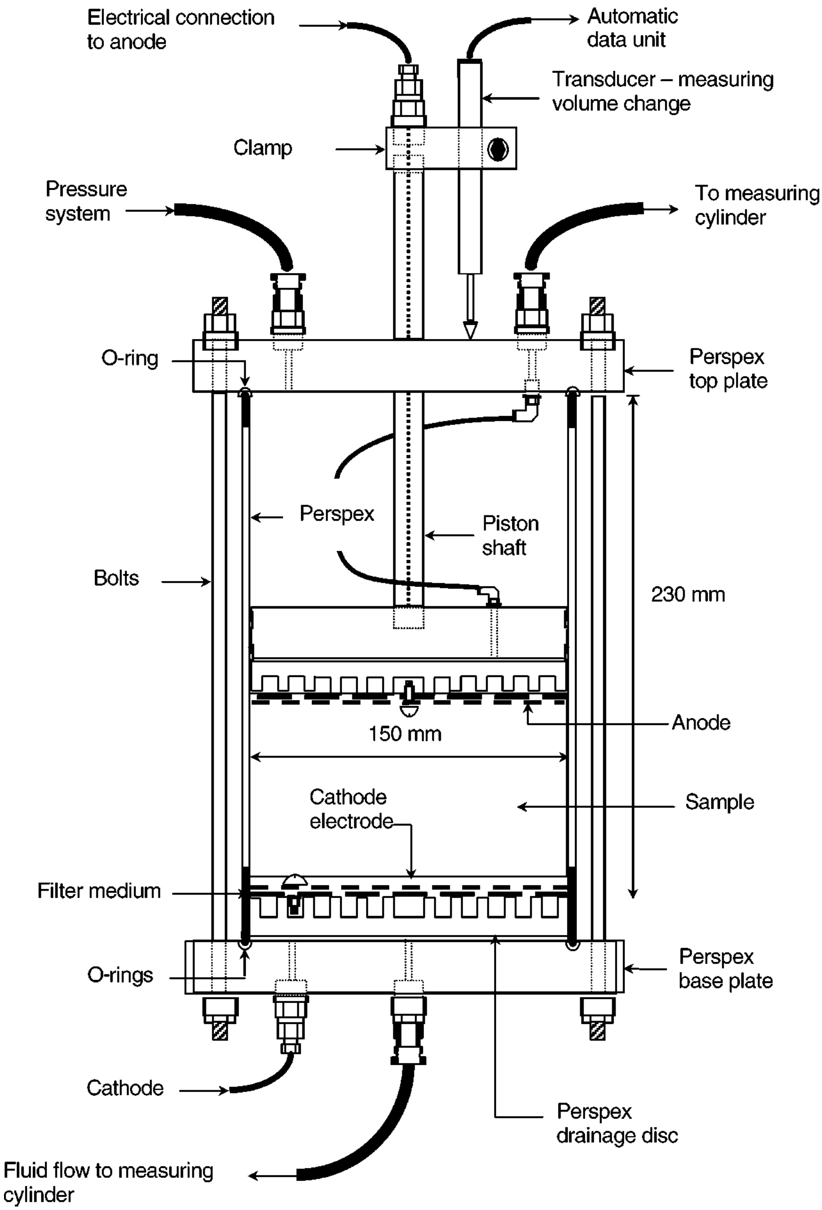
A review of the available literature for the range of c_u for soils that have been successfully treated by electrokinetic techniques indicates a range 0.7–81 kPa (Casagrande *et al*., 1961; Bjerrum *et al*., 1967; Wan and Mitchell, 1976; Lo *et al*., 1991; Milligan, 1994; Abiera *et al*., 1999). Comparison of the liquidity index criteria with the undrained shear strength criteria indicates that preferable soils (i.e. with a liquidity index greater than 0.6) will have a c_u of less than 20 kPa whereas soils that may still be acceptable (i.e. with a liquidity index greater than 0.2) will have a c_u of less than 55 kPa.

Shear strength–effective stress

Table 5.3 presents typical values for the range of ϕ' for silts and clays that are amenable to treatment by electro-osmosis. Electro-osmosis may be expected to increase the value of ϕ' for treated soils as Pugh (2002) and Casagrande (1952) have demonstrated for London clay and fine quartz sand, respectively, in that the

Table 5.3 Typical ϕ' values for compacted clays (after Carter and Bentley, 1991)

Soil type	Unified classification system designation	ϕ' (deg)
Silty clays	SM	34
Silts and clayey silts	ML	32
Clays of low plasticity	CL	28
Clayey silts, elastic silts	MH	25
Clays of high plasticity	CH	18



5.6 Electro-osmotic cell (not to scale) (after Hamir, 1997).

electro-osmotic process causes a shift in the particle size distribution to a coarser grain size.

Specialist electro-osmotic testing

The suitability of a material for electrokinetic treatment is best determined using an electro-osmotic cell. The cell used with EKG materials was originally developed by Banerjee and Vitayasupakorn (1985) and later modified by Hamir (1997). The cell is shown schematically in Fig. 5.6 and consists of a Perspex cylinder with a fixed base plate and an internal moveable piston whose movement may be monitored by means of a linear variable-displacement transducer. Provision is made for the location of disc-type electrodes both on the piston and on the fixed base plate by means of cable glands that permit the passage of an electrical cable into the cell, without the loss of pressure. Additionally, the cell incorporates side ports through which the water pressure and voltage gradient of pores may be measured if required by means of a hypodermic needle tipped with porous ceramic. The chamber behind the moveable piston may be pressurized to apply a consolidation pressure to the soil sample. Back pressure may also be applied to the soil sample through tubing which passes through the piston and the base plate; this tubing also acts as a drain for any excess pore water pressure. An alternative to the electro-osmotic cell is the use of an electro-osmosis box described by Nettleton (1996) and Adali (1999).

The usefulness and relevance of the various tests, which can be undertaken to assess the suitability of materials suitable for electrokinetic treatment, are shown in Table 5.4.

Table 5.4 Usefulness of soil tests for assessing acceptability for electro–osmosis (after Pugh, 2002)

Test	Usefulness[a]	Acceptability range
Atterberg limits	✓✓✓	Plasticity index, 5–30%
Water content	✓✓✓	Liquidity index, 0.6–1.0
Particles size distribution (sieve/sedimentation)	✓✓✓✓	See British Standards Institution (1986)
Particle density	✓	Not applicable
Organic content	✓✓	Up to organic
One-dimensional consolidation parameters	✓✓✓	$mv = 0.3–1.5\ MN/m^2$
Disc electrode	✓✓✓	0.5–0.005 S/m
Hydraulic permeability	✓✓✓	< 1–8 m/s
Undrained shear strength	✓✓	< 55 kPa
Drained shear strength	✓	$\phi' < 30°$
Electro-osmosis cell	✓✓✓✓	Not applicable
Electro-osmosis box	✓✓✓✓	Not applicable

[a] ✓✓✓✓, excellent; ✓✓✓, good; ✓✓, reasonable; ✓, poor.

5.4.13 Civil and environmental applications of active geosynthetics

It has been established that conductive geosynthetics, acting as electrodes, can be used to effect the movement of contaminants through soil to the electrodes and then to adsorb them. Using electrophoresis it is possible to dewater industrial wastes, which are currently untreatable and can only be disposed of in tailing lagoons.

Conductive geosynthetics can be used to consolidate soil or to reduce the volume of industrial wastes by electro-osmosis, thus significantly lowering the cost of disposal. The use of conductive reinforcement can permit the use of fine, very wet material as fill for reinforced structures. Initial studies have shown that the use of electrically conductive band drains could prevent liquefaction of susceptible soils during earthquakes.

Other applications can be seen in different fields such as sport and horticulture. The use of conductive geosynthetics can resolve some of the problems inherent in many large sports stadia, such as the detrimental effect of shade on growing surfaces. An electrically conductive geosynthetic laid as a continuous porous membrane at root level provides oxygen directly to the root system as well as offering a method to control drainage, aeration and ball bounce (Lamont-Black *et al*., 2003). A working model of this technology was exhibited in the Science Museum in London during the 2002 Football World Cup.

Three basic applications areas can be identified for active geosynthetics.

1 Accelerated settlement of solids from liquids (electrophoresis).
2 Dewatering to reduce volume alone (electro-osmosis).
3 Consolidation to improve strength by consolidation (electro-osmosis).

The application areas of active geosynthetics can be further identified by consideration of the electrical conductivity and the water content of the material to be treated (Table 5.4). The use of EKG reinforcement represents an extension of current reinforced soil technology which has important implications with respect to the type of fill that can be used with these structures.

The application of EKGs for the construction of reinforced soil, consolidation of weak soil and sports applications has been reported by Pugh *et al*. (2000), Jones and Pugh (2001), Lamont-Black *et al.* (2003), Glendinning *et al*. (2005) and Jones *et al*. (2005). The results presented demonstrate that EKGs can lead to improvements and forms of construction beyond those that can be achieved by conventional methods. The use of active geosynthetics to stabilize mine tailings has been reported by Fourie *et al*. (2002).

5.5 Future trends

5.5.1 Novel applications of active geosynthetics

A number of novel applications for active geosynthetics have been proposed including those by Jones *et al*. (2005).

Trenching and excavation

It would be possible to combine EKG dewatering with conventional well pointing technology in low-permeability soils to achieve a more rapid drawdown of the phreatic surface than that currently possible. An additional benefit of electro-osmotic dewatering is that the soil properties may be improved which will make excavation easier and more stable.

Earth pressure balance machine or mining methods could be enhanced using EKG technology to improve the soil conditions in the vicinity of the tunnel or to reduce post-construction settlements associated with the tunnel. If access to the surface was also available, the EKG could be installed at the surface to assist the process. In addition, electrokinetics could be used in very soft soils to reduce the possibility of chimney formation.

Piling

Electrokinetic phenomena can assist with piling in two principal ways. If EKGs are installed in close proximity to the piling area and steel piles are made into the cathodes the result would be to soften the soil locally in the vicinity of the piles and to facilitate driving. Once the desired piling depth has been achieved, the electrokinetic process can be shut down and the soil will then gradually regain its equilibrium and the soil properties return to their former state around the piles. Preferentially, the polarity could be reversed, making the piles anodic and therefore improving the soil properties in the vicinity of the piles. The latter process has been successfully employed in Canada (Milligan, 1994).

Enhanced lime migration

EKGs also have a potential application in improving existing lime pile techniques. In this case, the EKG could be used as a conductive element to establish electro-osmotic flow to induce the migration of calcium ions through the soil, thereby increasing the zone of influence of the pile by causing lime migration to a greater radius than occurs by conventional diffusion.

Slope stability

The stability of slopes may be improved through the use of an EKG to achieve a more rapid dewatering of the soil and to act as drains to reduce pore water pressures within an unstable cohesive slope. The application of a potential difference generates negative pore water pressures at the anode and increase the soil strength and an increase in bond between the EKG nail or anchor and the soil, resulting in an overall increase in the stability of the slope.

Shrinkage and swelling prevention of shallow foundations and pipelines

EKG technology may be able to offer a remedy to the problem associated with low-rise structures built with shallow foundations, which are constructed on soils prone to shrinkage and swelling. In this case, the EKG would allow moisture control of the susceptible strata and either add or remove water as necessary to prevent the volume change of the founding stratum.

Remediation of contaminated sites

The use of electrokinetic techniques to provide *in situ* treatment of contaminated soils is a developing technology. Active geosynthetics in the form of EKG materials have been shown to facilitate this form of treatment including the application of *in situ* bioremediation (Nettleton, 1996).

Electrokinetic dewatering of geotubes

Geotubes used to contain and dewater difficult waste materials can be enhanced using electrokinetic techniques. In this application the geotube is formed as a cathode with an anode installed within the waste material.

Improvement in process dewatering

The treatment and disposal of sewage sludge are one of the most problematical issues affecting waste water treatment in the developed world. It has been shown that electrokinetic dewatering of sludge is more efficient than conventional hydraulic-driven methods and this is anticipated to be a major application of electroconductive geosynthetics.

Improvement in horticultural processes

Conductive geosynthetics have been shown to provide technical benefits to the sports turf industry; the same principles can be transferred to horticulture.

Production of fertilizer from sewage waste

One method of treating humic sludge is by thickening, pressing or centrifuging and then drying in the open air in elongated stockpiles or windrows. Wood waste is added to the sludge to improve the mechanical handling characteristics; this has a cost implication and results in a significant increase in volume to be finally disposed of. Electrokinetic treatment of the windrows has shown that the drying process can be accelerated and that there is the potential for the use of green waste in place of wood waste which has major economic potential. In addition, the product of the electrokinetic treatment could be used as fertilizer.

5.6 Sources of further information

General information relating to geosynthetics and related products can be obtained from the web site of the International Geosynthetics Society (IGS) (2006).

The conferences run under the auspices of the IGS cover the latest developments of the technology and all major manufacturers of geosynthetics are members of the IGS.

There are two official journals of the International Geosynthetics Society: *Geotextiles and Geomembranes* published by Elsevier, and *Geosynthetics International* published by Thomas Telford. Both are available online.

Information on electrically conductive geosynthetics is available from Electrokinetic Ltd (2006)

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6

The use of geosynthetics as filters in civil engineering

J. FANNIN
University of British Columbia, Canada

Filters are typically constructed as part of a drainage system and have a wide range of applications in civil engineering works. The primary objective of the filter is to protect against soil erosion in applications where groundwater flow has the potential to cause a seepage-induced movement of particles while, at the same time, to provide for adequate discharge capacity and therefore unimpeded drainage of the soil. Accordingly, properly designed filters are integral to the performance of construction works, both with respect to the economic concerns governing serviceability and also for safety concerns governing stability at the ultimate limit state.

6.1 Introduction

Applications in which filters are commonly specified include drainage systems behind retaining walls, adjacent to roads and foundations, within slopes, and under hydraulic structures, landfills and sports fields. Consider, for example, placement of a geotextile filter for stabilization of the cut slope illustrated in Fig. 6.1; where groundwater exits the lower portion of the slope, the geotextile provides for retention of erodible soils and permits seepage flow to exit freely into an overlying blanket of rock. In many construction applications, the design life of the filter and companion drainage system is typically the same as that of the constructed works. However, this is not always the case. Consider the use of prefabricated vertical drains to accelerate consolidation of compressible soils under an embankment, a case of ground improvement for which the design life of the filter is considerably shorter than that of the related engineering works.

The principle of using a filter zone to control groundwater seepage and to protect against erosion was studied by Karl Terzaghi, with application to foundation design of small weirs, for which he was first granted a patent in 1922. Further studies led directly to applications in large-zoned earthfill dams, and the development of empirical rules for specifying the grain size distribution of the filter layer. The filter medium consisted of one or more select gradations of cohesionless soil,

6.1 Cut-slope stabilization: geotextile filter and rock blanket.

for which the characteristic grain sizes D_n are reported from laboratory testing. In effect, for a granular filter, by specifying directly the size distribution of grains, the corresponding size distribution of openings in the porous medium is determined indirectly. Hence, the opening size distribution is governed by soil type and may be influenced by its method of field placement and any potential for segregation during placement. In contrast, the opening size distribution of a geotextile is controlled by the process of manufacturing, for which one or more characteristic values of opening size O_n in the fabric are established directly by means of laboratory testing and reported by the manufacturer with reference to a statistical database for purposes of quality control.

6.1.1 Geotextiles

The manufacturing process yields several styles of geotextile, two of which, a non-woven and a woven fabric, are typically used in filtration applications. The styles are inherently different. A non-woven geotextile consists of a layer of many randomly oriented polymer strands that are bonded to obtain a planar fabric. The individual strands are usually a short fibre or a continuous filament, generally made of polypropylene and occasionally of polyester or polyethylene. The common methods of bonding are either physical entanglement of the strands, yielding a needle-punched non-woven geotextile, or thermal fusing of contact points between the strands during a calendering operation, which produces a heat-bonded non-woven geotextile. In contrast, a woven geotextile is made from individual polymer strands that are aligned and interweaved on an industrial loom, again yielding a planar fabric. The strand itself is usually a tape, a monofilament or a multifilament yarn. A fibrillated strand is one that has been intentionally split along portions of its length, as a part of the manufacturing process, to condition its properties. Geotextiles are supplied on a roll, with the machine direction being perpendicular to the roll, and the cross-machine direction being parallel to the roll.

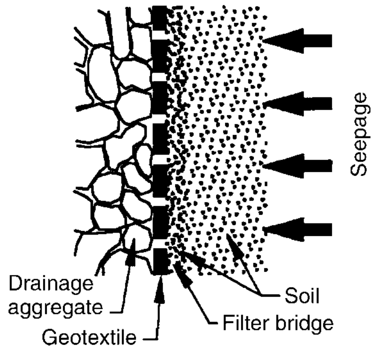
Inherent differences between each of these manufacturing processes, and the resulting style of a geotextile, impart subtle differences to the characteristic opening size and opening size distribution of the fabric and, by association, to the capacity for flow of water across the plane of the fabric. It also imparts variations in tensile strength and stiffness. Accordingly, in the design approach, assessment of a candidate geotextile for a proposed filtration application must account for a series of compatibility requirements.

6.2 Compatibility requirements

Filtration compatibility requires that there is no unacceptable erosion as a consequence of soil loss through the geotextile while, at the same time, provision is made for unimpeded flow of water seeping from that soil. Therefore, the principal requirements for compatibility are those of, firstly, soil retention and, secondly, cross-plane permeability. They represent competing interests, insomuch as soil retention is assured by relatively small pore-size openings in the geotextile whereas cross-plane permeability is assured by many relatively large pore-size openings. In addition to these geometric and hydraulic provisions, there is also a requirement for, thirdly, provision of adequate strength to ensure that the geotextile is not damaged in the process of installation and can accommodate, thereafter, any loads that are imposed on it over the service life of the installation.

Consider the first requirement, for soil retention. Filtration compatibility is contingent on the fact that the geotextile has a distribution of pore-size openings that prevents any significant movement of soil particles through those openings. The expectation, as with granular filters, is that retention of coarser particles in the soil then promotes the development of a stable interface or 'bridging zone' in a thin zone of soil adjacent to geotextile (Fig. 6.2). Given this expectation, the design approach is predicated on matching a characteristic pore size opening of the geotextile (O_n, e.g. O_{95}) to a characteristic particle size of the soil (D_n, e.g. D_{85}). The approach is very similar to that adopted in granular filters, where a characteristic particle size of the filter (e.g. D_{15}) is used as a default measure of opening size (e.g. D_{15}) and matched to a characteristic particle size of the soil.

With respect to the second requirement, for cross-plane permeability, filtration compatibility is contingent on the fact that the geotextile has a capacity for discharge flow significantly greater than that of the soil against which it is placed. The expectation, as for granular filters, is that, if each successive layer in the direction of seepage flow exhibits a greater permeability, there is no potential to



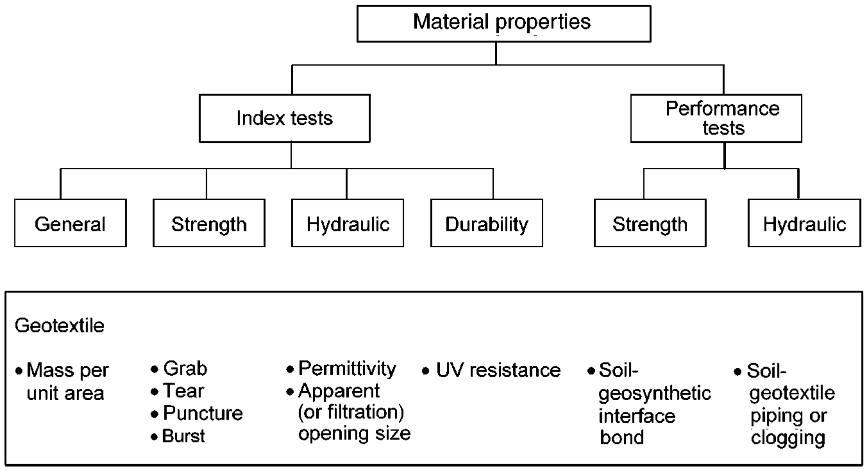
6.2 Soil retention by means of a bridging zone (after Holtz *et al.*, 1997).

impede discharge flow through those layers. The design approach is commonly based on matching an index value of cross-plane permeability (k_n) for the geotextile to the permeability (k_s) of the soil. The approach differs from that adopted for granular filters, where the permeability of both filter and soil are characterized, indirectly, by a characteristic particle size (e.g. D_{15}).

In considering the third requirement for strength, filtration compatibility is contingent on the fact that the geotextile exhibits adequate capacity to resist loads mobilized in the polymer strands as a consequence of localized deformation, without any loss of integrity in the fabric. The greatest demand on the geotextile is typically that encountered during the installation process. Thereafter, it must also be sufficiently durable to ensure that those loads can be sustained over the service life of the installation. The design approach is usually based on one or more index values of strength that are used to categorize the geotextile according to a classification system (e.g. a three-class system of high, moderate or low strength). A required strength class is then established for the application, with reference to the anticipated severity of the installation process. Durability over the service life of the structure is often addressed with reference to the polymer from which the geotextile is manufactured, and appropriate provisions for ultraviolet (UV) protection, taking into account the nature of the soil and groundwater chemistry. Although the approach again differs substantially from that used for granular filters, which do not exhibit any tensile strength, it is similar with regard to the need to ensure durability of the filter medium itself.

6.3 Material properties for design

Geotextile properties are established from laboratory testing of specimens taken from a sample of the fabric that is cut from the manufactured roll. The sample is deemed representative of the roll and chosen accordingly; the number of specimens is specified in the methodology of the laboratory test itself. Material



6.3 Material properties of geotextiles (adapted from Fannin, 2000).

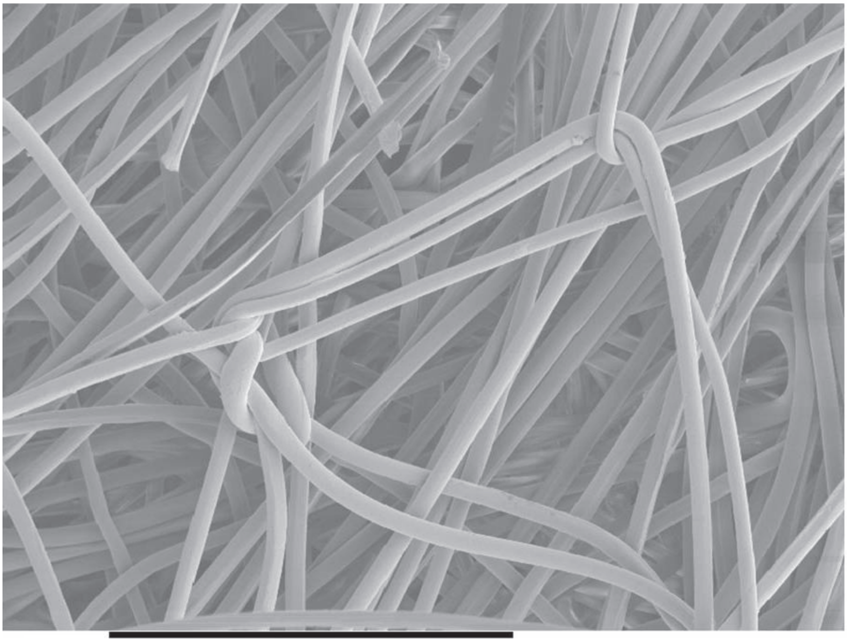
properties are commonly established with reference to a standard test method. The development and publication of such test methods are usually coordinated by governmental or intra-governmental standards agencies, following experimental studies in specialist laboratories. Standard test methods developed at the national level often vary between countries. The work of the International Organization for Standardization (ISO) and the European Committee for Standardization (Comité Européen de Normalisation (CEN)) seeks to harmonize those differences at the international level.

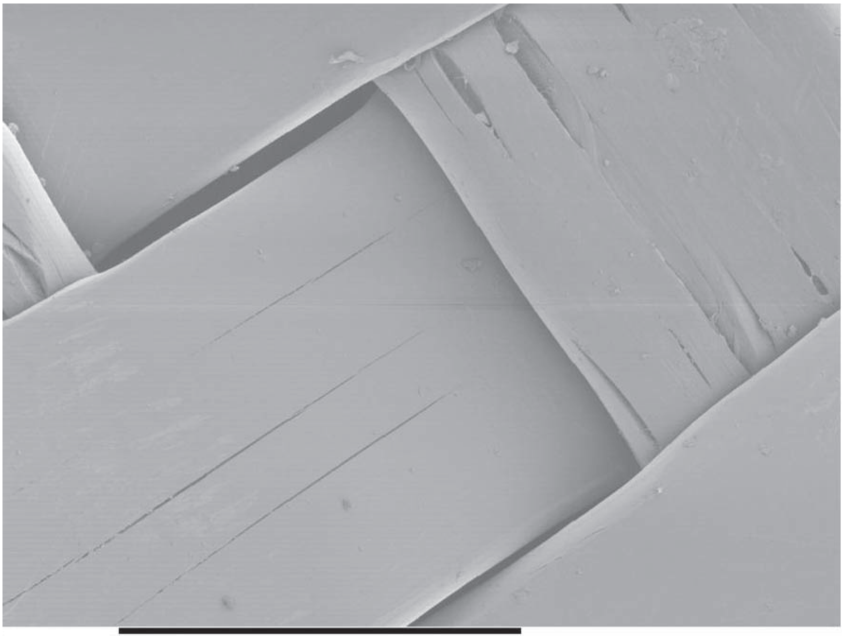
In considering the intent of a laboratory test, it is convenient to distinguish between an index test and a performance test (Fig. 6.3). Index tests are used to establish material properties that are specific to the geotextile alone, with little or no direct consideration given to the relevance of the laboratory test conditions to those of the field application. Examples include the characteristic pore size opening, cross-plane permeability, strength and UV resistance of the geotextile. In contrast, performance tests are used to establish material properties of the geotextile that do take into consideration aspects of the field application, often with reference to soil–geotexile interaction. Examples include that of permeameter testing of soil and geotextile to examine the potential for piping or clogging under the influence of seepage over a designated range of hydraulic gradient, and that of direct shearbox testing to determine the interface strength between a geotextile and soil over a designated range of vertical effective stress.

6.3.1 Index tests

Pore size opening

Geotextiles exhibit a distribution of pore size openings, with the variation in size





6.4 Pore-size openings of (a) a needle-punched non-woven geotextile: (b) a woven geotextile.

being largely determined by attributes of the polymer strand and the manufacturing process. In contrast to a non-woven geotextile, see Fig. 6.4(a), which has a wide range of opening sizes, a woven geotextile tends to have narrow range of relatively larger openings, see Fig. 6.4(b). A characteristic opening size of the fabric is established through indirect means, typically by sieving a gradation of glass ballotini or sand through a specimen of the geotextile, and subsequent determination of the grain size distribution curve of the fraction that passes through the fabric under a prescribed disturbance. The disturbing action typically involves either dry shaking or hydrodynamic flushing. A characteristic opening size, e.g. O_{95} (μm), is taken to be the equivalent grain size of the fraction passing, in this case D_{95}, with the implicit understanding that 95% of the pore openings are less than or equal to this value.

Cross-plane permeability

Geotextiles exhibit a relatively wide range of volumetric flow rate per unit area across the plane of the fabric, with discharge capacity again being largely determined by attributes of the polymer strand and the manufacturing process. The geotextile is mounted in a permeameter, and subject to flow under the influence of either a constant differential head or a falling head. A calculation is typically made of the normal permeability k_n (cm/s), which may also be reported as a value of permittivity ψ (s^{-1}) if divided by the thickness of the fabric.

Strength

Geotextiles are thermoviscoelastic materials, which cause the relation between load (or stress) and deformation (or strain) to be governed by ambient temperature and imposed rate of displacement. Accordingly, geotextiles do not exhibit a unique strength in testing. Strength is generally established from testing of a specimen that is subject to either axisymmetric loading, or a loading condition that approximates plane strain. The imposed rate of displacement can vary significantly between types of test. Examples of axisymmetric loading include a burst or puncture test, while an example of the latter is tensile loading of a relatively wide specimen in either the machine or the cross-machine direction. Axisymmetric loading yields a value of strength (N), and plane strain loading a value of strength per unit width (kN/m).

Ultraviolet resistance

Geotextiles are made from polymers, all of which degrade in the presence of sustained exposure to UV light, yielding a loss of strength. The impact of UV exposure is mitigated through the addition, during manufacture, of stabilizing agents to the polymer of the geotextile. Resistance is characterized in laboratory testing by a measurement of percentage strength loss that occurs in the geotextile following a prescribed period of exposure to a UV light source. The loss (%) is reported over the duration of exposure time.

6.3.2 Performance tests

Piping or clogging behaviour

Filtration compatibility is predicated on the geotextile satisfying a requirement for soil retention. Incompatibility may take the form of unacceptable piping or clogging. Piping refers to particle migration through the geotextile, whereas clogging is a result of entrapment of particles within the geotextile. With reference to the permeability of the soil that is retained, piping yields a zone of relatively high permeability adjacent to the geotextile while, in contrast, clogging generates a zone of relatively low permeability. Compatibility may therefore be evaluated by placing the soil and geotextile in a permeameter, imposing a prescribed seepage regime, and monitoring any change in the permeability of the soil–geotextile interface relative to that of the undisturbed soil. Interpretation of the results involves comparison of observed change against a threshold value of acceptability.

Interface strength

A geotextile invokes strength at the interface with a soil through mobilization of a shear resistance that is largely controlled by friction. The available strength is therefore governed both by the type of soil and by attributes of the fabric that are dependent on the type of polymer, strand and manufacturing process. Interface strength is commonly established in direct shearbox testing, for a prescribed range of vertical effective stress. In cohesionless soils, the efficiency of the bond with the geotextile is expressed as a ratio of interface strength, tan δ, to the angle of shearing resistance of the soil, tan φ.

6.4 Design criteria

Compatibility of a geotextile filter and the soil against which it is placed is governed by the three principal requirements of soil retention, cross-plane permeability and strength. Many generalized design criteria have been proposed for each these requirements, all of which have a common basis.

6.4.1 Soil retention

For retention of the soil, the maximum value of the characteristic opening size of the geotextile is limited by the larger grains of soil against which it is placed, where

$$O_n \leq C_1 D_n \tag{6.1}$$

and C_1 is a dimensionless constant that may be used to differentiate between soil type and style of geotextile. Given the empirical basis of the relation, it is important to account for the test method used to establish the characteristic pore size opening since different test methods have been noted to yield a variation in opening size for the same geotextile (Bhatia *et al.*, 1996). Many design criteria have been proposed for soil retention, most of which relate to unidirectional flow in coarse-grained soils, and some of which are appropriate to fine-grained soils (Table 6.1). The geotextile is typically characterized by a relatively coarse opening size O_{90} or O_{95} and the soil by a relatively large grain size D_{50} or D_{85}, with distinction frequently made between a woven and non-woven geotextile, and recognition occasionally given to the shape of the grain size distribution of the soil. A comprehensive framework has been proposed by Mylnarek (2000), which considers both coarse- and fine-grained soils, the phenomena of dispersion and internal instability in those soils, and the concept of both a minimum and a maximum opening size for the geotextile. Further, it recommends performance testing of some geotextile–soil combinations in order to evaluate filtration compatibility (Fig. 6.5).

6.4.2 Cross-plane permeability

For cross-plane permeability, the capacity of the geotextile must meet or exceed the cross-plane permeability (k_s) of the soil , where

$$k_n \geq C_2 k_s \tag{6.2}$$

C_2 is a constant that may be used to differentiate between soil type and severity of the seepage condition, and k_n is established from index testing of the geotextile. Typical hydraulic gradients may vary from about 1.0 in a trench drain to more than 10 in shoreline protection works (Giroud, 1996) leading to suggested values for the constant that vary between 10 and 100. If a performance test, such as the ASTM D5101-01 gradient ratio test (ASTM International, 2005) is conducted for filtration compatibility it will yield, in addition to a qualitative measure of soil retention, a quantitative measure of relative soil–geotextile permeability which is independent of that based on index testing of material properties.

6.4.3 Strength

For strength, it is customary to ensure that the available strength, established from one or more index tests, exceeds required values that may be either established from a site-specific analysis of anticipated loading or, as more usually occurs, defined by minimum values that are believed appropriate to the field application

Table 6.1 Soil retention criteria for geotextiles[a] (modified from Palmeira and Fannin, 2002)

Source	Criterion	Remarks
Calhoun (1972)	$O_{95}/D_{85} \leq 1$	Woven geotextiles, soils with ≤50% passing through sieve no. 200 sieve
	$O_{95} \leq 0.2$ mm	Woven geotextiles, cohesive soils
Ragutzki (1973) (from Faure, 1988)	$O_f \leq 0.5D_{50}$–$0.7D_{50}$	Woven geotextiles and non-woven geotextiles, dynamic or reversing flow, unconfined soil
	$O_f \leq 0.5D_{50}$–$1.3D_{50}$	Woven geotextiles, dynamic or reversing flow, confined soil
	$O_f \leq 0.5D_{50}$–$1.5D_{50}$	Non-woven geotextiles, dynamic or reversing flow, confined soil
Zitscher (1974) (from Rankilor, 1981)	$O_{50}/D_{50} \leq 1.7$–2.7	Woven geotextiles, soils with $C_U \leq 2$, $D_{50} = 0.1$– 0.2 mm
	$O_{50}/D_{50} \leq 2.5$–3.7	Non-woven geotextiles, cohesive soils
Ogink (1975)	$O_{90}/D_{90} \leq 1$	Woven geotextiles
	$O_{95}/D_{85} \leq 1.8$	Non-woven geotextiles
	$O_f \leq D_{85}$ (from Faure, 1988)	Dynamic or reversing flow, woven geotextiles and non-woven geotextiles, with formation of a natural filter
	$O_f \leq D_{15}$ (from Faure, 1988)	Dynamic/reversing flow, woven geotextiles and non-woven geotextiles, without the formation of a natural filter
US Army Corps of Engineers (1977)	0.149 mm $\leq O_{95} \leq$ 0.211 mm	$D_{50} > 0.074$ mm
	0.149 mm $\leq O_{95} \leq D_{85}$	$D_{50} \leq 0.074$ mm
		Geotextiles should not be used if $D_{85} < 0.074$ mm
Schober and Teindl (1979)	$O_{90}/D_{50} \leq 2.5$–4.5	Woven and thin non-woven geotextiles, dependent on C_U
	$O_{90}/D_{50} \leq 4.5$–7.5	Thick non-woven geotextiles, dependent on C_U, silts and sands
Millar *et al.* (1980)	$O_{50}/D_{85} \leq 1$	Woven geotextiles and non-woven geotextiles

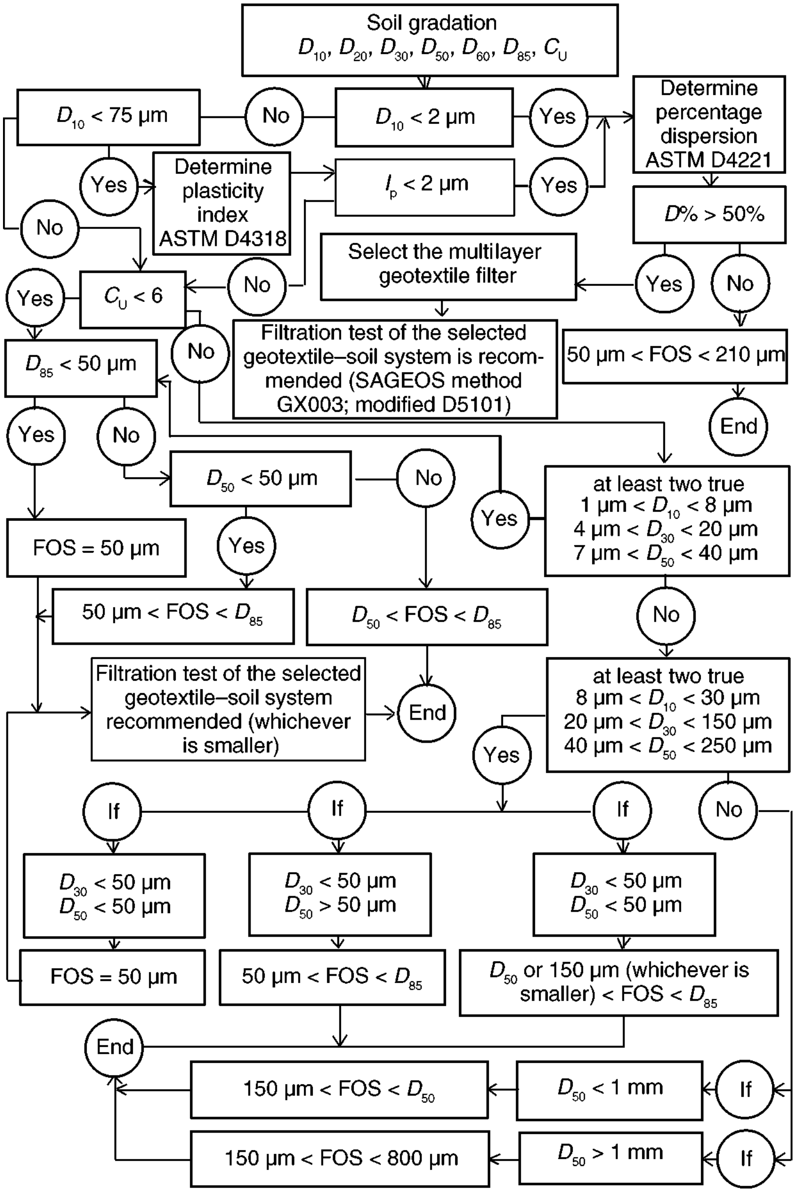
Rankilor (1981)	$O_{50}/D_{85} \le 1$	Nonwoven geotextiles, soils with 0.02 mm $\le D_{85} \le$ 0.25 mm
	$O_{15}/D_{15} \le 1$	Nonwoven geotextiles, soils with $D_{85} >$ 0.25 mm
Giroud (1982)	$O_{95}/D_{50} < C'_U$	$I_D <$ 35%, $1 < C'_U < 3$
	$O_{95}/D_{50} < 9/C'_U$	$I_D <$ 35%, $C'_U > 3$
	$O_{95}/D_{50} < 1.5C'_U$	35% $< I_D <$ 65%, $1 < C'_U < 3$
	$O_{95}/D_{50} < 13.5/C'_U$	35% $< I_D <$ 65%, $C'_U > 3$
	$O_{95}/D_{50} < 2C'_U$	$I_D >$ 65%, $1 < C'_U < 3$
	$O_{95}/D_{50} < 18/C'_U$	$I_D >$ 65%, $C'_U > 3$
		Assumes fines in soil migrating for large C_U
Heerten (1982)	$O_{90} < 10D_{50}$ and $O_{90} \le D_{90}$	Cohesionless soils, with $C_U > 5$ and static load conditions
	$O_{90} < 2.5D_{50}$ and $O_{90} \le D_{90}$	Cohesionless soils, with $C_U < 5$ and static load conditions
	$O_{90} < 10D_{50}$ and $O_{90} \le D_{50}$	Cohesionless soils, dynamic load conditions
	and $O_{90} \le 0.1$ mm	Cohesive soils and all load conditions
Carroll (1983)	$O_{95}/D_{85} < 2–3$	Woven and non-woven geotextiles
Christopher and Holtz (1985)	$O_{95}/D_{85} \le 1–2$	Dependent on soil type and C_U
	$O_{95}/D_{15} \le 1$ or $O_{50}/D_{85} \le 0.5$	Dynamic, pulsating and cyclic flow, if soil can move beneath geotextile
Mlynarek (1985), Mlynarek *et al.* (1990)	$2D_{15} < O_{95} < 2D_{85}$	Non-woven geotextiles
Comité Français Géosynthétiques (1986)	$O_f/D_{85} \le 0.38–1.25$	Dependent on soil type, compaction, hydraulic and application conditions
	$O_f \le 0.5D_{85}$ (from Faure, 1988)	Reversing flow, woven geotextiles and non-woven geotextiles, loose soil
	$O_f \le 0.75D_{85}$ from Faure, 1988)	Reversing flow, woven geotextiles and non-woven geotextiles, dense soil
Lawson (1986)	$O_{90}/D_n = C$	Developed for residual soils from Hong Kong Values of *n* and *C* are obtained by a chart defining regions of acceptable filter performance

Table 6.1 (*cont.*)

Source	Criterion	Remarks
Lawson (1987) (from Geotechnical Engineering Office , 1993)	$O_{90}/D_{85} \leq 1$	For predominantly granular soils with $D > 0.1$ mm, e.g. residual soils which are granular in nature and alluvial sandy soils
	$0.08 \text{ mm} \leq O_{90} < 0.12 \text{ mm}$	For non-cohesive soils, e.g. silts of alluvial or other origin, and for non-dispersive cohesive soils
	$0.03 \text{ mm} \leq O_{90} \leq D_{85}$	For dispersive cohesive soils
John (1987)	$O_{95}/D_{50} \leq (C'_U)^a$	a is dependent on the size of the particle to be restrained ($a = 0.7$ for D_{85})
Fischer *et al.* (1990)	$O_{50}/D_{85} \leq 0.8$ $O_{95}/D_{15} \leq 1.8–7.0$ $O_{50}/D_{50} \leq 0.8–2.0$	Based on geotextile pore size distribution, dependent on C_U of soil
Rollin *et al*. (1988)	$O_{95} < 1D_{85}–1.5D_{85}$	Tests with a fine sandy soil and three non-woven needle-punched geotextiles using an upflow filtration apparatus
Luettich *et al*. (1992)	Design charts	Based on geotextile void size and type, hydraulic conditions and other factors
Ontario Ministry of Transportation (1992)	$O_f/D_{85} < 1.0$ and $O_f > 0.5D_{85}$ or 40 μm	Non-woven geotextiles preferred, $t_{GT} > 1$ mm; avoid thermally bonded geotextiles
Murray and McGown (1992) (from Corbet, 1993)	$O_{90}/D_{90} = 1–3$	Soils with $1 \leq C_U < 5$, woven and non-woven geotextiles
	$O_{90}/D_{90} < 1–3$	Soils with $5 < C_U < 10$, woven and thin non-woven geotextiles ($t_{GT} \leq 2$ mm); alternative criterion
	$O_{90}/D_{50} < 1.8–6$	Soils with $5 < C_U < 10$, thick non-woven geotextiles ($t_{GT} > 2$ mm); alternative criterion

Bhatia and Huang (1995)	$O_{95}/D_{85} < 0.65–(0.05\ C_c)$ $O_{95}/D_{85} < 2.71–(0.36\ C_c)$ $O_{95} < D_{85}$	$n < 60\%$ and $C_C > 7$ $n < 60\%$ and $C_C < 7$ $n < 60\%$
Lafleur (1999)	$O_f/D_I < 1$	Stable soils ($C_U \leq 6$ and $D_I = D_{85}$ in this case), soils with $C_U > 6$ but linearly graded ($D_I = D_{50}$ in this case), gap-graded ($C_U > 6$) internally stable soils ($D_I = D_G$) and soils with $C_U > 6$ with gradation curve concave upwards and internally stable ($D_I = D_{30}$)
	$1 < O_f/D_I < 5$	Unstable soils with $D_I = D_{30}$ for gap graded internally unstable soils and for internally unstable soils with gradation curves concave upwards (risk of piping of fines) Criteria developed for cohesionless soils
Mylnarek (2000)	$A < O_f < B$ (from charts)	Values of *A* and *B* established with reference to grain size, coefficient of uniformity, plasticity index and dispersion potential of the soil, and for seepage flow that is either unidirectional or bi-directional (dynamic)

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



6.5 Design criteria for soil retention in unidirectional flow (after Mylnarek, 2000): FOS, filtration opening size.

based on experience rather than analysis. Additionally, it is usual to ensure that the UV resistance of the geotextile from index testing is greater than a minimum value which again is based entirely on experience, rather than analysis.

6.4.4 Commentary

The design criteria for soil retention, cross-plane permeability and strength are empirical in origin and, unless specifically noted, are considered appropriate to filtration applications where the following hold.

1 Seepage flow across the geotextile is predominantly low and unidirectional.
2 Confining stress on the geotextile is essentially constant and subject to little change.
3 There is consideration given to quality assurance, through site supervision, during placement of the geotextile.

These conditions are commonly encountered in earthworks associated with slopes, retaining walls, shallow foundations and sports fields. Confidence in the design criteria is founded on a considerable body of laboratory experience from testing that simulates these relatively simple field conditions. In contrast, the occurrence of reversing or pulsating flow and cyclic or vibration loading of the ground produce more complex conditions that are not easily reproduced in simple laboratory tests. Accordingly, in these more severe applications, it is recommended that design criteria for soil retention and cross-plane permeability (see for example Holtz *et al.*, 1997 and Mylnarek, 2000) be applied with caution rather than confidence, and with recourse to performance tests to support a site-specific evaluation of filtration compatibility (Fannin and Pishe, 2001).

6.5 Specification of materials

Design criteria are used to identify suitable material properties of a geotextile. More specifically, they are used to establish a maximum opening size, a minimum value for cross-plane permeability, and minimum values for strength of the material. They are required values of material properties for design, based on index tests, and are commonly reported in a construction specification document, together with additional guidance on required provisions for placement of the geotextile on site. On occasion it may be necessary to include additional reference to performance tests, to ensure that a comprehensive statement is provided on the compatibility requirements of a proposed filtration application.

The specification document enables comparison of required material properties, established from the design process, with available material properties, based on the range of geotextile products that can be delivered to site through contact with commercial suppliers. Accordingly, one intended use of the specification document is to identify a candidate product that satisfies design criteria for the application, in the expectation that this will provide for compatibility over the service life of the geotextile filter.

Two approaches may be adopted in the writing of a specification document, with the choice of approach being largely determined by the scope of the construction application. Consider an application that involves, for example, challenging soils or an unusual flow regime, for which project-specific design criteria have been established. The criteria are typically established with reference to data from a site investigation and may account for special features of the proposed construction. Required material properties are therefore specific to that site, yielding the need for a specification document that is unique to the proposed application, and one that should enable selection of a candidate geotextile that is expected to be fully compatible with the project requirements.

In contrast, consider an application that is, in all respects, a small and routine example of filtration practice for which no detailed site investigation is warranted. From a cost–benefit perspective there is no compelling need to develop a project-specific or unique specification document and, therefore, recourse can legitimately be made to a standard specification. Filtration applications in subsurface drainage and permanent erosion control, as a part of earthworks for road construction, represent good examples of such relatively small and routine provisions. The standard specification document may be used to establish default values for the opening size, cross-plane permeability and strength of the geotextile (see for example AASHTO M 288-00 (American Association of State Highway and Transportation Officials, 2000)). The standard specification therefore acts as a guide to the selection of a candidate geotextile that is expected to be compatible with the project requirements.

Accordingly, the choice in specifying material properties of a geotextile lies between that of a site-specific or unique specification document and that of a generic or standard specification document. The merits of each approach are largely determined by the scope of the construction project, and the nature of the soils and groundwater flow regime.

6.6 Construction considerations

Geotextiles are products manufactured to designated ranges of specific material properties, with commensurate attention given to quality assurance on the rolls that are supplied to site. Each roll identification tag, with product name and roll number, should allow for a direct cross-reference to quality control testing of the production lot by the manufacturer. Where appropriate, e.g. in permanent earthworks associated with significant capital costs or potential for loss of life in the event of failure, the specification document may be written to require submission of a manufacturer's certificate of compliance: the certificate declares the date and location of manufacture and affirms both product style and relevant material properties. The specification document may also be written to allow for independent testing of samples taken from geotextile rolls supplied to site, at the discretion of the engineer, in order to verify the material properties claimed by the manufacturer.

6.6.1 Site delivery and field inspection

The geotextile roll is covered with an outer wrapping. It is important that the roll is protected during shipping and storage, and that there is no prolonged exposure at the time of installation on site. During placement, rolls should not be dragged, and construction equipment should not operate directly on the surface of the geotextile, as this may result in material damage. Placement procedures should be reviewed, including provisions for ground clearing and grading, subgrade preparation, overlaps or seams between rolls, cover soils and construction equipment.

6.6.2 Placement of the geotextile

A check should be made to ensure that the direction in which the roll is deployed is consistent with that specified for installation and to confirm that the method of placement on site complies with that approved for construction. Filtration compatibility between soil and geotextile relies on an intimate contact between each material, which thereby limits the potential for development of preferential flow and localized erosion. Accordingly, ground protrusions and depressions should be trimmed to yield a smooth and flat surface. Where adjacent rolls overlap, a lap distance of 0.3 to 0.5 m should be provided for, or more if the ground is likely to deform, and about 0.1 m if the overlap is to be sewn. Sewn seams should exhibit strengths that are similar to those of the geotextile itself. Provisions for repair of any damage, including patching or replacement, should be described in the specification document.

6.7 Sources of further information

Additional reading on the use of geotextiles in filtration applications may, for purposes of convenience, be considered in two general categories: firstly, construction guidance that is focused on basic requirements governing routine design practice and, secondly, a series of specialist conference proceedings on applied research to improve upon knowledge of filtration compatibility and on case study reports of performance monitoring in construction projects.

Routine design issues have been addressed in several documents, of which two deserve specific mention. The US Federal Highway Administration (Christopher and Holtz, 1985) first published a comprehensive review of filtration principles with reference to supporting research studies. The Geotechnical Engineering Office of Hong Kong (1993) subsequently prepared a review of principles and practices related to the design of filters for use in earthworks, with additional commentary on construction methods: Part III of this publication addresses geotextile filters, with reference to types of geotextile, durability issues, hydraulic properties, long-term behaviour, a review of design criteria and specific guidance for the soils of Hong Kong. It provides a very useful contribution to design practice.

A series of international conferences has been organized on the use of geofilters, with specific emphasis on applications of filters and drainage in geotechnical and geoenvironmental engineering. It commenced with Geofilters '92 (Karlsruhe, Germany) and continued with Geofilters '96 (Montreal, Canada), Geofilters 2000 (Warsaw, Poland) and Geofilters '04 (Stellenbosch, South Africa). The intent of the conference series is to disseminate the findings of advanced studies on both granular and geotextile filters, thereby defining the state of the art in analysis and design. The technical sessions address topics that include theoretical developments, laboratory testing, design criteria, long-term behaviour, waste disposal, landfill drainage, and hydraulic structures. The conference proceedings contain special or keynote lectures on a comparison of granular and geotextiles (Giroud, 1996), analytical modeling and experimental verification of filter behaviour (Indraratna and Locke, 2000), design criteria (Mylnarek, 2000), and filtering and drainage of contaminated water (Rowe and VanGulck, 2004).

6.8 Future trends

At the time of writing, it is reasonable to conclude that the behaviour of geotextile filters in earthworks subject to unidirectional flow of groundwater seepage through soils is reasonably well understood and, consequently, the companion design criteria may be used with confidence. The confidence is predicated on a thorough understanding of the physical processes that govern compatibility. The design criteria are wholly empirical and, importantly, assume that the soils are internally stable.

In contrast, there is limited experience with soils that may be problematic, such as internally unstable gradations (including gap-graded soils). Internal instability refers to a potential for seepage-induced migration of the finer fraction of a soil gradation, where the grain size distribution curve exceeds a limiting geometric constraint and the seepage flow exceeds a limiting hydromechanical constraint. It is likely that, with an improved understanding of such problematic soils, the use of geotextile filters will grow to include design recommendations for these soils that can be used with equal confidence.

Likewise, a good understanding is currently emerging of the operating conditions in filters in landfills that are subject to unidirectional seepage of leachate flow. The challenge, in this application, is one of accounting for the elevated temperatures that are present in the filter, which may approach 50–60 °C, and the ongoing chemical and biological processes, which tend to promote the growth of biomass and precipitation of deposits on the surface of the filter medium. These influences also combine to generate a significant amount of suspended soils in the seepage flow, with a commensurate potential for clogging of the filter openings. Although laboratory studies have sought to replicate the behaviour of these systems, performance monitoring and forensic observations from waste containment facilities suggest that the spatial and temporal variations in seepage flow are often considerable. Accordingly, with the trend toward more reporting of experience gained in the operation of such facilities, it is expected that tentative design recommendations will be refined and used with increasing confidence.

In contrast with unidirectional seepage flow in routine filter applications, where there is a long-standing body of knowledge on use of geotextiles that is based on considerable field experience and many laboratory studies, the issue of bidirectional or reversing flow is one for which our current understanding is very much limited. This may be attributed to several factors, including the nature and occurrence of reversing flow in routine engineering works, and corresponding lack of good documented field experience, coupled with a paucity of laboratory studies that address the specifics of such flow regimes. Yet considerable challenges exist in the confident provision of filters for protection of civil infrastructure in estuarine and coastal environments, where a subtle distinction can be made between slow reversing flow, such as that of tidal environments, and the relatively faster reversing flow that occurs in the presence of wave action. Future trends will probably include a greater emphasis on building upon the existing confidence in use of geotextiles for routine applications of unidirectional flow, in order to develop a similar confidence in seepage applications that include reversals in the direction of flow.

6.9 References

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7
The use of geosynthetics as separators in civil engineering

W. WILMERS
Germany

7.1 Introduction

7.1.1 The separation layer between coarse granular soil and fine-grained soft soil

Separation is filtration under load. In the case of a granular layer over a fine-grained soft soil, the separator must prevent the fine particles of soft soil from entering the gaps between the particles of granular material above it, as well as preventing the larger grains of the granular layer from sinking into the soft soil below. The separator must also permit water to pass through to prevent pore water overpressure in the soft soil, and all this must function under load. The simplest example is shown in Fig. 7.1. The boot and the crushed stones do not penetrate the soft soil, but the water, pressed out under load, rises up.

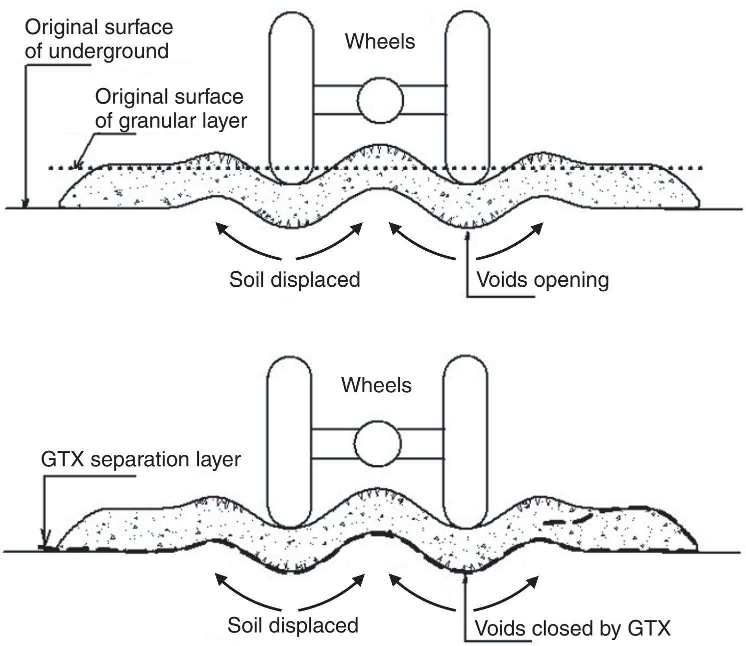
The physical demands for a separator under an access road are more complex. When vehicles pass along the road, the granular layer is pressed down by the wheels and deformed in accordance with the shape and load of the wheels. This deformation widens the gaps between the granular particles, which permits the finer soil particles to penetrate the granular layer (Fig. 7.2). Successive transits by vehicles increase the amount of fine particles in the granular layer and the coarse aggregates start to sink into the fine soil. Eventually, the granular layer collapses and the road becomes impassable.

The separator must therefore have the following characteristics.

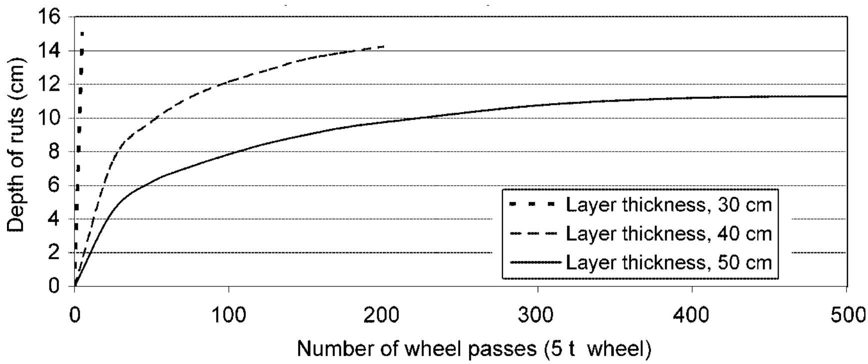
1 It must follow the deformation under rolling loads.
2 It must have a high elongation, to allow rutting without the layer rupturing.
3 It must possess sufficient strength to prevent a local collapse.
4 It must be robust enough to withstand mechanical stresses during installation and under traffic.

Separation is distinct from reinforcement as the latter aims to reduce the depth of ruts through tensile strength.

7.1 Non-woven geotextile over soft soil as a separator for crushed stones and a worker's boot.



7.2 Granular layer under a wheel load, without and with a separation layer.



7.3 Development of ruts with a separation layer between the bearing layer and soft soil and with different thicknesses of bearing layer. Where the depth of the ruts (normalized for $I_c = 1$) is plotted against number of wheel passes over layers of crushed stone, 0–56 mm, over silt with $0.5 < I_c < 1.0$ (Brau *et al.*, 1987).

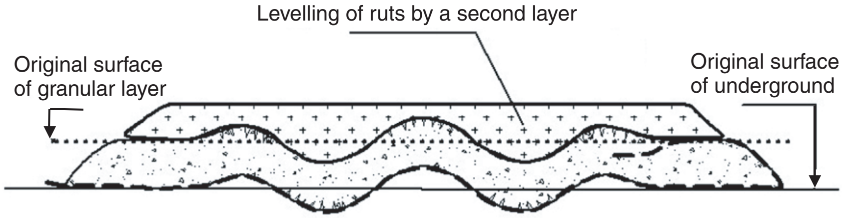
7.1.2 Bearing capacity

The bearing capacity in this situation relates to the number of wheel passes for a 'layer–separator–soft soil' system and depends on the thickness and inner friction of the granular layer and the deformability and inner friction of the underlying soft soil. The granular layer should be a well-graded granular material and should be strongly compacted, to obtain high friction between the grains and maximum stiffness. The only function of the separator is to preserve the inner friction of the granular layer, but it also helps to consolidate the underlying soft soil. Under wheel loads, the pores of the fine soil are subjected to pressure and overpressure can build up in the water that they contain. When the water is allowed to rise up through the separator into the granular layer, the fine layer consolidates and this improves the bearing capacity. This explains why separators often limit the development of deep ruts (Fig. 7.3). The first layer over a separation layer must therefore be water permeable and the grains must be resistant against weathering.

However, consolidation takes time and, when the soft soil cannot consolidate, e.g. when the water permeability is too low or when there is not enough time between vehicle passes, then rut development is faster. In this case, a separation layer with high elongation that can adjust to the ruts without rupturing is needed, or the depth of the ruts should be kept to a minimum through reinforcement.

7.1.3 Levelling of surfaces with ruts

Rutting not only deepens the surface of the bearing layer but also compresses the soil layer between and on the sides of the ruts, moving it vertically and horizontally (Fig. 7.2). When the ruts become too deep for vehicles to pass, the surface



7.4 Levelling of ruts.

must be levelled. This is achieved by filling the ruts and compacting the fill (Fig. 7.4). If the levelling is achieved by grading, the layer is thinner at the higher parts between and beside the ruts and the bearing capacity of the system is partially reduced.

7.2 Applications

7.2.1 Unpaved roads and trafficked areas

Access roads

The main application is as a separation layer in access roads. In most cases, these are temporary roads, used by heavy and very heavy vehicles for a limited period. Usage can vary from heavy to none at all, and the road may be dismantled once it is no longer needed. Therefore it is important to minimize the quantity of material used, and so the thickness of the bearing layer is kept as thin as possible. Deformation is permitted, but not to the level of collapse. Access roads are typically narrow and vehicles use the same path in both directions, so rutting is inevitable (Fig. 7.2). To optimize the thickness of the layer for the given traffic load, site tests are needed because an estimation of the reaction of the subsoil and the influence of the friction characteristics of the fill is not realistically possible. Figure 7.6, shown later, gives an example for sandy gravel with round grains.

When the access road is to be used for only a short time and the materials are to be recovered, geotextiles made from natural fibres are preferable, because they rot in landfills. The separation layer is helpful in restoring the native soil beneath because, after aggregates are recovered, it can be removed together with any residual particles.

The choice of geotextile separation layer should take into consideration the grain size of the fill and the expected depth of ruts, and it must demonstrate the required filter characteristics (see Section 7.3).

Forest and agricultural roads

Forest roads or roads in agricultural areas are typically unpaved. Vehicles of different sizes and wheel loads use them, but traffic is limited to certain periods and not as frequent as for access roads. Rutting is allowed, but not too much, because otherwise maintenance demands for the road become too high. Under these conditions, consolidation of the subsoil over time occurs more easily and helps to ameliorate the bearing capacity.

The choice of geotextile for this type of separation layer should take into account the grain size of the fill and the expected depth of ruts, as well as the required filter characteristics (see Section 7.3). Geotextiles with high durability should be chosen when the expected lifetime of the road is long; these will generally be those made from synthetic fibres with proven durability.

Work platforms

Work platforms are sometimes used temporarily, like access roads, and sometimes consist of the first layer of an embankment over compressible soils. However, there is an important difference here because work platforms must not deform or yield, especially when they are used for machines with high towers, such as pile boring machines. The separation layer in this case does not improve the bearing capacity of the system, but it does help to hinder deterioration. The strength is provided by the material and thickness of the fill, taking into account the stiffness and deformability of the subsoil.

7.2.2 Paved roads and trafficked areas

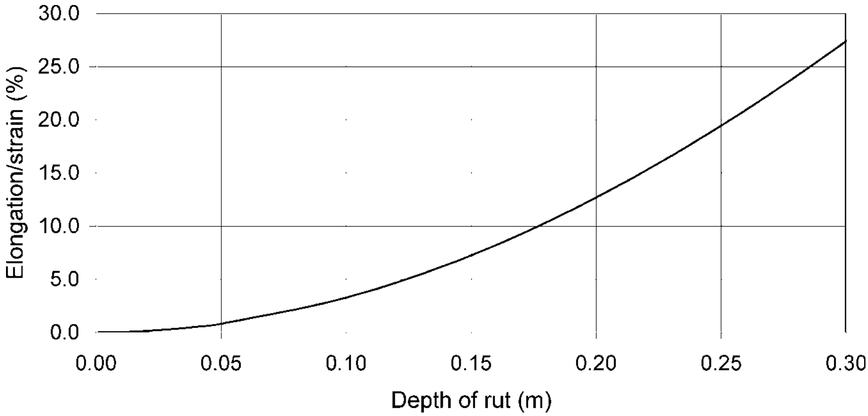
Geosynthetics in road bases

The permitted elastic deformation for subgrades under road bases and superstructures is less than 0.5 mm. If the soil is not strong enough, geosynthetics cannot guarantee a sufficient strength, because geosynthetics require deformation in their plane to develop a reaction force. The vertical deformation of a rut, necessary for an elongation of only 1.0% in a textile on the base of a layer of 300 mm thickness, is more than 50 mm for a wheel rut of 300 mm width (Fig. 7.5).

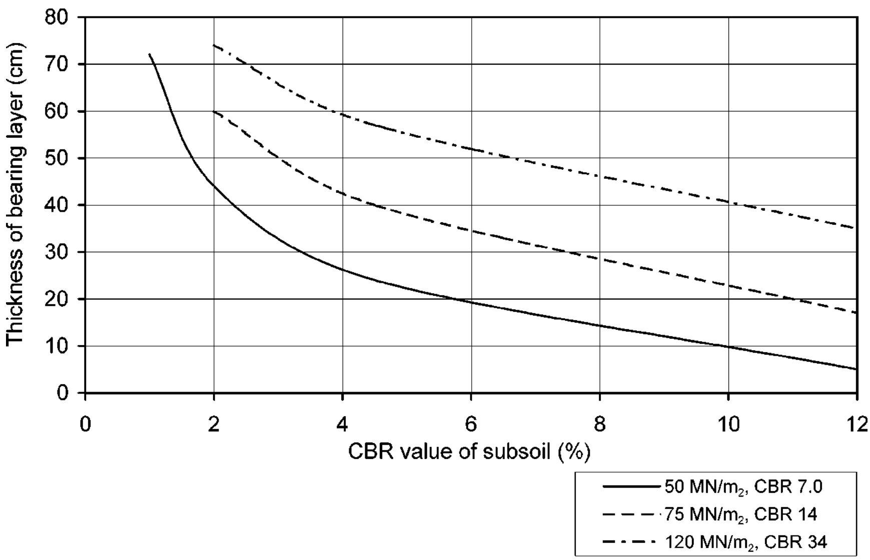
Separation layers cannot therefore improve the strength of a subgrade, but they are very helpful under a granular layer used to improve the bearing capacity of the subgrade (by excavating soft soil and installing granular soil of higher bearing capacity), because they guarantee that the granular material keeps its properties even when disturbed by construction traffic. The bearing capacity is given by the thickness of the bearing layer. Fig. 7.6 shows the effect of layer thickness for sandy gravel over soft soil with different strengths (see also Fig. 7.3).

Textiles under cement concrete pavements

Concrete pavements are often laid on a bound base. This can be a coarse base of



7.5 Depth of ruts and elongation of fabric under a layer of 300 mm thickness for ruts of size 300 mm.



7.6 Thickness of a layer of sandy gravel over soft soil to reach a stiffness Ev2 (MN/m^2), measured by a static plate bearing test, in relation to the stiffness of subsoil measured as the CBR value (Voss, 1961).

cement concrete or of asphalt concrete. Water can enter into the horizontal crack between the base and the pavement from the side or through joints. The impact of wheel loads squeezes the water aside and the thin water layer can reach very high velocities, sufficient to corrode the upper surface of the bound base, and in some cases also the lower surface of the pavement. This corrosion begins near the transverse joints, because under the arris the vertical movement is higher than in the middle, and also when there are dowels. After partial corrosion of the bedding, the concrete pavement can rise and break; therefore nowadays, in Germany, non-woven textiles are usually placed under new concrete pavements.

The functions of the textile layer are as follows (Sulten and Wilmers, 2000).

1 *Separation.* The fabric creates a clear division by separating the concrete pavement from the base; secondary bending strains are avoided and internal strains are reduced.
2 *Drainage.* Water is discharged to the sides.
3 *Bedding*. The fabric creates even bearing conditions for the concrete surface and functions similarly to elastic bedding; dynamic traffic loads are cushioned and absorbed.

German regulations for concrete pavements include the following requirements, according to ZTV Beton-StB (Forschungsgesellschaft für Strassen- und Verkehrswesen, 2001).

1 Non-woven textile, alkali resistant (polypropylene or polyethylene).
2 Thickness d, measured under 20 kPa load: $d > 2$ mm.
3 Mass ma per unit area: 450 g/m² < ma < 550 g/m².
4 Tensile strength T_f in the machine direction and cross machine direction: $T_f >$ 10 kN/m.
5 Elongation ε_t at tensile strength in the machine direction and cross machine direction: $\varepsilon_t < 130\%$.
6 Water permeability k_H in the plane under 20 kPa load and hydraulic gradient $i = 1$: $k_H > 5 \times 10^{-4}$ m/s.
7 Water permeability k_V vertical to the plane under 20 kPa load and hydraulic gradient $i = 1$: $k_V > 1 \times 10^{-4}$ m/s.

Asphalt interlayers

Asphalt interlayers are mostly used in the rehabilitation of roads and pavements subjected to thermal fatigue and reflective cracking, and to reduce the amount of cracking in a new pavement or an asphalt overlay according to prEN 15382 (European Committee for Standardization, 2005). They can have different functions.

1. *Stress relief.* Separation between a structure which is jointed or fissured (concrete pavement, cement bound base, old fissured asphalt) and a new asphalt pavement.
2. *Barrier.* Waterproofing of a fissured layer over a water- or a frost-susceptible base, to be covered by asphalt.
3. *Reinforcement.* Reinforcement of the new asphalt layer over a jointed or fissured structure.

Table 7.1 Asphalt interlayers: function-related characteristics and test methods to be used as condition for CE marking (extract from EN 15382, Table 1)

Characteristic	Test method	Function[a]		
		Reinforcement	Stress relief	Barrier
Tensile strength[b]	EN ISO 10319	H	H	H
Elongation at maximum load	EN ISO 10319	H	H	H
Dynamic perforation resistance[c]	EN 918	H	–	H
Static puncture (CBR)[c]	EN ISO 12236	H	H	H
Resistance to weathering	EN 12224	S	S	S
Bitumen retention	EN 15382, Annex C	–	H	A
Melting point	EN ISO 3146	S	S	S
Alkaline resistance	EN 14030	S	S	S

[a]H, required by the mandate; A, relevant for all conditions of use
S: relevant for some conditions of use.
[b]The wide-width tensile test may not be suitable for specific purposes such as durability assessment or behaviour under cyclic loading. In these cases, more appropriate methods such as EN ISO 13934-1 or ASTM D 6337-01 shall be used.
[c]It should be considered that this test may not be applicable for some types of product, e.g. geogrids. If tensile strength and static puncture are coded H, the producer shall give data for both.

There are different types of product for these tasks, e.g. non-woven geosynthetics, grids and combinations of grids with non-woven geosynthetics. prEN 15382 defines functions and function-related characteristics (see Table 7.1). The fabrics are placed on the surface to be covered and pasted with bitumen enriched with polymers either spread as hot bitumen or as bitumen emulsion. Hot-mixed asphalt pavement is then placed on top. The heat of the asphalt melts the bitumen, which permeates the layer and thus pastes the interlayer to the asphalt pavement.

Particular problems occur in roads over shrinking soils with very low traffic, e.g. roads in arid or semi-arid environments, or paved agricultural roads. The asphalt overlay may be placed as surface dressing over a non-woven interlayer impregnated with bitumen emulsion.

Parking lots

Parking lots are stressed under low-velocity circulation and by vehicles that rest in the same place for a long period of time. Parking lots mostly have bound pavements (asphalt or cement concrete) or concrete stone set paving. In all cases, the construction must be stiff enough to hinder local deformations under load, which can be followed by settling. The separation layer between subsoil and first fill aids during construction, because it prevents deformation and mixing of the

Table 7.2 Separation layer in railroad construction: requirements for geotextiles as filter- and separation elements under bearing layers: German Rails (from EBA, 2003)

Characteristic	Test method	Requirement
Woven geotextiles: tensile strength	EN ISO 10319	> 45 kN/m
Non-woven geotextiles: static puncture	EN ISO 12236	> 2.5 kN
Water permeability normal to the plane	E-DIN 60500-4	$k_V > 1 \times 10^{-3}$ m/s
Characteristic opening size	EN ISO 12956	Related to design
Weather resistance	EN 13249	CE document
Durability	EN 13249	CE document
Compatibility to environment	M Geok E-StB 05	List of parameters

underlying soil. The bearing capacity is not influenced by the separation layer but only by the bearing capacity of the subsoil and the properties and thickness of the fill.

Stone-set paving

In some constructions, a geotextile separates a sandy bed of concrete stones from an open-graded gravel layer below, or a water-permeable bound layer with large pores. This is to stabilize the bedding of stone set paving.

7.2.3 Railroads

A separation layer between ballast and the underlying soil can be very efficient because, under a dynamic load, the coarse grains can be pressed or vibrated into the ground, making maintenance necessary. The very high erosion of coarse sharp-edged grains under the dynamic loads of railway circulation destroys even very thick geotextiles in a short time. Therefore, in railroad tracks with frequent circulation, it is better to install a layer of, for example, sandy gravel under the ballast. This layer is filter stable against the ballast under load and is known as the 'protection layer'. A geotextile separation layer should be placed between the fine-grained soil and the gravel. German Rail has developed special requirements for this purpose (Table 7.2).

7.3 Requirements for geotextiles

7.3.1 Values required by EN 13249 ff

EN 13249 ff (European Committee for Standardization, 2000) defines a list of characteristics of geotextiles that describe the properties necessary for use as a separation layer in road construction and in other construction fields (Table 7.3). This table is identical with that in EN 13250 concerning railroad construction.

Table 7.3 Separation layers: function-related characteristics and test methods to be used as condition for CE marking (extract from EN 13249, Table 1)

Characteristic	Test method	Function, separation[a]
Tensile strength[b]	EN ISO 10319	H
Elongation at maximum load	EN ISO 10319	A
Tensile strength of seams and joints	EN ISO 10321	S
Static puncture (CBR)[b,c]	EN ISO 12236	H
Dynamic perforation resistance[c]	EN 918	A
Friction characteristics	EN ISO 12957-1/-2	S
Damage during installation	EN ISO 10722-1	A
Characteristic opening size	EN ISO 12956	A
Water permeability normal to the plane	EN ISO 11058	A
Durability	EN 13249 Annex B	H
Resistance to chemical ageing degradation	EN ISO 12960, EN ISO 13438, EN 12447	S
Resistance to microbiological	EN 12225	S
Resistance to weathering	EN 12224	A

[a]H, required by the mandate; A, relevant for all conditions of use; S, relevant for some conditions of use.
[b]The mechanical properties of tensile strength and static puncture are coded H in this table; this requires the producer to provide data for both. The use of only one, either tensile strength or static puncture, is sufficient in the specification.
[c]It should be considered that this test may not be applicable for some types of product, e.g. geogrids.

There, the values with an H are harmonized, which means that they are obligatory for the CE label. For values labelled A, the CEN Technical Committee 189, which has developed the standard, states that these values must be given for all cases of use, and an S means that the values are necessary for some cases of use. However, only the values labelled H are given in the CE documentation; so the user must request the others from the producer.

7.3.2 Requirements for asphalt interlayers (prEN 15382)

prEN 15382 (European Committee for Standardization, 2005) defines a list of characteristics to describe the ability of textiles for a given purpose (see Table 7.1). For an explanation of H, A and S see Section 7.3.1.

7.3.3 Requirements for separation layers

Calculation of damaging influences is not realistically possible; therefore, in different countries, individual approaches are developed, on the basis of experiences with particular applications. The following are outlined.

France (AFNOR G 38-063)

According to AFNOR G 38-063 (Association Française de Normalisation, 1993), the subsoil is separated into three classes: soil 1, $c_u > 60$ kPa; soil 2, 20 kPa $< c_u <$ 60 kPa; soil 3, $c_u < 20$ kPa.

The fill is defined by four properties.

1 Permeability in two classes: lower or higher $k_f = 1 \times 10^{-5}$ m/s or 100 times the permeability of subsoil.
2 Particle shape in two classes: sharp-edged or round.
3 Particle size in two classes: greatest particle size smaller or larger than 250 mm.
4 Thickness of first layer in two classes: middle, 0.30–0.50 m; thick, 0.50–1.00 m.

Geotextiles are characterized by the following properties.

1 Tensile strength and tensile strain.
2 Tear resistance.
3 Water permeability vertical to plane.
4 Water permeability in plane.
5 Opening size.

In a screen for different fill conditions, the required properties of geotextile separators are defined.

Germany

For separators in road construction, according to M Geok E-StB 05 (Forschungsgesellschaft für Strassen- und Verkehrswesen, 2005a) and TL Geok E-StB 05 (Forschungsgesellschaft für Strassen- und Verkehrswesen, 2005b) the fill is characterized by the following.

1 Angularity: sharp-edged or round
2 Particle size and grading in four classes
3 Stress during installation and use in five classes, defined by depth of ruts.

In a screen, these factors are combined to give five classes of requirements for the geotextiles. Corresponding to this the robustness of geotextiles is characterized in five classes depending on strength and mass per unit area. The strength of non-woven geotextiles is measured by the static puncture test CBR (EN ISO 12236), of woven geotextiles by the tensile test.

The hydraulic properties are as follows.

1 Permeability: $k_V > 1 \times 10^{-4}$ m/s and $k_V > k_{f\ soil}$.
2 Opening size O_{90}; for non-woven geotextiles, 0.06 mm $< O_{90} <$ 0.20 mm and, for woven geotextiles, 0.06 mm $< O_{90} <$ 0.40 mm.

For separators between protection layers and fine-grained soils in railtracks, German Rail has developed special requirements (Eisenbahn-Bundesamt, 2003) (see Table 7.2).

Scandinavian countries (NorGeoSpec, 2002)

According to NorGeoSpec, 2002 (Stiftelsen for Industriell og Teknisk Forskning, 2002), five specification profiles are based on the following.

1 Subsoil conditions (two classes), defined by shear strength c_u.
2 Construction conditions (two classes), based on construction traffic, angularity of fill material, particle or stone size and layer thickness.
3 Traffic in use (two classes), vehicles per day.
4 Maximum grain size and grading (four classes).

For five specification profiles, requirements for characteristics of geotextiles are given for the following.

1 Tensile strength and strain.
2 Cone drop diameter.
3 Energy index (product of maximum tensile strength multiplied by strain at maximum strength, divided by two).
4 Water flow velocity index.
5 Characteristic opening size O_{90} (0.15 mm or 0.20 mm).
6 Allowable tolerances for mass per unit area and static puncture strength (CBR).

Switzerland (SN 640 552a)

According to SN 640 552a (Vereinigung Schweizer Strassenfachleute, 1997), the fill is characterized by angularity and particle size, and particle grading in three classes. The bearing capacity of subsoil is separated into five classes by CBR value or plate-bearing test, and the traffic load into two classes by the addition of axel loads over the period of use. For three layer thicknesses of bearing layers, the requirements for geotextiles are defined according to tensile force and elongation.

The hydraulic properties are as follows.

1 Permeability: four classes of $k_G > 1 \times 10^{-4}$ m/s to $k_G > 1 \times 10^{-6}$ m/s in relation to the soil.
2 Opening size O_w for non-woven geotextiles: four classes, 0.05 mm < O_w < 0.20 mm, up to 0.05 mm < O_w < 0.50 mm in relation to the soil.

UK

In road construction, the following requirements are given (Highways Agency, 2001a, 2001b). The tensile load is to be defined by the client. The water permeability at a right angle to its principle plane shall be not less than 10 l/m² s under a constant water head of 100 mm, and the pore openings such that mean opening O_{90} is between 100 and 300 μm.

British Rail (1996) have the following requirements for separation layers. The minimum tensile breaking load shall be 10 kN/m and the CBR puncture resistance greater than 3000 N at a displacement of less than 60 mm. Water permeability shall be not less than 10 l/m² s under a constant water head of 100 mm and the pore openings such that mean opening O_{90} is between 30 and 85 μm.

7.3.4 Durability

For durability, all countries follow EN 13249 ff, Annex B (European Committee for Standardization, 2000).

Weathering resistance

All products are more or less susceptible to weathering. The best protection is by covering with fill. EN 13249, Annex B.1, gives the maximum exposure time after installation based on the results of an accelerated weathering test. According to EN 13249, the producer must define the time of covering in the CE-accompanying document: 'to be covered on the day of installation or to be covered within 2 weeks or within 1 month'.

Resistance against chemical and/or biological damage

The polymers are susceptible to chemical degradation in different ways. EN 13249, Annex B.2 to Annex B.4, defines test procedures for this. Based on these tests, the producer must define in the CE-accompanying document predicted to be durable for a minimum of 25 (or 5) years in natural soils with $4 < pH < 9$ and soil temperatures < 25 °C. For separation layers, these periods are normally sufficient.

7.4 Requirements for fill material

The fill material must be resistant against weathering for the length of time that it will be used, the grain size distribution must guarantee good water permeability, and the angularity and grain size distribution must provide a good inner friction. When recycled construction material with a higher content of cement concrete or cement mortar is used, then only polymers not susceptible to alkaline degradation (e.g. the polyolefins polypropylene and polyethylene) should be used, unless the fill is required only for a short period and the product is proven to be sufficiently durable.

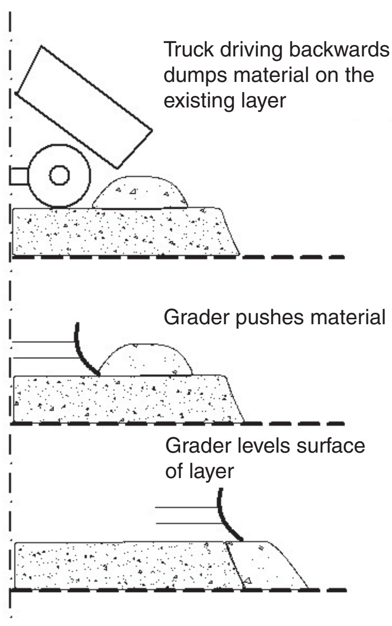
7.5 Construction

7.5.1 Placing of the separation layer

The product should always be placed evenly and smoothly. There must be sufficient overlapping that the soft soil under pressure beneath cannot erupt through any gaps. The requirements under various national specifications range from an overlap of 30–50 cm. Normally, the web should be placed across the direction of the work because, when unrolled lengthwise there is a high risk that the webs will drift apart when distributing the fill. However, when the overlaps are fixed, e.g. by sewing, glueing or clamping on subsoil with soil nails, the layer can be unrolled lengthwise. Overlapping should be in the direction of the work, like tiles on a roof.

7.5.2 Placing of fill

Driving directly on the fabric is not allowed, because of the vulnerability of the products and the risk that the web will drift apart. The transport vehicles delivering the fill should not drive on the geotextile. The fill should be unloaded where fill has already been put down and then distributed by pushing (Fig. 7.7). Only fill transport vehicles should be allowed to drive on the fill before compaction, because the fill layer does not reach full bearing capacity until after compaction.



7.7 Placing of the separation layer and of the fill.

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8
Landfill applications

T. MEGGYES
Bundesanstalt für Materialforschung und -prüfung, Germany

Before the passage of environmental regulations, wastes were dumped indiscriminately in open areas with no care for the environment. Many countries have developed stringent regulations on waste disposal and landfills in the last few decades. Today, landfills are engineered structures that are aimed at containing the wastes effectively. Upon stabilization of municipal solid waste, the landfill sites may be used for a beneficial purpose such as golf course, recreational park, industrial park and other such uses.

Stief (1986) proposed the concept of a multibarrier system for landfills based on the following.

1 The subsoil (the geological barrier).
2 A basal liner and capping (the technical barrier).
3 The waste body itself.
4 The control of landfill performance.
5 Aftercare or post-closure plan.

The most important element for containment is the technical barrier, the standard for which is a composite liner throughout most of Germany. Landfill technology has evolved with well-defined design and construction procedures. This chapter presents the salient features of landfill technology: waste separation and pretreatment, landfill concepts, standard composite and alternative liner systems, design considerations for earthen and geomembrane liners, safety analysis of liners, leachate drainage systems, and gas generation and management. Finally, cut-off walls for the control of pollution at the landfill sites are briefly described.

8.1 Waste separation and pretreatment

The waste body is affected by waste collection. A separate collection is more favourable than an assortment of non-separated collected waste. Even more important is the pretreatment of waste which can serve several goals. It is advantageous if processes that convert the waste into a less reactive form proceed before landfilling. In an ideal situation, the aim is to obtain inert materials. Pretreatment reduces the amount of waste which is particularly beneficial because landfill capacity is becoming increasingly scarce. Pretreated waste can often be used for new purposes, such as energy generation, or it can be recycled.

There are thermal, biological, chemical and physical pretreatment procedures; thermal pretreatment is quantitatively the most important. TI Municipal Waste (1993) requires municipal waste to be deposited with a maximum of 5% of organic components. To adhere to this limiting value, incineration is often the only possibility. The heat generated by incineration can be used for energy generation. The solid residues, namely slag and ash, may provide raw materials for building purposes.

Biological pretreatment can be aerobic (composting) or anaerobic (fermentation). Biological pretreatment exhibits great potential for waste reduction. The biological–mechanical pretreatment aims at the degradation of organic pollutants, a reduction in the quantity and better compactibility.

Landfill mining is a method of treating old landfills. After eliminating emission sources, the waste body is excavated, the remaining waste compacted and useful materials are separated and recycled. As a result, landfill void space is increased, pollution potential reduced and financial gains are achieved by material utilisation (Spillmann *et al*., 2007).

8.2 Landfill concepts

Landfill classes are defined by the type and concentration of pollutants in the waste and, accordingly, different design standards are required. In Germany, the standards for hazardous waste are defined by TI Hazardous Waste (1991) and those for municipal waste by TI Municipal Waste (1993). These two documents set the limits for various waste strength parameters and for some 20 pollutants in the leachate. The landfill classes thus defined (characterized by for example the permitted nickel content in the leachate) are as follows in order of increasing potential risk.

Municipal solid-waste landfills.
- Class I. Virtually inert waste, e.g. Ni content < 0.2 mg/l.
- Class II. Waste with higher pollutant contents, e.g. Ni content < 1 mg/l.

Hazardous solid-waste landfills, e.g. Ni content < 2 mg/l.

If the waste fails to meet the requirements for hazardous solid-waste landfills, it must be incinerated or disposed of underground.

Landfills may also be classified based on the waste composition and operations as inert material landfill, containment landfill, reactor landfill and co-disposal landfills.

8.2.1 Inert material landfill

In an ideal case, the inert material landfill contains only wastes that are neutral concerning emission and do not change the natural background concentration.

8.2.2 Containment and reactor landfills

Landfill strategy can be of the 'containment' type and 'reactor' type. Containment means an isolation from environmental effects and, in particular, water. For this purpose, the waste body is generally provided with liners. The capping protects the waste body from precipitation, while the basal liner aids the collection of leachate which is then transported to treatment.

In contrast, the reactor landfill encourages the processes which, in an ideal case, produce an inert content, which is stable under environmental conditions and releases no pollutants to the environment. One will be able to reach such a state only in exceptional cases.

When landfilling untreated municipal waste, a certain quantity of water has to be added to enable biological degradation processes. Otherwise the organic components may become 'mummified'. When water is added, mummification can be interrupted or replaced by biological degradation processes. In old deposits, the biochemical processes may be interrupted if a (subsequent) containment obstructs the access of water. A leaky capping may entail an uncontrolled inflow of water and uncontrolled reactions. In such cases it may be reasonable to feed water purposefully into old dry deposits to accelerate the biochemical reactions to bring them to completion within the operational time of the landfill.

The biological processes go through an aerobic and an anaerobic phase. In these phases both gas develops, e.g. methane, and organic pollutants accumulate in the leachate. At the end of the processes, the landfill content is mineralized. The landfill operator is interested in encouraging the biological degradation so that it is finished within the active aftercare phase which is 50–100 years. If the waste body has reached the mineralized state, it still contains inorganic pollutants, e.g. heavy metals of which only a limited amount is supposed to be released.

For all these reasons 'containment' and 'reactor landfill' are not necessarily contradictory strategies since the bioreactor needs a liner to protect the environment and a capping and irrigation system to control the inflow of water and to minimize leachate production the treatment of which is very cost intensive. However, in bioreactor landfills, the question of repercussion of elevated temperatures on performance of technical barriers has to be considered carefully.

8.2.3 Co-disposal landfill

The UK Department of the Environment (1990) define co-disposal as a calculable and supervised treatment of industrial and commercial liquid and solid wastes in interaction with biologically degradable wastes in a controlled landfill (Campbell, 1994). If co-disposal is to have advantages, the basic materials must be exactly defined and mixed with one another in a controlled way. The mixing process must produce as homogeneous a product as possible.

Sewage sludge and industrial sludges often contain so much water that they are not stable. The addition of municipal waste can yield a consistency which makes landfilling possible. Mixing harbour mud with clay, lime or cement may produce a stable material that can be landfilled. Mono landfills should be used to avoid inhibition of biological processes in municipal wastes by industrial pollutants. Co-disposal is outlawed in many countries today (Spillmann *et al.*, 2007).

8.3 Landfill phases

Landfill phases are important to distinguish for landfill construction and operation:

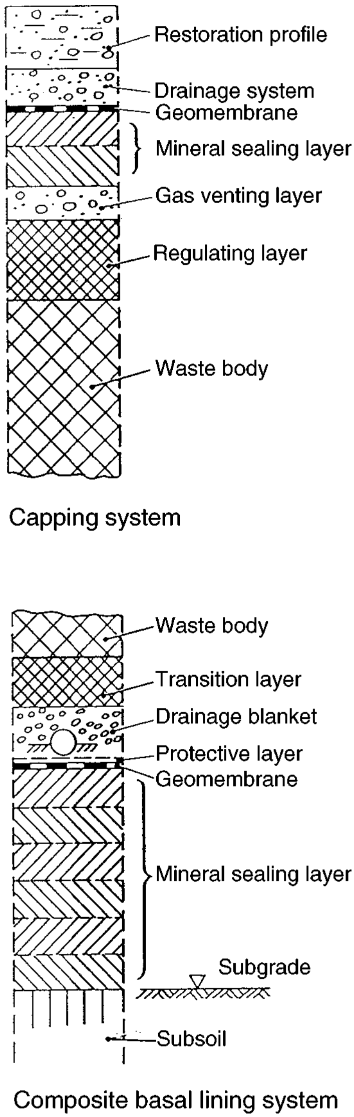
Phase 0 Basal liner construction.
Phase I Before filling wastes: the basal liner is fully or partially exposed.
Phase II Period of waste filling: this phase ends with the construction of the capping and the reclamation layer.
Phase IIIa Aftercare, part 1: it lasts 50–100 years. No more construction measures take place. Drainage and monitoring facilities are operated and maintained.
Phase IIIb Aftercare, part 2: unlimited in time, ageing of components of the liner systems occurs, impairing their effectiveness.

8.4 Landfill liners

Generally, landfills must be provided with a composite basal liner and a capping, both of which are composed of sealing, protective and drainage layers, although alternative liners with proven barrier equivalence can be used (August *et al.*, 1997; Müller, 2001). The capping system's main function is to prevent gaseous pollutant emission into the atmosphere and rainwater from percolating into the landfill so as to keep leachate generation to a minimum, while basal liners must protect the surrounding soil and groundwater from leachate pollution. Both the sealing layers are overlain by a drainage system which in the basal liner collects leachate and transports it to a treatment plant to be cleaned of pollutants and subsequently discharged in the nearby surface water. A capping drainage layer collects rainwater which requires no processing before being released.

8.4.1 Composite liners

Figure 8.1 illustrates the basal liner and capping systems for Class II of municipal waste according to TI Municipal Waste (1993). The most important element of these systems is the composite liner consisting of several lifts of earthen (mineral) material with a minimum thickness of 0.25 m of each lift and a minimum certified geomembrane 2.5 mm thick. The geomembrane is installed on top of the earthen layer in intimate contact with it and is protected by a protective layer against damage by gravel or aggregates in the drainage layer (Müller, 2001, 2006).



8.1 Liner and capping system for Class II landfills.

Liners for Class I municipal solid-waste landfills differ inasmuch as no geomembrane is required and the minimum total thickness of the earthen layer is only 0.5 m while basal liners in hazardous waste landfills are similar to those for Class II, except that the earthen layer must be a minimum 1.5 m thick. The permeability of the earthen layer of basal liners for all classes and in the capping system of hazardous solid waste landfills must be $k < 5 \times 10^{-10}$ m/s, and for Class II capping systems $k < 5 \times 10^{-9}$ m/s.

Composite liners are required according to TI Hazardous Waste (1991) and TI Municipal Waste (1993). One of their most important properties is the synergistic effect whereby the combined effect of the earthen layer (also called mineral or clay layer) and geomembrane is greater than the sum of the effects of the individual components. The properly installed and welded geomembrane is an effective barrier against convection: pollutants can only permeate it by diffusion, the rate of diffusion for various chemicals varying widely. For heavy-metal ions and other inorganic materials, the geomembrane is virtually a perfect barrier, although water can diffuse through it to some small extent. Hydrocarbons and chlorohydrocarbons can slightly penetrate geomembranes and diffuse through them, however they will then sorb within the earthen layer. Any further transport is only then possible by diffusion. Based on conservative assumptions on organic pollutant concentrations at the landfill base, breakthrough times of 16–18 years were obtained, indicating that the first pollutant molecule only reaches the lower surface of the earthen layer after this period of time, assuming a steady 'supply' of pollutants.

The long-term durability of geomembranes cannot be quantified exactly since it strongly depends on the quality of the individual product chosen. The minimum period of time of full efficacy is often given as 50 years; however, geomembranes of well-stabilized and highly stress-crack-resistant high-density polyethylene (HDPE) resins are likely to remain intact for several hundred years. Most importantly, geomembranes exhibit their full protective function in the active aftercare period when pollutant concentrations in the leachate are highest. Earthen sealing layers are generally assumed to maintain their efficacy as basal liners over geological periods of time even under chemical attack. However, they are likely to fail as capping liners in the long run. Since earthen materials are never absolutely watertight, small amounts of pollutants will permeate over longer periods of time.

The construction of the composite liners to meet the prescribed strict standards, in particular ensuring high-quality intimate contact between the geomembrane and earthen layer, represents a great challenge to construction companies. In order to solve this complex problem, general quality management systems closely defining building procedures are required.

8.4.2 Alternative liners

Instead of composite liners, alternative liners can be used if they can prove to be equally effective as composite barriers. Some of the alternative liners currently in use are as follows.

Mineral liners

The statutory requirement for not less than 20% of fines (less than 2 μm) and not less than 10% of clay minerals in the earthen layer is very often difficult to meet; therefore, equivalent earthen materials of different compositions are also used.

Well-graded materials have grain size characteristics approximating Fuller's curve, i.e. there is a stable structure and only a small fines content is required to achieve low permeability. This type of liner exhibits less shrinkage on losing water, but they are not as flexible as earthen materials with a high clay content and this can be a disadvantage in the event of differential settlement. DYWIDAG dry liners consist of 8% bentonite and two well-graded sand and gravel fractions. They yield an extremely high dry density and, on the addition of water, they tend to swell (volume increase), thus cracks in the liner will not be expected.

Multiple liner with a sorption layer

A multiple liner is composed of two layers with principally different functions: one of these serves as containment, i.e. to prevent pollutants from leaving the landfill, and the other's function is to minimize the consequences of pollution, should containment measures fail.

Improved mineral liners

Silica glass, quaternary ammonium compounds, montan wax and other additives can provide high levels of imperviousness even in the absence of the clay fraction in the earthen material and flexibility can even be greater than that of pure mineral mixtures. Silica glass combined with silanes has been used successfully to improve cut-off materials. Silicone-carbon bonds in organosilane hydrogel modifiers are extremely resistant to biochemical degradation. Recently polymer-amended sand–bentonite mixtures have proven to be an alternative to compacted clay liners in capping systems.

Asphalt liners

Asphalt is composed of a well-graded grain skeleton, a filler (rock flour) and a bitumen. Based on experience in hydraulic engineering, liner systems incorporating asphalt are increasingly being used; in Switzerland, more than 20 landfills have been constructed with asphalt liners. In Germany, a combination of a mineral layer directly overlain by an asphalt liner is preferred. Short-term tests have shown asphalt liners to be a good barrier to the convective transport of aqueous solutions, while their stability and resistance to heat, weather and penetration are advantageous features. A liner made of asphalt concrete in accordance with the requirements is almost impermeable to water and gas. It is very robust against mechanical stresses and desiccation is irrelevant. However, it is susceptible to chemical attack by hydrocarbons and concentrated hydrocarbons can virtually destroy it.

Geosynthetic clay liners

Geosynthetic clay liners are also known as bentomats, and they generally consist of bentonite sandwiched between two geotextile layers which are needled or sewn together to provide shear strength. Water-saturated bentonite exhibits extremely low permeability and, in principle, a bentomat with a few millimetres thickness is capable of producing the required barrier function. For practical reasons, thicknesses of a few centimetres are used. Bentomats can be manufactured industrially and installed in rolls, similar to laying techniques for geomembranes. They are light, easy and quick to lay, exhibit low susceptibility to settlement and are easy to repair; however, root penetration is a potential risk.

Capillary barriers

Capillary barriers consist of an upper fine-grained capillary layer and a lower coarse-grained capillary block. Capillary menisci are formed at the interface between the upper and lower layer by water percolating into the upper layer. Capillary force on these menisci and a very low unsaturated permeability in the capillary block prevent water from penetrating the coarse-grained layer. Capillary barriers are almost insusceptible to desiccation and subsidence. Both the sand used for the capillary layer and the gravel used for the capillary block must be well graded, their optimum layer thicknesses being 0.4 m and 0.3 m, respectively, and a minimum slope gradient of between 1:10 and 1:5 is advisable. Capillary barriers are most effective when used in combination with a gas-tight sealing layer (e.g. geomembrane or asphalt) and they are most often used on sloping areas of landfill caps.

8.5 Design considerations for earthen liners

8.5.1 Permeability

The imperviousness of an earthen material is characterized by the hydraulic conductivity (or permeability), the k value, defined by

$$v = kI$$

where

v = velocity, i.e. the flow rate per unit cross-sectional area
I = hydraulic gradient

The requirements with regard to imperviousness are much more exacting in landfill construction than in earthworks and hydraulic engineering. No other geotechnical parameter is as difficult to obtain reliably as permeability, as even small variations in water content and density may result in changes in permeability by up to two orders of magnitude. Three types of laboratory apparatus are used for permeability measurements: rigid-wall permeameters, triaxial cells and consolidometer permeameters. However, the latter produces a one-dimensional deformation state which corresponds most closely to that in a landfill basal liner and is therefore best suited to investigate the influence of a load on permeability. In the field, permeability is measured by infiltration; a hollow cylinder is inserted or pressed into the liner from the surface and filled with water, and then losses due to infiltration measured. Large-area lysimeters are also used to catch all the percolated water. Field tests are considered necessary since it has repeatedly been found that permeability measured in the field is 10–1000 times higher than in the laboratory. Acceptably low permeability values can only be achieved if there are no macropores in the earthen liner, thus the homogeneity of the installed liner is by far its most important parameter.

8.5.2 Mechanical properties

Heavy landfill overburden pressures can result in subsoil subsidence, and the basal liners have to cope with differential subsidence without sustaining any damage, especially at the toe of slopes and on the slopes themselves. Collapses within the waste body and the subsidence of the subsoil are of particular importance for capping and interim liners. Radii of curvature of between 40 and 70 m were found as deformation limits in a large-scale test rig, as indicated by a sudden increase in permeability.

Deformation here is judged by the extent to which it impairs imperviousness and this impairment increases in the following order: homogeneously distributed distortion, limited shear zones or shear joints and, finally, gaping cracks. Unlike cracks, shear joints are capable of transferring compressive and shearing stresses. If the liner material contains a considerable amount of sand and gravel, shear zones form, leading to disturbance which can result in an increase in permeability. For liner materials, strain is a more important factor than stress.

Self-healing is the capacity of a clay material to close up again over time cracks which may be of mechanical or other origin, e.g. desiccation or pollutant influence. The most important factors of self-healing are plasticity, swelling and colmation. Bentonite addition and lateral pressure can increase self-healing capacity. Generally, the low surcharge in landfill caps is not sufficient to suppress cracks.

8.5.3 Moisture and pollutant transport

The moisture balance beneath landfills differs from that in undisturbed soil, as a containment landfill restricts or prevents moisture supply both to earthen layers and to the subsoil, and exothermic processes within the landfill may, over a long period of time, lead to desiccation in the earthen layer of the liner.

The tests carried out have shown that desiccation cracks are not expected to form if suction does not exceed the overburden pressure in absolute terms and this is shown to be in agreement with several theoretical considerations. Whether or not an earthen liner suffers desiccation must be decided for each individual case based on a coupled water transport and soil deformation analysis, analogous to static calculations for engineering structures. This analysis consists of a description of the geometry, selection of appropriate physical and mathematical models, the determination of stresses, temperature and boundary conditions, evaluation of soil mechanical and transport parameters, calculation of heat and water transport and of soil deformation, assessment of the probability of cracking and, finally, a discussion of preventive measures. The most important aspect of this last point is that the suitability of earthen liners should be judged by virtue of their long-term properties rather than by those on placement. The efficacy of earthen liners is likely to be challenged after a period of 100 years or more, when it is predicted that the drainage system and the geomembrane may fail. Placement water content should not therefore be greater than that at long-term equilibrium. Placing the earthen layer in the form of small clods rather than in a highly homogenized form minimizes tensile stresses, thus preventing gaping cracks, should desiccation occur in the distant future (Holzlöhner, 1997).

8.6 Design considerations for geomembrane liners

A composite liner must be constructed so that the geomembrane (most commonly HDPE) will lie sufficiently flat on the flawless surface of the mineral liner and that subsequent surcharges such as the protective, drainage, waste and recultivation layers produce a complete contact, known as intimate contact, between them. This can only be achieved by means of a construction method which is well coordinated throughout the landfill construction industry. The geomembrane must be smoothed and ballasted after placement to prevent installation failures such as excessive waves and wrinkles, and sagging and damage to the mineral liner. Stability and the friction behaviour between the geomembrane and adjacent layers on slopes is critical and geomembranes with rough or textured surfaces are especially suited to inclined areas by virtue of their high coefficients of friction. To avoid any tensile overload of the geomembrane in basal liners, shear resistance between the geomembrane and protective layer must not exceed that beneath it, i.e. between the earthen layer and the geomembrane (Averesch and Schicketanz, 1998; Müller, 2006).

Load plate experiments into the effectiveness of geotextile and clay–synthetic protective layers showed that temperature, time and surcharge all have a considerable influence on strain in geomembranes. Overlapping and folds in protective layers should be avoided, and drainage gravel must not get beneath the protective layer. Heavier geotextiles and combined earthen–synthetic layers (geocontainers) have been shown to provide more effective protection for geomembranes.

Leaks from landfills can be monitored using grids of electrically conductive material installed directly above and below the geomembrane to measure local electrical resistance, changes in which can be attributed to leaks.

8.7 Drainage systems

The main functions of the drainage system at the base of the landfill are to remove leachate from the waste body and to collect it at defined points and to avoid leachate build-up on top of the basal liner. A basal drainage system consists of three elements.

1 Drainage layers (when laid flat these are also known as area filters).
2 Drainage pipes.
3 Leachate shafts.

The drainage layer should be installed as a blanket of mineral drainage material, specified as round grained with a particle size of 16–32 mm, or where necessary 8–16 mm. The pore volume content of 16–32 mm material is needed to counteract incrustation, i.e. microbiological deposition processes, which can drastically reduce the pore space in the drainage layer and the free cross-section of drainage pipes. Finer materials can become completely impermeable within a year and round grain is required to reduce the pressure from the individual grains on the underlying geomembrane and thus to avoid perforation. Requirements also demand a minimum thickness of 0.3 m and a long-term minimum permeability of 1×10^{-3} m/s, and fine particles must be prevented from being washed out of the waste into the drainage layer.

The clogging effect due to incrustation in drainage blankets and leachate collection pipes increases approximately proportionally to the concentration of organic matter in the leachate. While drainage pipes are quite easy to clean using mechanical methods, the rehabilitation of drainage blankets is somewhat difficult. Redissolving and disinfection measures combined with biological pretreatment of the waste have been suggested, hydrochloric, perethanoic, nitric and perchloric acids being effective redissolving agents, and perethanoic acid and hydrogen peroxide effective disinfectants, killing bacteria which cause incrustation. Biological pretreatment of the waste reduces the organic content of the leachate and, consequently, bacterial activity as well. It also prevents the development of the acidic phase in the landfill, thus calcium, magnesium, manganese and sulphate will be not precipitated from the leachate.

Drainage pipes are arranged herringbone fashion on both sides of a central longitudinal axis. Leachate should run freely off the landfill, the surface of the basal liner being pitched like a roof for this purpose. The lateral slope down to the pipes should be not less than 3% in the long term, and the gradient of the pipes greater than 1%. A lateral spacing between the drainage pipes should not be greater than 30 m. In Germany, the approved materials for landfill drainage pipes are HDPE (the most commonly used), polypropylene and plasticizer-free polyvinyl chloride. The chemical and biological resistance of all three materials is well established. The pipes should have a minimum nominal diameter of 0.25 m and the pipe walls of the upper 240° of the cross-section should be perforated with a minimum water intake area of 0.01 m^2 per metre length of pipe. The geometry and manufacturing technology of apertures are important parameters which can influence the pipes' strength and durability under landfill conditions.

Leachate shafts are arranged at the deepest points of the drainage pipe network, preferably on the external slope edge of the landfill, allowing access to the drainage pipes and hence facilitating monitoring or maintenance work.

Geocomposite drains are a cost-effective and technically attractive alternative to mineral drainage layers. However, water flow capacity might be impaired in the long run by creep and degradation processes of the plastic material. Therefore, the long-term water flow capacity has to be carefully assessed by special test methods (long-term creep test and long-term shear strength test).

8.8 Landfill gas generation and management

Landfill gases are the result of microbial decomposition of solid waste. Gases produced include methane, carbon dioxide and lesser amounts of other gases (e.g. hydrogen, volatile organic compounds and hydrogen sulphide). Landfill gas production rates vary spatially within a landfill. Migration of landfill gases occurs owing to concentration gradients and pressure gradients. Generally, landfill gas moves through the path of least resistance. Uncontrolled migration of landfill gas can cause explosion hazards, undesirable odours, physical disruption and damage of caps, and toxic vapour emissions. Regulations require that landfill gas should be monitored to ensure that methane concentration does not exceed 25% of the lower explosion limit. Generally, gas collection systems that include passive and active gas collection are used. At large landfills, the collected gases are used for energy recovery. Thus, landfill gas recovery systems can reduce landfill gas odour and migration, and the danger of explosion and fire, and may be used as a source of revenue that may help to reduce the cost of closure (Sharma and Reddy, 2004).

8.9 Cut-off walls

Cut-off walls are used in order to prevent horizontal pollutant migration from landfills and contaminated sites, the most frequently used walls being slurry trench cut-offs and steel sheet pile walls. In the former case, trenches are excavated supported by fluids capable of solidifying and forming a diaphragm wall (single-phase diaphragm wall) or which are then displaced by another material which, in turn, is capable of solidifying (two-phase diaphragm wall). Cut-off slurries consist of clay (bentonite), cement, water and filler materials (rock flour). Cut-off walls are especially effective if they can be socketed into an impermeable layer. The most common construction principles include excavation of soil and placement of sealing material, displacement of soil, installation of sealing material and reducing permeability of soil in place (injection and freezing) (Holzlöhner *et al.*, 1995).

8.10 Safety analysis of landfill liners

Safety analysis is based on the assessment of the effects of an incident (e.g. leakage), assuming a worst-case scenario. It is aimed at estimating groundwater quality in the wake of leachate escape into the subsoil. The safety of a liner (sealing) system is measured by the extent of leachate containment. A liner (sealing) system is considered safe (suitable) if it provides a sufficiently strong barrier against certain pollutants or their mixtures in leachate over a long period of time. A barrier effect is considered suitable if the liner (sealing) system yields the same leakage behaviour as a standard liner system (e.g. a composite liner according to TI Hazardous Waste, 1991) (Heibrock and Jessberger, 1995).

Since landfill liners are expected to maintain their efficacy over periods of time several orders of magnitude greater than are conventional engineering structures, long-term behaviour and very slow physical processes are of special interest. Transport processes, desiccation in earthen layers and conditions which may endanger geomembranes, protective layers and drainage systems are fairly well understood. The mechanical properties of earthen materials can be improved and testing methods and apparatuses are available for the determination of critical values. Safety analysis shows that composite liners are extremely safe under the right conditions and, therefore, represent the standard by which the efficacy and equivalence of alternative liners should be measured (August *et al*., 1997).

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9

The use of geosynthetics as barrier materials in civil engineering

H. ZANZINGER
SKZ-TeConA GmbH, Germany

9.1 History

9.1.1 First applications

The use of geosynthetics in containment ponds, reservoirs and canals can be traced back to the 1940s and actually emerged in the 1960s and 1970s (Duquennoi, 2002). It is generally admitted that liquid containment represents the first use of geosynthetic barriers (GBRs), or at least what can be considered as historical GBR forerunners. Koerner (2005) records the probable use of rubber liners as early as the 1930s and the use of poly(vinyl chloride) (PVC) liners in the 1940s. Monjoie *et al.* (1992) confirmed these beginnings by citing the use of isoprene–isobutylene (better known as butyl) rubber liners in the 1940s. Almost at the same time, canal lining techniques using the *in situ* application of sprayed-on bituminous coatings were developed in the USA. It is estimated that, between 1947 and 1951, more than 10^6 m² of bituminous canal liners were applied in that way. Together with this first-generation synthetic liners the first pathologies appear, including brittleness and cracking of unprotected PVC, localized mechanical damage of unreinforced bituminous coatings, etc. These early inconveniences set the trend for technical enhancement, which, from that time, took the following directions.

Another early polymeric geosynthetic barrier (GBR-P), chlorosulphonated polyethylene (CSPE), was introduced for reservoir and landfill liners in the late 1960s. In the 1970s, polymeric, elastomeric and bituminous GBRs were installed as liners to contain water and contaminated industrial liquid effluents in regions where natural clay was not available. While the incentive to develop GBRs may have originated from the substitution where there was a shortage of natural clayey soils, now they are used worldwide. The use of high-density polyethylene (HDPE) GBRs as liners in geotechnical hydraulic applications started in the 1960s while their growth in the USA and Germany was stimulated by governmental regulations originally enacted in the early 1980s in landfill applications. The primary function is always containment as a liquid or vapour barrier or both. Koerner (2005) reported that the US market is divided as follows: HDPE (40%), linear low-density polyethylene (LLDPE) (25%), PVC (20%), CSPE (5%), flexible polypropylene (fPP) (5%) and ethylene propylene diene terpolymer (EPDM) (5%).

In some countries (such as France and the Netherlands), bituminous geosynthetic barriers (GBR-Bs) are seeing some use for linings of reservoirs, surface impoundments and landfills.

Since the late 1980s, clay geosynthetic barriers (GBR-Cs) have been used as an alternative to soil barriers in various applications to contain liquids, industrial effluents and in association with HDPE polymeric GBRs in landfills. The first product was adhesive-bonded bentonite between two geotextiles. The second product at about the same time was a needle-punched geocomposite consisting of two geotextiles and bentonite powder between. A further bonding method for GBR-Cs is stitch bonding. Another manufacturer uses an adhesive to bond bentonite powder on to an HDPE or LLDPE GBR-P. The engineering function of a GBR-C is containment as a hydraulic barrier to water, leachates or other liquids and sometimes to gases. As such, GBR-Cs are used for replacements to either compacted clay liners (CCLs), or GBR-Ps or GBR-Bs, respectively, or they are used in composite manner to augment the more traditional liner materials.

9.1.2 Terms

The term 'geomembranes' was used in the 1990s and is still in use. It was originally devised to replace previous imprecise terms such as 'pond liners', 'flexible membranes' and 'waterproofing sheets'. The term 'geosynthetic clay liners' (GCLs) is still used today. GBR-Cs are geocomposites consisting of geosynthetics and bentonite. Other names are 'clay blankets', 'bentonite blankets', 'bentonite mats', 'prefabricated bentonite clay blankets' and 'geocomposite clay liners'.

As there were too many definitions on geomembranes and geosynthetic clay liners in various countries, the ISO Technical Committee 221 of the International Organization for Standardization and the CEN Technical Committee 189 of the European Committee for Standardisation decided together, to come up with the new term 'geosynthetic barriers' which covers all GBR-Ps, GBR-Bs and clay geosynthetic barriers. The definitions given in EN ISO 10318: 2005 are the following: 'A geosynthetic barrier is a low-permeability geosynthetic material, used in geotechnical and civil engineering applications with the purpose of reducing or preventing the flow of fluid through the construction. A polymeric geosynthetic barrier is a factory-assembled structure of geosynthetic materials in the form of a sheet, which acts as a barrier. The barrier function is essentially fulfilled by polymers. It is used in contact with soil and/or other materials in geotechnical and civil engineering applications. A clay geosynthetic barrier is a factory-assembled structure of geosynthetic materials in the form of a sheet, which acts as a barrier. The barrier function is essentially fulfilled by clay. It is used in contact with soil and/or other materials in geotechnical and civil engineering applications. A bituminous geosynthetic barrier is a factory-assembled structure of geosynthetic materials in the form of a sheet, which acts as a barrier. The barrier function is essentially fulfilled by bitumen. It is used in contact with soil and/or other materials in geotechnical and civil engineering applications.'

9.2 Products

9.2.1 Polymeric geosynthetic barriers

Various types of polymers have been used. The principal types appeared in the following order: chlorinated polyethylene (CPE), chlorosulphonated polyethylene 'Hypalon' (CSPE), PVC, polychloroprene 'neoprene', EPDM, HDPE, low-density polyethylene (LDPE), LLDPE, very-low-density polyethylene, fPP and thermoplastic polyolefins (TPO). Various compounds, such as plasticizers, antioxidants and mineral fillers, have been developed and used to enhance chemical stability, durability, thermal, biological and mechanical properties of the GBR-Ps. The inclusion of reinforcing materials was originally used to strengthen bituminous liners but they are not the only liners to be reinforced; CPE-R, CSPE-R, PVC-R, fPP-R and EPDM-R are also reinforced, which the letter R indicates.

The production of the raw materials includes the polymer resin itself, and various additives such as antioxidants, plasticizers, fillers, carbon black and lubricants (as a processing aid). The raw materials are then processed into sheets of various widths and thicknesses by extrusion, calendering and spread coating.

HDPE, LLDPE and fPP are manufactured by an extrusion method. One process uses a flat die, which forces the polymer formulation between two horizontal die lips, resulting in a smooth polymer sheet of closely controlled thickness from 0.75 to 3.0 mm. The sheet from one extruder varies from 1.8 to 4.6 m. If two parallel extrudes are used, the width can be increased to 9.5 m. Another process uses a circular die (called blown film extrusion), which forced the polymer formulation between two concentric die lips oriented vertically upwards. The creation of a roughened surface on a smooth sheet in a process is called texturing. A high-friction surface can be created. These methods are subjected to co-extrusion with nitrogen gas, impingement of hot polyethylene particles, lamination of polyethylene foam and structuring, or pattering, a surface.

PVC, CSPE and scrim-reinforced GBR-Ps including CSPE-R and fPP-R are manufactured by calendering, a method in which the polymer resin, carbon black, filler, plasticizers (if any) and additive package are weighed and mixed. During mixing, heat is added, which initiates a reaction between the components. As a continuous mass it is conveyed through a set of counter-rotating rollers (called a 'calender') to form the final sheet. When an additional fabric scrim is included, they are called 'reinforced'.

A thermoset GBR-P is EPDM, both non-reinforced and scrim reinforced

(EPDM-R). The processing is similar to the calendering process and an additional autoclaved manufacturing.

9.2.2 Bituminous geosynthetic barriers

A GBR-B is a prefabricated geocomposite, which includes reinforcing fabrics consisting of multiple layers such as geotextiles, glass fibre woven or non-woven and root barriers. The bitumen is coated with sand.

Modified styrene–butadiene–styrene bituminous GBRs should not be in contact with non-polar solvents (aromatic solvents, aliphatics and halogenics) for a long time, nor with very strong acidic and basic solutions (pH <2 and pH >9, respectively). By virtue of their inherent porosity, geotextiles rarely serve in the containment function. The exception is when the geotextile is purposely impregnated with bitumen or polymer. These few products will not be seen as GBRs.

9.2.3 Clay geosynthetic barriers

There are two bonding concepts. One is adhesive bonding of the product and the other is needle punching and stitch bonding of the product. The second types of technique can transfer shear stresses through the plane of the plane even in the hydrated stage of the bentonite. Others deliver already prehydrated bentonite for certain reasons. Some products may have a bituminous coating on the outside geotextiles or a polymer-impregnated cover geotextile and others even have an additional LDPE foil on or beneath the cover geotextile.

Bentonite essentially consists of clay minerals of the smectite group, montmorillonite being the dominant species. Because of their particular physicochemical properties, montmorillonite crystals contain exchangeable cations, mostly sodium or calcium. For practical purposes, a distinction is made between bentonites that predominantly consist of montmorillonite with sodium cations, and bentonites that predominantly consist of montmorillonite with calcium cations. They are encountered in nature and obtained by open-pit mining.

Most of the GBR-Cs use natural sodium bentonite. Some use a bentonite, which in nature occurs as calcium bentonite but by processing is converted to a sodium bentonite. Until now there is only one GBR-C that uses natural calcium bentonite. Some manufacturers add chemicals to improve the performance of their bentonite. The bentonite is delivered to the GBR-C manufacturer as a typically dry (about 10% moisture content owing to the extremely high hydrophilic nature of the bentonite) powder or in granulated form. The masses per unit area range from 3 to 6 kg/m^2 for sodium bentonite and from about 8 to 10 kg/m^2 for calcium bentonite. Although it is derived from a mined natural product, there is very little scatter in the properties of processed bentonites delivered by competent suppliers.

Although bentonite particles are extremely small, their surface area is very large. They can adsorb great amounts of water. Sodium bentonite can reach water contents of about 600%, and calcium bentonite up to 300%. They swell while they acquire these high water contents. As noticed by the difference in free swell water contents, the swelling capacities of sodium and calcium bentonite are different and so are their hydraulic conductivities, as well as their shear strengths. The hydraulic conductivity is lowest for sodium bentonite which is why this mineral is preferably used for GBR-Cs. The permittivity is typically in the range $(5–1) \times 10^{-9}$ s^{-1}. However, at the same time, the shear strength is also lowest for sodium bentonite, which causes concerns with respect to the stability of slopes (Madsen and Nüesch, 1995).

Since the properties that make sodium bentonite perform as an excellent sealing material are related to its interaction with water, the quality of bentonites can be evaluated on the basis of their swelling capacity (free swelling), water adsorption capacity, methylene blue adsorption, cation exchange capacity and density of cations.

These properties can be determined by relatively simple tests used to identify bentonites in practice (Egloffstein, 1995).

Since bentonite exhibits swelling with substantial volume increase when water is added, it undergoes shrinkage by equivalent amounts of volume reduction when it desiccates. Sodium bentonite can be subjected to an unlimited number of swelling and shrinkage cycles without changes in its properties as long as the chemistry of the water–clay mineral system is not changed. Desiccation cracks heal and close again because of swelling, provided that the soil structure is not influenced by chemical processes. However, water percolating through soil invariably contains some cations and anions in solution, which can react with the sodium montmorillonite minerals. This means that an exchange of some of or even all the sodium ions by other cations, in practice mainly by calcium ions, has to be anticipated, when wetting–drying cycles occur. So, depending on the physical and chemical milieu parameters, the properties of the GBR-C in place may undergo some changes with time owing to cation exchange.

GBR-Cs are composites of bentonite and geotextiles. The geotextile components provide strength. They determine the mechanical properties of GBR-Cs. The shear strength of hydrated bentonite is extremely low. In fact, a thin wet bentonite layer acts as a lubricant. GBR-Cs without a mechanical bond between the three layers would have no internal shear strength at all. So the ties between the two geotextile layers with the sandwiched bentonite between them are of prime importance. These ties are stressed when the bentonite is hydrating and the swelling bentonite experiences an increase in volume. The swelling pressure that builds up within the GBR-C during hydration, as a result of the restriction of the bentonite volume increase by the bonds, improves the sealing effect. The fibres that tie the GBR-C together are permanently stressed as long as the GBR-C is wet. Their tensile forces increase when the GBR-C is placed on a slope and the textile ties prevent the soil layers above the GBR-C from sliding on a slip plane within the bentonite layer.

Consequently, the internal strength of GBR-Cs must be warranted for long-term conditions. Tests and experience up to now prove that the long-term internal shear strength of the currently available GBR-Cs meets design requirements for slopes in landfill engineering. In most practical cases, interface friction at the upper and lower surfaces of the GBR-C turns out to be the controlling parameter in stability analyses of landfill slopes rather than the internal shear strength.

9.3 Design

9.3.1 Subgrade preparation

The subgrade of any GBR must be free of any vegetal and organic matter. All elements that are potentially aggressive toward the GBR (e.g. sharp stones) should be eliminated and/or avoided. The subgrade then has to be compacted in order to optimize its bearing capacity, according to state-of-the-art soil mechanics. The bottom of the structure should form a slight slope, between 1 and 2% lengthways, and between 2 and 3% sideways. The embankment slopes should be designed according to state-of-the-art soil mechanics; this is important, as GBR lining systems cannot be used to reinforce slopes. For many applications, a 1V:3H slope is advised, and 1V:1.5H is to be considered as the maximum. The embankment top should be wide enough to enable geosynthetic anchoring; minimum anchoring length is generally 2 m for ponds and reservoirs and 1 m for canals, but specific designs must be taken into account.

9.3.2 Underliner drainage and protection

In hydraulic applications such as reservoirs and canals, it is generally not recommended to lay a GBR directly on the subgrade, except in particular cases such as landfills when the risks of puncture of the GBR and underliner pore water of gas pressure have been catered for. A better way to prevent the above-mentioned risks is to design specific underliner systems. Geosynthetics are particularly adapted to this application.

An underlining water drainage system can collect water leaking from a pond, canal or reservoir. It can also prevent an uplift of the GBR due to back pressure from a raised water table. Either gravel layers or geosynthetic drainage layers can be used. They can additionally be used as leakage detection systems. Another issue is the collection of gas from organic fermentation or compressed soil pore air. Both water and gas must be separated and collected in water trenches at the bottom of the system and in gas vents passing through the GBR at the top of the lining system respectively. Geotextiles are generally preferred as they combine different functions: gas drainage and protection of the underliner.

9.3.3 General criteria of the choice of a barrier system

The core of a lining system is the GBR. As the variety of the products is so wide, criteria have to be given to choose the suitable barrier system.

1 The cost of a geosynthetic lining system is always critical and depends on many parameters, which cannot all be accurately predicted in a technical article.
2 The hydraulic criterion will determine the choice of the behaviour regarding liquid transfer. Mass transfer will generally be smaller through polymeric or bituminous geosynthetic barriers, especially for non-organic chemical species, than through GBR-Cs. Very small liquid flow is possible through GBR-Cs, following Darcy's law. It must be emphasized that this statement holds true for undamaged, continuous polymeric or bituminous GBRs only, which highlights the necessity to protect these systems against puncture and seam defects.
3 Mechanical criteria are also very important, even if GBRs are never designed for mechanical functions in a structure. They may nevertheless be subjected to mechanical stress, such as tension on slopes, and in the case of subgrade settlement, or puncture by gravel and irregular subgrade. Even if mechanical stress is lessened or eliminated by proper design, e.g. with a geosynthetic protection, it is generally necessary to select GBRs with mechanical properties adapted to the expected mechanical conditions. The safety criteria depend on the experience and philosophy in different countries and regions.
4 Chemical resistance and durability criterion concern the polymer resin as well as the various compounds. It is well known that HDPE is the most chemically stable polymer available; nevertheless, the ageing phenomena depend on the use of an adequate stabilization. The physicochemical durability of PVC GBRs is highly dependent on the durability of the plasticizers and the principal factor of a GBR-C is the ion exchange capacity of the bentonite in combination with desiccation. For a liquid-waste containment, it is therefore of the utmost importance to determine correctly the expected composition of contained liquid and to compare it with the chart of chemical resistance of the GBR.
5 The question of whether a mineral liner or a GBR, and which type of GBR would be the better choice in a considered case, has to be answered on a rational basis, reflecting upon the following aspects: type of waste in a landfill or type of chemical solutions in an area where chemicals are stored and trans-shipped, requirements as related to the environment, gas-tightness and/or watertightness of the liner, expected lifetime considering ageing (resistance against environmental stress cracking and oxidation resistance), slope stability analyses, availability of sealing materials and local climatic conditions.
6 Nevertheless, it is important also to consider the ease of installation and seam performance criteria. Indeed, the best geosynthetic lining product will always be limited by its ability to be correctly installed and seamed. For example, phenomena such as thermally induced wrinkling or moisture-dependent welding quality may affect some GBRs and must be taken into account in the

planning of the installation. A strict application of a state-of-the-art installation procedure for each type of GBR must be required, as well as a state-of-the-art quality assurance and quality control programme. GBR-Cs are easy to install, but free swelling has to be avoided. The confinement–hydration procedure together with correct overlapping and seaming must be strictly respected.

Besides single-lining systems, it is possible to install double-lining systems using two polymeric geosynthetics with a drainage layer in between, or two GBR-Cs, or a combination of a GBR-P and a GBR-C.

9.3.4 Overliner protection and cover

Generally the best way to prevent ageing of geosynthetics is to limit their exposure to weather by an early covering with soil, for example. Such overliner soil layers prevent damage of the GBR caused by floating or transported ice and wood in canals, by operating vehicles and machines, by burrowing animals and by vandalism. The GBR must be protected against coarse granular material. Therefore, geotextiles are often used as protection layers between the GBR and the cover soil layers. On landfill bottom liner systems, combinations of sand and geotextiles or geocomposites are used as protection systems for the highest requirements against puncturing. Also, in some canals, sand-filled geotextiles were installed as protection layers.

Other common designs consider concrete covers using precast blocks or *in situ* poured reinforced concrete covers. In some installation procedures temporary ballast over the GBR is necessary in order to prevent uplift of the GBR due to wind action. Other applications of GBRs do not need any covering, e.g. many containment ponds and reservoirs. Most of the GBRs in canals use a covering system. All GBRs in landfills are covered as long as it is not only a temporary covering of the waste body.

9.3.5 Connections

GBRs have to be connected to structures such as manholes, shafts, pipes, walls and embankment tops. Difficulties arise when details are required. If the space is limited and automated equipment cannot be used, hand labour and experience are all important. Design in this case is really a matter of detailing and visualizing how settlements, deformations and other stress and strain mobilizing phenomena might influence the connection. A proper construction is not as simple as the drawings may appear. Care and true craftsman-like work are required for trouble-free and leak-free performance (Koerner, 2005).

9.4 Hydraulic applications

9.4.1 Liquid-containment ponds and reservoirs

In former times, polymeric GBRs were also named 'pond liners', which was the original use of the polymeric materials. Hazardous and non-hazardous liquids are stored in containment ponds as well as liquid waste and water from the agriculture industry.

The construction is not simply digging a hole, putting a liner in it and then filling it with the liquid. It is a straightforward task but has much to do with the atmospheric exposure and possible damage to the GBR. To shield the liner from ozone, ultraviolet (UV) light, temperature extremes, ice damage, wind stresses, accidental damage and vandalism, a soil cover of 30 cm is usually required. Slope stability analyses and an adequate anchorage of the GBR are necessary. A geotextile placed beneath the GBR that is directly on the prepared soil subgrade before the placement of the GBR is strongly recommended. It provides a clean working area for making field seams and it adds puncture resistance to the GBR.

In certain cases, reservoirs for liquids and quasisolids such as industrial and agricultural sludge are covered with GBRs. The reasons for that are losses due to evaporation for irrigation reservoirs, savings on algae control chemicals for water reservoirs, reduced air pollution for reservoirs holding chemicals and agricultural waste, reduced need for draining and cleaning, increased safety against accidental drowning, protection from natural pollution entering the reservoir, temperature control for anaerobic decomposition of agricultural and organic wastes, and protection from intentional pollution. GBRs with superior UV and exposed weathering resistance have to be used for those covers. For smaller structures, the cover can be fixed and remain stationary. For larger span lengths, the use of a floating cover that resides directly on the liquid's surface as it varies in elevation will be considered (Koerner, 2005).

9.4.2 Canals

In canals the liquid is moving. The usual liquid is water, but many other liquids, including industrial chemicals and wastes, also need to be conveyed.

The GBR is placed either directly on the prepared soil subgrade or on a previously installed geotextile. A uniform thickness soil cover is commonly placed over the GBR. The cover soil type and its thickness depend on the velocity of the liquid and the turbidity of the flowing water. If these forces are such that the soil cover is eroded, they will act directly on the GBR. Therefore it is not uncommon to cover the liner with a non-erodible cover of asphalt, shotcrete or concrete.

Also the rehabilitation of old canals and their linings with GBRs is a challenging task in a rapidly growing field. It is not only polymeric and bituminous GBRs that are used in canals. Fleischer and Heibaum (2002) reported on an underwater installation of clay GBR in a waterway engineering project in Germany. Normally, the GBR-C will be placed in the dry. In these waterways, the GBR-C is loaded by severe dynamic hydraulic loading due to navigation, i.e. sudden drawdown of the water level, waves and return current. The overlap of the GBR-C sheets was 1 m and divers carefully checked this. The GBR-C was covered with a sandmat of

8000 g/m² mass per unit area. Additionally there are very high hydraulic gradients acting on the GBR-C. Since the thickness of the lining is only 1 cm, the gradient in a canal 4 m deep is more than 400. The experience showed that it is possible to use a GBR-C as an impermeable lining for navigable waterways.

9.4.3 Dams

The position of a waterproofing system is on the upstream face of a dam. The GBRs are typically protected with a facing of, for example, cast-in-place concrete slabs or a soil cover of gravel and random rock fill, if it is an earth dam with adequate slope inclinations. In these cases, the GBRs are protected against UV radiation and against damage from ice, for example. However, there is a need to protect the GBR against puncturing from the bedding layer caused from the high water pressure on the GBR and puncturing from the coarse gravel of the cover layer. Typically, geotextiles are used as protection layers. In many dams, PVC-P geosynthetic barriers are used with thicknesses of 2–3 mm. Thick GBRs provide strength against handling and placement damages and allow high-quality controlled welded seams (Sembenelli *et al.*, 1998). Other GBRs are made of HDPE (Sembenelli *et al.*, 1998) and also heavy bituminous GBRs are used in certain regions (Girard *et al.*, 1998; Gautier *et al.*, 2002).

On old concrete dams with typically vertical facings, GBRs and geocomposites (GCO) are applied for rehabilitation of these dams. In these cases, the GBRs are exposed to all weather conditions without any external protection. Such dams are often located in mountains and are very remarkable elevations with significant UV radiation. The GCOs could consist of a PVC-P GBR and a non-woven geotextile (Cazzuffi, 1998). The advantage of such a GCO is that it allows a continuous fastening along the vertical lines on the concrete and also a horizontal prestressing of the GBR itself. As these GBRs are unprotected, they have to be designed to withstand the action of ice and UV rays, especially when they are exposed to south orientation.

9.5 Tunnelling

Tunnels can be constructed near the surface and also at depth. Flüeler and Böhni (2001) reported studies on long tunnels through the Swiss Alps. Because of the large mountain cover, the GBR-P together with a drainage geocomposite should continuously drain the attracted mountain water, protect the concrete construction against water and locally transfer high compressive loads on to the concrete support structure. The GBR-P is installed between the relatively rough shotcrete outer shell and the concrete inner shell. The drainage geocomposite has to act as protection layer as well. At the base, the large mountain cover of 2500 m also results in rock temperatures of the order of 45 °C owing to geothermal effects. These conditions thus apply to the intruding water that is mostly alkaline, but acidic in some areas. The expected service lifetime is 100 years with no major repairs within 50 years.

The GBR-Ps used are made of TPO, LLDPE, fPP and PVC-P. They have a light signal layer on a black core layer for easy detection of scrapes and damages in the GBR-P caused from improper installation techniques.

9.6 Transportation

Roads or railways through water protection areas require GBRs in order to prevent the migration of pollutants to the groundwater in cases such as accidents of oil or chemicals' transporters. The barrier has in addition to prevent migration of de-icing salt into the underground (Rathmayer, 2002). These jobs will be done by either GBR-Ps or GBR-Cs. Schmidt (1995) reported on a motorway in south Germany where the whole subgrade was covered with a GBR-C. Heerten (1995) described a project on the new Munich airport where GBR-Cs prevent groundwater contamination due to runway de-icers. GBR-Cs have also been used for the sealing of railway substructures of tracks passing through water conservation zones (Göbel *et al.*, 2002). In these cases, the GBRs have to be installed in such a manner that neither puncturing caused from high traffic loads nor freeze–thaw impacts may affect the barrier function of the GBRs.

9.7 Landfills

Waste material may contain substances that can be harmful to the environment. It is therefore mandatory to handle and store waste in such a way that any contamination of the ground as well as of the groundwater is prevented. So, the primary engineering assignment in designing, constructing and operating solid-waste landfills is to provide efficient barriers against contamination. Since water is the most important transporting agent for pollutants, the infiltration of water into and the extraction of water out of the solid-waste body must be controlled by reliable technical means. Liners and landfill covers are the most significant technical members of landfill structures for this purpose. In connection with dewatering facilities and the leachates collection and removal system, the basal liner and the cap seal are crucial elements with respect to landfill safety.

There is a close relationship between sealing and dewatering elements of the basal and of the cover barrier. Drainage facilities must maintain minimum gradients to facilitate gravitational flow. So, to some extent, the dewatering systems dictate the geometry of the surfaces of sealing layers. On the other hand, collection pipes for leachates should be placed in such a way that the unavoidable penetrations through the sealing layers do not impede the efficiency of the liners. These few examples show that the sealing layers and the dewatering elements form integral parts of barrier systems and have to be designed accordingly. They also influence each other during and after construction. Obviously, the placement of drainage gravel above GBR-Ps must be executed with greatest care to avoid perforations of the liner.

9.7.1 Basal lining systems

The landfill containment is sealed at the bottom by a basal lining system composed of several layers, each one serving a particular purpose. It consists of the seal, the protector and the drainage blanket. In order to provide a continuous system of low permeability, the seal is placed directly above the subsoil without a drainage layer in between. The main seal may consist of a single or a double liner, and the liner itself could consist of an impervious monolayer or a composite. For example, a composite liner could be composed of a CCL and a GBR-P.

Since the GBR-P is rather thin and sensitive to mechanical damage, a special protective layer is needed above the GBR-P. This layer can be a geosynthetic product, a soil or a composite of both materials. In order to prevent any build-up of the leachates' pressure head above the sealing layers, a drainage blanket is incorporated in the basal lining system. Finally, it may be necessary to place a transition or filter between the drainage blanket and the waste body to maintain the long-term performance of the drainage system.

Extensive research in Germany by August *et al.* (1992), during the 1980s has led to the conclusion that a composite liner is the most efficient seal against the migration of the harmful components of leachates. Accordingly, the German instructions require such a composite liner at the bottom of municipal solid waste landfills and of hazardous waste landfills as a standard solution.

The polymer component acts as a cut-off for the flow of water. Because of its non-polar molecular structure, it prevents the diffusion of polar substances and therefore it is an absolute barrier against heavy-metal cations. Non-polar molecules of hydrocarbons or chlorinated hydrocarbons that may permeate through the GBR-P are retarded at the surface of the CCL owing to its strongly polar molecular structure. The effect is a decrease in the concentration gradient across the GBR-P and, consequently, a reduction in the rate of permeation. So, it is especially the interface of the GBR-P with the CCL that acts as an efficient barrier against the movement of contaminants such as hydrocarbons, provided that both components of the sealing system are in intimate contact.

The function of the GBR-P in the basal liner system of a solid-waste landfill is to retain leachates, a liquid that may be composed of many different substances, some of which can be harmful. In most cases the composition of the leachates cannot be predicted with sufficient certainty; it may vary with time. The only basis for an assessment of the properties of leachates is chemical analyses carried out on a great number of samples taken at many different landfill sites.

GBR-Ps used for landfill liners should be impervious to all the components found in leachates and also to those that might occur, they should resist chemical and biological attack in the landfill milieu without losing their functional properties and, furthermore, they must be mechanically strong enough to survive transport, handling, placement and subsequent construction activities. The deformation behaviour has to be within acceptable limits, it has to be compatible with the deformations of the other components of the landfill structure, and it has to be predictable. The strength and the interface frictional resistance have to be in agreement with the stability requirements of the landfill structure.

In summary, the GBR-P has to meet a number of requirements concerning its physical, mechanical and endurance properties. Koerner (2005) listed 20 test methods for the determination of the parameters that describe the relevant properties of GBR-Ps. All-important material parameters have to be specified to make sure that the GBR-P is suitable for a landfill liner.

Most European countries have instructions for landfill GBR-Ps. In Germany, an approval system has been installed by legal action. The federal instructions Technische Anleitung Abfall (Federal Ministry for the Environment, Protection of Nature and Safety of Reactors, 1991) and Technische Anleitung Siedlungsabfall (Federal Ministry for the Environment, Protection of Nature and Safety of Reactors, 1993) specify that only approved GBR-Ps shall be used in landfill construction. Experts representing the GBR-P manufacturers, the testing and research institutions, the designers and the regulators agreed upon the approval criteria.

The procedure for the approval of GBR-Ps for landfill applications is executed by the Bundesanstalt für Materialforschung und -prüfung (1999). On the basis of more than 20 years' experience with polyethylene in civil engineering applications and comparative testing of other different GBR-P materials in the laboratories for polymers of Bundesanstalt für Materialforschung und -prüfung (August *et al.*, 1984), it has been decided that only polyethylene should be used in sealing systems of landfills.

The GBR-Ps approved in Germany exhibit excellent chemical resistance and sealing performance against a large variety of substances that could be encountered in the leachates of municipal or hazardous waste landfills. HDPE with a density between 0.932 and 0.942 g/cm³ is used. It must have a carbon black content of 1.8–2.6% and meet a number of strict requirements with respect to physical and chemical properties.

In order to warrant a sufficient robustness of the GBR-P in handling, the specified minimum thickness of approved GBR-Ps is 2.5 mm. This thickness also happens to be very satisfactory with respect to the sealing function. However, HDPE GBR-Ps of 2.5 mm are not very flexible.

The minimum width of the GBR-P roll is 5 m in order to minimize the amount of field seaming needed to create large waterproof sheets. The size and weight of the GBR-P rolls are specified, to make sure that they can be transported to the site and placed without severe handling problems.

The basal lining system includes a drainage blanket of very coarse gravel or crushed rock of typically 16–32 mm grain diameter above the GBR-P liner. Below the waste body with a thickness of tens of metres, and also below moving construction equipment, the coarse grains exert considerable point loads on to the basal sealing layers. In order to avoid perforations of the GBR-P, a special puncture protection is needed.

9.7.2 Cover systems

When the filling process of a solid-waste landfill or of a larger portion of it is completed, the surface of the waste body has to be covered by a cap. The cover system has to prevent the infiltration of rainwater and the emission of odours, dust and gas, and it has to facilitate landscaping and the growth of vegetation. The main components of the cover of a landfill are a regulating soil layer immediately above the waste body, a gas-venting system, the sealing layers, a drainage system and the restoration profile. Depending on the requirements for the different landfill categories, these layers vary to some extent.

The properties and the behaviour of the waste influence the performance of the cap. They have to be taken into account in design and construction. For waste bodies that contain mineral solids that do not undergo chemical or biological reactions, no major long-term settlements are expected. This applies to landfills, which mainly contain ashes from incinerators, and it should apply to hazardous waste as well. For landfills without long-term differential settlements, the placement of the cover can be carried out as soon as the design height is reached.

Common municipal waste landfills are essentially bioreactors, where degradation processes take place in the waste body, associated with significant volume changes and gas production. The surfaces of this type of landfills usually experience large settlements for quite some time. It is likely that also substantial settlement differences occur locally which sometimes cannot be followed by mineral seals without the development of leaks. Since the bioreactors need a certain amount of water to continue the degradation processes, some leakage is probably of no concern. It makes sense to provide municipal waste landfills with CCLs or GBR-Cs, mineral layers of low hydraulic conductivity, as interim covers. Later these interim covers become parts of the final capping systems which contain a GBR-P as the main seal. The GBR-P should be placed when most of the anticipated differential settlements have occurred. To determine the right time for this action the deformation of the interim cover surface should be monitored.

As an alternative to a CCL, a GBR-C can be installed. The question of the equivalency of GBR-Cs and CCLs has been discussed by Koerner and Daniel (1995) and by Stief (1995) among others. The properties, testing methods and quality assurance aspects of GBR-Cs were compiled by Gartung and Zanzinger (1998). Practical experience shows that GBR-Cs as members of capping systems have some advantages over CCLs. Handling and installation are much easier, less time is needed for placement, waste storage space can be saved owing to the smaller thickness and the quality of the manufactured geosynthetic product shows less scatter than of the natural clay soils.

On the other hand, it has to be kept in mind that because of their small thickness and small mass of bentonite they are extremely sensitive to damage during and after construction. So great care has to be taken in construction with GBR-Cs. The design of a GBR with GBR-Cs has to consider that desiccation of GBR-Cs has to be avoided because of the fact that dried-out sodium bentonite, for example, will exchange cations, and so sodium bentonite will change to calcium bentonite with much poorer swelling properties than those of sodium bentonite. In some projects in Germany, it was found that under certain conditions with the inadequate protection of the GBR-Cs against desiccation the barrier function of the GBR-Cs was lost (Gartung and Zanzinger, 1998).

Fissuring and growth of roots in mineral seals of landfill capping systems can be prevented by the placement of a GBR-P. A GBR-P can function as a barrier against root penetration as well as against moisture migration. The final sealing layers of cover systems of landfills should consist of the combination of CCLs or GBR-Cs with GBR-Ps. Since the seal at the top of the landfill is not acted upon by chemicals, the synergistic composite effect of polymers and clay soils that facilitates the retention of polar as well as non-polar substances at the bottom of the landfill does not become effective in the capping system. So at the cover the two components do not really act as a composite but rather as a double liner.

Even though GBR-Ps of covers are not exposed to a corrosive chemical environment, the same types of GBR-P could be used for caps as for basal liners. The advantages are high robustness and reliable quality. Their limited flexibility is of some disadvantage. The installation of GBR-Ps of softer polymers such as LDPE would be more favourable with respect to the anticipated deformations of the landfill surface.

The construction requirements and installation techniques are essentially the same for GBR-Ps of the cover as of the bottom liner. The seaming technique and all details of construction quality control and construction quality assurance described for basal liners, apply to covers as well.

The surface of the landfill or of the regulating layer has to be modelled to a shape that allows plane GBR-Ps to be spread without distortions. This design requirement is especially important when HDPE membranes of 2.5 mm thickness are used. It is impossible to place them on three-dimensionally curved surfaces with small diameters of curvature.

Usually landfills are hills with sloping surfaces. So slope stability is a very important issue in designing and constructing landfill covers. Often it is not possible to mobilize enough shear resistance for stability on smooth GBR-P surfaces. Then GBR-Ps with specially structured rough surfaces are used in cover construction. These structured GBR-Ps undergo the same stringent suitability tests as the smooth GBR-Ps for the basal liner do. Particular attention is paid to their long-term tensile strength and stress cracking resistance. In order to avoid tensile forces in the GBR-P, the mobilized friction at the lower surface of the GBR-P should be greater than at the upper surface. If the slope stability analysis leads to the conclusion that a sufficient safety in the balance of forces can only be reached by additional reinforcing elements, geogrids are placed above the sealing layers of the capping system.

9.8 Construction of geosynthetic barriers

9.8.1 Preparations

The construction materials of the composite basal liner, namely clay soils and GBR-Ps, differ greatly in their material properties. While the mineral component can follow any three-dimensional geometrical feature as long as it can be shaped by earth-moving and compaction equipment, GBR-Ps are plane elements. That is why the subbase has to be designed such that the GBR-P can be spread evenly without distortions. Therefore, in the ideal situation, the surface of the soil consists of only planes that intersect at straight lines.

9.8.2 General aspects of installation

The construction personnel must be highly quality minded. It has to be taken into account that all construction operations at a site are sensitive to weather conditions. The installation of GBR-Ps requires favourable weather. It cannot be carried out in the rain. The minimum temperature for seaming polyethylene sheets is 5 °C. Sufficient time has to be allocated to the placement of GBR-Ps to cope with unavoidable delays arising because of the unfavourable weather that occurs frequently in many parts of Europe.

The manufacturer of the GBR-P must establish his own instructions for handling and installation. If the construction work is not executed by the manufacturer, it must be subcontracted to a specialist. The GBR-P manufacturers list authorized firms for the placement and seaming of the specified GBR-Ps. The construction personnel must be qualified by education and experience. The technicians must be certified welders. The seaming methods to be applied are specified in the approval documents. Strict rules are to be followed for the execution of the construction work.

9.8.3 Placement of geosynthetic barriers

The GBR-P has to be placed without any voids trapped between it and the soil surface. So, ideally, the spread GBR-P should not exhibit any waves. This is very difficult to achieve in practice. In particular, when the weather is fine and sunny, the black polyethylene membrane heats up owing to its high coefficient of thermal expansion. The formation of waves in the GBR-P cannot be avoided under such conditions. However, at night, when the sun disappears and the air temperature drops, the GBR-P will contract, and the waves will disappear. This physical effect is used systematically. Schicketanz (1992) has developed great expertise in a technology for the placement of GBR-Ps that follows the daily rhythm of temperatures at the construction site.

9.8.4 Welding of polymeric GBRs

Double hot-wedge and extrusion methods are mostly required. The quality of welding work essentially depends on the skills and knowledge of the welder; in landfills, only certified welders should be employed (Corbet and Peters, 1996). Before the welders are allowed to start their work on a landfill construction site, the third-party inspector must satisfy himself that the welders are capable of producing a satisfactory seam with the planned GBR-P's welding machinery and equipment under the conditions of construction site. It is recommended that welding machines that document continuously the welding parameters (the hot-wedge and/or hot-gas temperatures, the force of the rolls and the speed) are used for the fusion welds. The various ambient conditions such as atmospheric humidity, air and GBR-P temperature should also be recorded. The seams are to be tested regarding the external appearance, dimensions, strength and tightness. The visual inspection of the seams and the continuity tests (vacuum, high voltage or compressed air) must be made continuously, and the dimensions and strength tests on random samples. The compressed-air test is used for imperviousness testing of welded seams with a test channel under a defined mechanical stress of 5 bar for 10 min without noticeable loss of pressure. In areas that are not accessible for the automatic hot wedge-welding machine, e.g. sump bottoms, pipe penetrations or patches, the extrusion fillet method is applied. It requires a great deal of good craftsmanship to reach the same quality as the automatic dual hot-wedge fusion technique.

9.8.5 Quality assurance

All personnel responsible for quality management must be experienced in construction with geosynthetics. Only GBR-Ps without any visible flaws are accepted. Experience shows that the quality of GBR-Ps manufactured under a quality assurance system such as that approved in Germany is generally very good. If a GBR-P roll has to be rejected upon delivery at the construction site, the objections are usually due to damage that occurred during loading, transport or unloading. Sometimes the action of unloading the bulky heavy GBR-P rolls from their shipping containers is very tricky, and exercise is needed for the personnel to handle GBR-P rolls successfully without damage.

The quality inspectors have to check whether the seaming equipment is suitable for the job, and whether it functions properly. In particular, the generator for electric power has to meet the demand of the welding operations to warrant uniform seams. Every day at the beginning and at the end of the seaming work, the controlling parameters of the seaming machine have to be determined by a test strip. The geometry is checked; the seam is examined visually and by peel tests. Once the welding parameters have been established for the day, the work proceeds at a rather constant rate. If the weather conditions change, adjustments have to be made on the basis of new test strips. The seaming machine records the data on advance rate, temperature and pressure automatically and occasional checks are made by the inspector.

Experience shows, that a reliable execution of the construction quality assurance programme is of utmost importance for the GBR-P. Even though the education of the installers is generally very good and, although the construction personnel are aware of the importance of their work, mistakes do occur. Fortunately, they are detected in time and can be corrected without delay, provided that the construction quality assurance personnel are at the site continuously. Sometimes, owners of landfills do not recognize the necessity for the external inspector to be present at the site during the entire period of GBR-P installation, and they order only occasional visits. It is likely that, in such cases, the savings in expenses for the presence of the external supervisor will be more than compensated by extra expenses for corrections and for delays due to deficiencies noticed at a later time.

9.9 References

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10

The use of geosynthetics to improve the performance of foundations in civil engineering

J. G. COLLIN
The Collin Group Ltd, USA

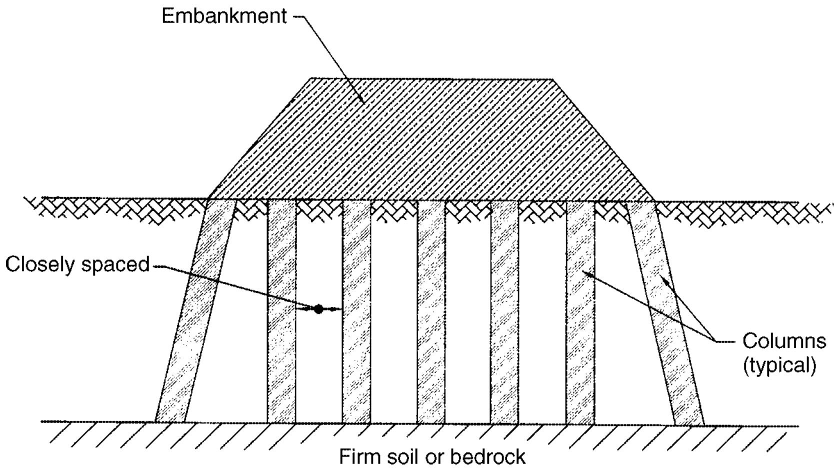
10.1 Introduction

The use of geosynthetics to improve the performance of foundations when constructing on soft compressible foundation soils has evolved considerably over the last two decades. Geosynthetics are being used for the following applications: embankments over soft soils, column-supported embankments (CSEs), shallow foundations constructed over reinforced soil, and bridging voids in the subsurface or roadway shoulders. The mechanism for soil improvement can be as simple as separating native soils from fills or can include tension membrane, soil arching and alteration of failure surfaces. This chapter will focus on two of these applications, namely column-supported embankments and shallow foundations, providing a brief overview of these applications and the research associated with the development of design procedures presented.

The design suggestions presented in this chapter cover the key design issues and the current state of knowledge and state of practice for their use (based on US practice). The state of practice varies considerably across the world and a discussion of this variation is beyond the scope of this chapter. As with any emerging technology, there are still gaps in our knowledge with respect to certain aspects of the design. These gaps will be presented and briefly discussed. However, only with further research will we be able to fill in the gaps in our knowledge.

10.2 Column-supported embankments

The problems associated with constructing highway embankments over soft compressible soil (i.e. large settlements, embankment stability and the long period of time required for consolidation of the foundation soil) have led to the development and/or extensive use of many of the ground improvement techniques in use. Wick drains, surcharge loading, geosynthetic reinforcement, stone columns, deep soil mixing and vibroconcrete columns (VCCs) have all been used to solve the



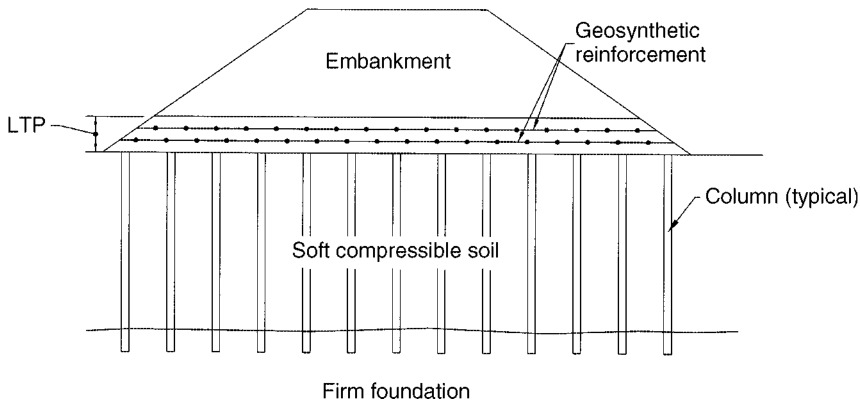
10.1 Conventional CSE (Han, 2003).

settlement and embankment stability issues associated with construction on marginal soils. However, when time constraints are critical to the success of the project, owners have resorted to another innovative approach: CSEs reinforced with geosynthetic reinforcement. In the last 15 years, this technology has been used successfully on numerous projects both in the USA and abroad.

CSEs consist of vertical columns that are designed to transfer the load of the embankment through the soft compressible soil layer to a firm foundation. The selection of the type of column used for the CSE will depend on the design loads, constructability of the column, cost, etc. The load from the embankment must be effectively transferred to the columns to prevent punching of the columns through the embankment fill which causes differential settlement at the surface of the embankment. If the columns are placed close enough together, soil arching will occur and the load will be transferred to the columns. Figure 10.1 shows a conventional CSE. The columns are spaced relatively close together, and some battered columns are required at the sides of the embankment to prevent lateral spreading. In order to minimize the number of columns required to support the embankment and to increase the efficiency of the design, a geosynthetically reinforced load transfer platform (LTP) may be used. The load transfer platform consists of one or more layers of geosynthetic reinforcement placed between the top of the columns and the bottom of the embankment. Figure 10.2 shows schematically a CSE with geosynthetic reinforcement.

10.2.1 Historical overview

The first documented use of a CSE with geosynthetic reinforcement was in 1984 for a bridge approach embankment in Europe (Reid and Buchanan, 1984). Concrete



10.2 CSE with geosynthetic reinforcement (Han, 2003).

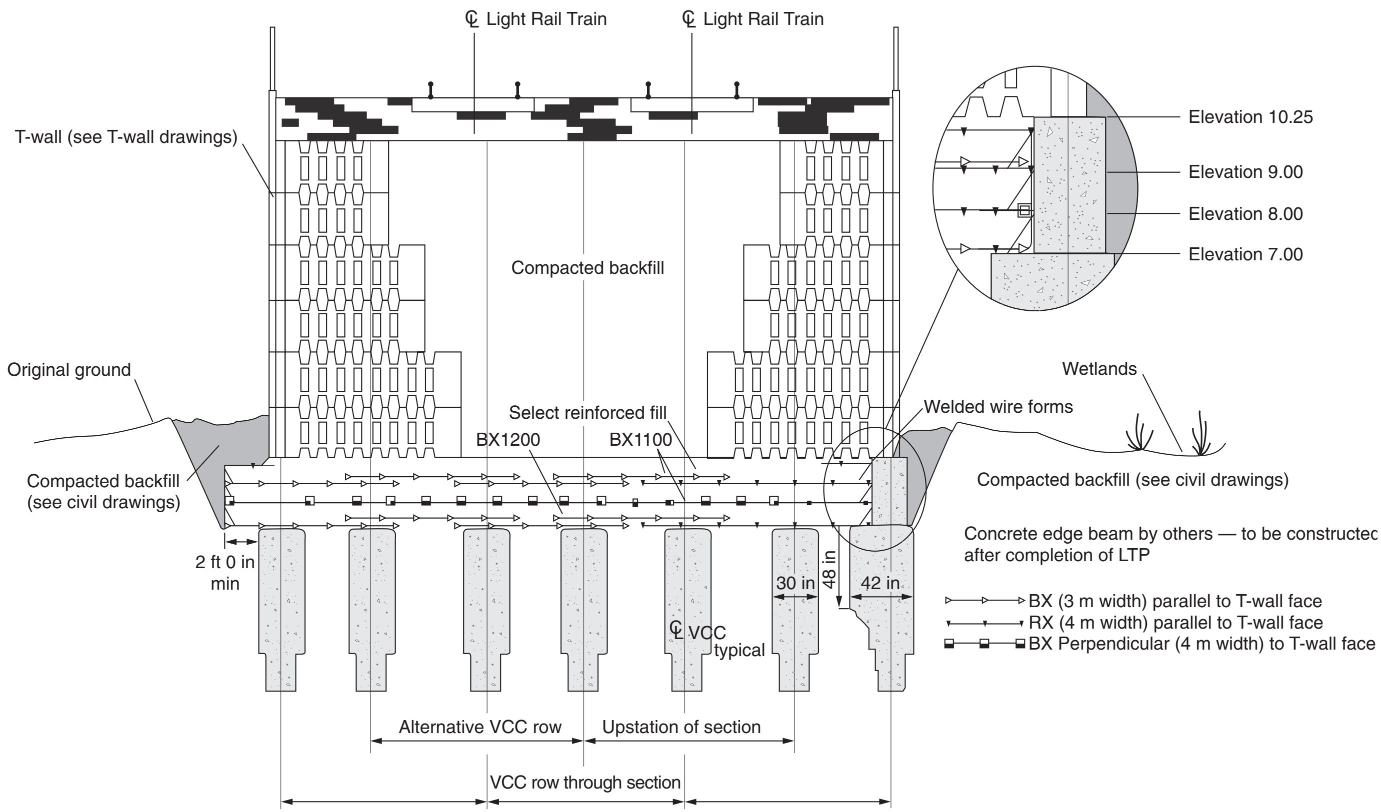
piles were used as the columns for the project. Each column had a reinforced concrete pile cap. The clear span between pile caps varied from 2 to 3 m (from 6.6 to 10 ft). One layer of geosynthetic reinforcement was used to create the LTP. The height of the embankment was 9 m (29.5 ft).

The first application of a CSE with geosynthetic reinforcement in the USA was in 1994 for the Westway Terminal in Philadelphia, Pennsylvania. This project involved the support of a large-diameter tank for the storage of molasses. The foundation consisted of VCCs and an LTP. The platform consisted of a well-graded granular fill, reinforced with three layers of geogrid reinforcement. The CSE was selected over a more conventional pile foundation with a concrete mat because of both time and money savings.

One of the first (2001) transportation related projects in the USA to use a CSE was for an embankment over soft soils, at a river crossing, for the New Jersey Light Rail (Young *et al.*, 2003). The foundation for the embankment consisted of VCCs and an LTP. The VCCs were placed on a 2.3–3.0 m (6.6–9.8 ft) center-to-centre triangular spacing. The platform was 1 m (3.3 ft) thick and was reinforced with three layers of geogrid. A well-graded granular soil was used as structural fill for the platform. Figure 10.3 shows a typical cross-section of the project. The CSE was selected for this project to eliminate the 'bump' at the end of the bridge without having to wait for the foundation soil to consolidate.

The use of a CSE with geosynthetic reinforcement has increased dramatically in the last decade both in the USA and abroad. More than 20 case histories are now available in the literature documenting the use of this technology (Han, 1999).

There is a wide range of columns that may be used for CSE. Conventional (i.e. timber, steel H, steel pipe, precast concrete and cast-in-place concrete shell) piles may be used for pile-supported embankments. However, conventional piles, with the exception of timber piles, have a rather high structural capacity (i.e. 400–2000 kN (90–450 klbf)) that is seldom required for CSE and are, therefore, economically



10.3 New Jersey Light Rail project (Young *et al.*, 2003).

not as attractive as non-traditional columns. Augered piles have been used in the USA and Europe with success.

The newer elements that have been used for columns in CSEs include soil mix columns, stone columns, geotextile-encased columns (GECs), geopier-rammed aggregate piers and VCCs.

Combined soil stabilization with vertical columns (CSV) is a new technology that has recently been brought to the USA from Germany. The columns are constructed by introducing dry granular material into the soft foundation soil by an auger, which has a compaction head attached at its tip. The auger rotates in the opposite direction to the pitch of the flights. Soil is, therefore, not removed during the drilling operation, but rather compacted around the auger. As the auger advances into the ground, it compacts the soil around the auger. During withdrawal of the auger, dry granular material (typically a sand cement mix) is transported to the tip of the auger and compacted. CSV columns have typical diameters of 150–200 mm (6–8 in) and have a capacity of 45–90 kN (10–20 kips).

The LTP transfers the embankment load to the columns. Two types of LTPs are available. A reinforced concrete structural mat may be used to transfer the embankment load to the columns. This requires a structural design of the mat to assure that the load is effectively transferred to the columns. Concrete mats have generally been found to be economically cost prohibitive and will not be discussed further.

The second type of LTP consists of one or more layers of geosynthetic reinforcement and select backfill to create a system to transfer the embankment load to the foundation columns, as shown in Fig. 10.2.

10.3 Advantages and disadvantages of column-supported embankments

10.3.1 Advantages

CSEs provide a technical and potentially economical alternative to more conventional construction techniques (i.e. surcharge loading, wick drains and staged construction with or without geosynthetic reinforcement). The key advantage to CSEs is that construction may proceed rapidly in one stage. There is no waiting time for dissipation of pore water pressure in the soft foundation soil. CSEs are also more economical than the removal and replacement of deep poor bearing soils, particularly on larger sites where the groundwater is close to the surface. Where the infrastructure precludes high-vibration techniques, the type of column used for the CSE system may be selected to minimize or eliminate the potential for vibrations. Total and differential settlement of the embankment may be drastically reduced when using CSEs over conventional approaches.

One major benefit of CSE technology is that it is not limited to any one column type. If contaminated soils are anticipated at a site, the column type may be selected so that there are no spoils from the installation process. If very soft soil is anticipated, VCCs, GECs, augered piles or timber piles may be selected as the column type for the project. In stronger foundation soils, stone columns or rammed aggregate piers may be economically more attractive. The designer has the flexibility to select the most appropriate column for the project.

10.3.2 Disadvantages

A major disadvantage of CSEs is often the initial construction cost when compared with other solutions. However, if the time savings when using CSE technology is included in the economic analysis, the cost may be far less than other solutions.

Another major disadvantage is that there is currently no single well-accepted design procedure. There are many different design approaches, and they all give different results. Without some standardization of the LTP design, the technology will be limited in its use and acceptance.

10.4 Feasibility evaluations

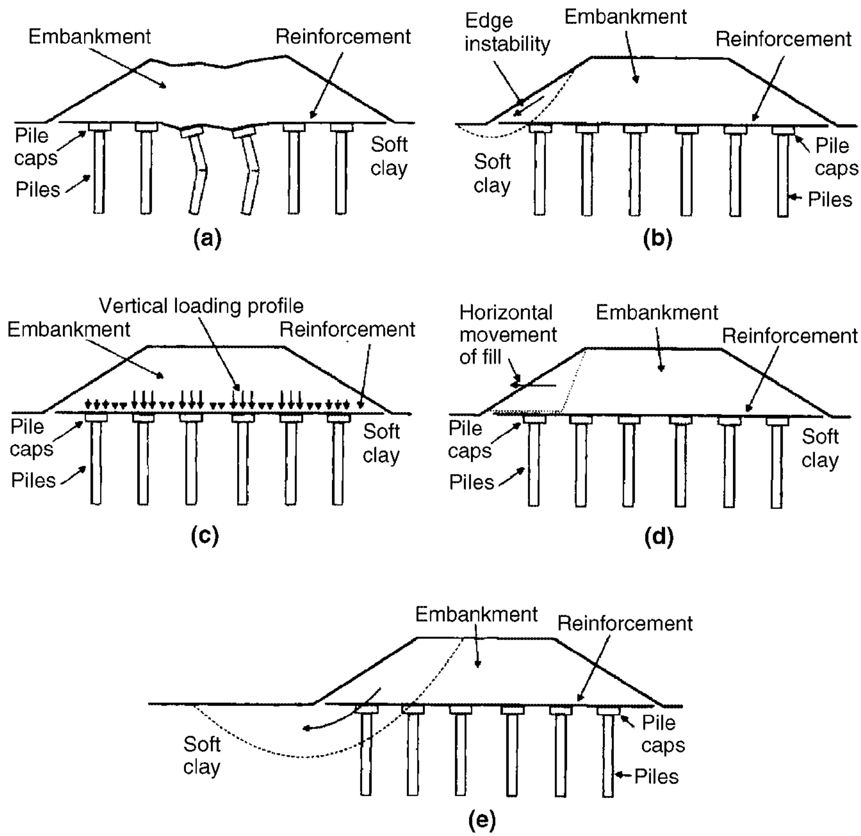
CSEs may be used whenever an embankment must be constructed on soft compressible soil. To date, the technology has been limited to embankment heights in the range of 10 m. The depth of the soft soil layer is not a critical component in the determination of feasibility because of the many different types of column available for use.

A generalized summary of the factors that should be considered when assessing the feasibility of utilizing CSE technology on a project is presented below. This summary is empirically based on the present author's experience with the design and successful construction of over 20 CSE projects in the USA.

1 The clear span between columns should be less than the embankment height and should not exceed 3 m (10 ft). This requirement is based on documented case histories. Wider clear spans may lead to unacceptable differential settlement between columns.
2 The fill required to create the LTP shall be a select structural fill with an effective friction angle greater than or equal to 35°.
3 The columns shall be designed to carry the entire load of the embankment.
4 CSE technology reduces-post construction settlements of the embankment surface to typically less than 50–100 mm (2–4 in).

10.5 Design concepts

The design of CSEs is a complex soil–structure interaction problem. There are currently several empirical methods for the design that focus predominantly on the analysis of the load transfer platform. The methods that will be presented in this

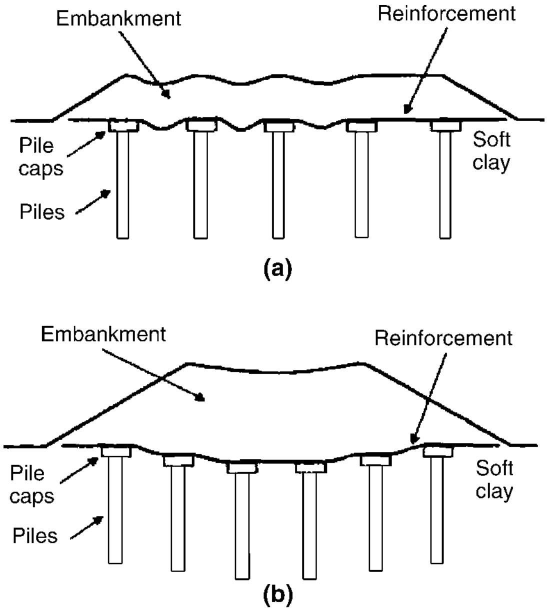


10.4 Limit state failure modes (British Standards Institution, 1995): (a) pile group capacity; (b) pile group extent; (c) vertical load shedding; (d) lateral sliding; (e) overall stability.

chapter include the British Standard BS 8006 (British Standards Institution, 1995), the Swedish method, the German method and the Collin method. Each approach has been used successfully on numerous projects and, through those projects, each method has been demonstrated to work well.

10.5.1 Fundamental concepts

The design of CSEs must consider both the limit state and serviceability state failure criteria. The limit state failure modes are shown in Fig. 10.4. The columns must be designed to carry the vertical load from the embankment without failing [Fig. 10.4(a)]. The columns are typically assumed to carry the full load from the embankment. The lateral extent of the columns under the embankment must be determined [Fig. 10.4(b)]. The LTP must be designed to transfer the vertical load from the embankment to the columns [Fig. 10.4(c)]. The potential for lateral



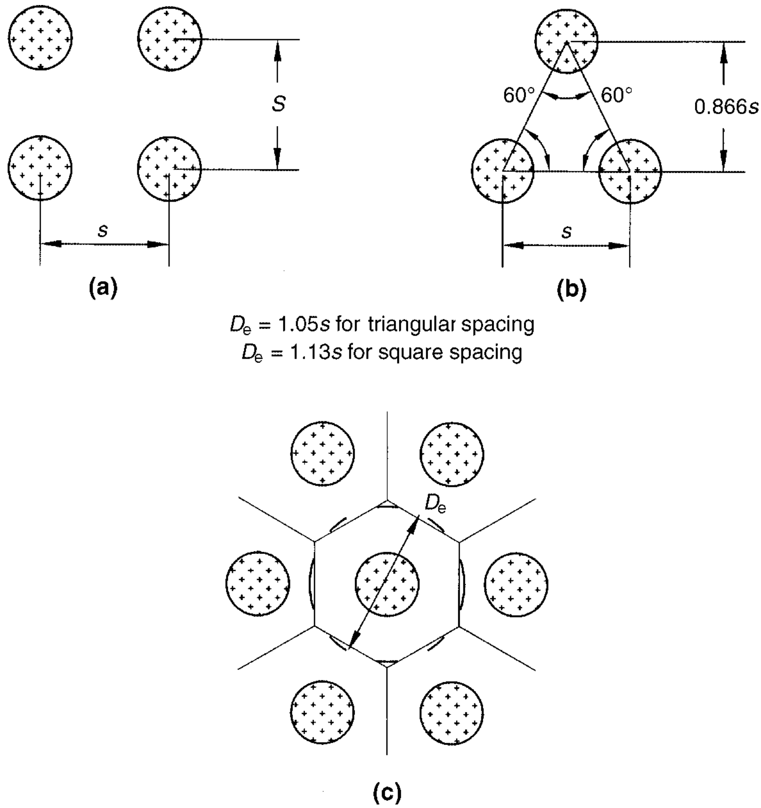
10.5 Serviceability state (British Standards Institution, 1995): (a) reinforcement strain; (b) foundation settlement.

sliding of the embankment on top of the columns must be addressed [Fig. 10.4(d)]. Finally, global stability of the system must be evaluated [Fig. 10.4(e)].

In addition to a limit state analysis, serviceability state design must be considered. The strain in the geosynthetic reinforcement used to create the LTP should be kept below some maximum threshold to preclude unacceptable deformation reflection (i.e. differential settlement) at the top of the embankment. Settlement of the columns must also be analysed to assure that unacceptable settlement of the overall system does not occur, as shown in Fig. 10.5.

The general design steps for a CSE are as follows:

Step 1 Estimate the preliminary column spacing (use feasibility assessment guidelines).
Step 2 Determine the required column load.
Step 3 Select preliminary column type based on the required column load and site geotechnical requirements.
Step 4 Determine the capacity of the column to satisfy the limit and serviceability state design requirements.
Step 5 Determine the extent of columns required across the embankment width.
Step 6 Select the LTP design approach (i.e. catenary or beam).
Step 7 Determine the reinforcement requirements based on the estimated column spacing (Step 1). Revise the column spacing as required.
Step 8 Determine the reinforcement requirements for lateral spreading.



10.6 Column layout: (a) square spacing; (b) triangular spacing; (c) effective diameter.

Step 9 Determine the overall reinforcement requirements based on LTP and lateral spreading.
Step 10 Check the global stability.
Step 11 Prepare the construction drawings and specifications.

10.5.2 Column design

The selection of column type is most often based on constructability, load capacity, and cost. The load that a column is required to carry is typically based on the tributary area for each column. In the USA, the embankment and any surcharge load are typically assumed to be carried in their entirety by the columns.

For purposes of determining the design vertical load in the column, it is convenient to associate the tributary area of soil surrounding each column, as illustrated in Fig. 10.6. Although the tributary area forms a regular hexagon about the column, it can be closely approximated as an equivalent circle having the same total area. For a square column pattern, the effective diameter D_e is equal to 1.13 times the centre-to-centre column spacing. For a triangular column pattern, the effective diameter is equal to 1.05 times the centre-to-centre column spacing [a typical centre-to-centre column spacing ranges from 1.5 to 3.0 m (from 5 to 10 ft)].

The required design vertical load Q_r in the column is determined according to:

$$Q_r = \pi\left(\frac{D_e}{2}\right)^2(\gamma H + q) \quad [10.1]$$

where:

D_e = effective tributary area of column
H = height of embankment
q = live and dead load surcharge (typically 12 kN/m^2 (250 lbf/ft^2))
γ = unit weight of the embankment soil.

The design of concrete, steel and timber piling is well established. Design guidelines have been developed by the Federal Highway Administration (FHWA) and may be found in *Design and Construction of Driven Pile Foundations* (Hannigan *et al.*, 1998). For the design of timber piles, the reader is also referred to the book by Collin (2002). The design and construction of micropiles has been provided by Armour *et al.* (2000).

The vertical load capacity design of VCC, and CSV is not well defined and is typically performed by the contractor. The design verification for these systems is typically achieved with a static load test.

10.5.3 Edge stability; lateral extent of columns

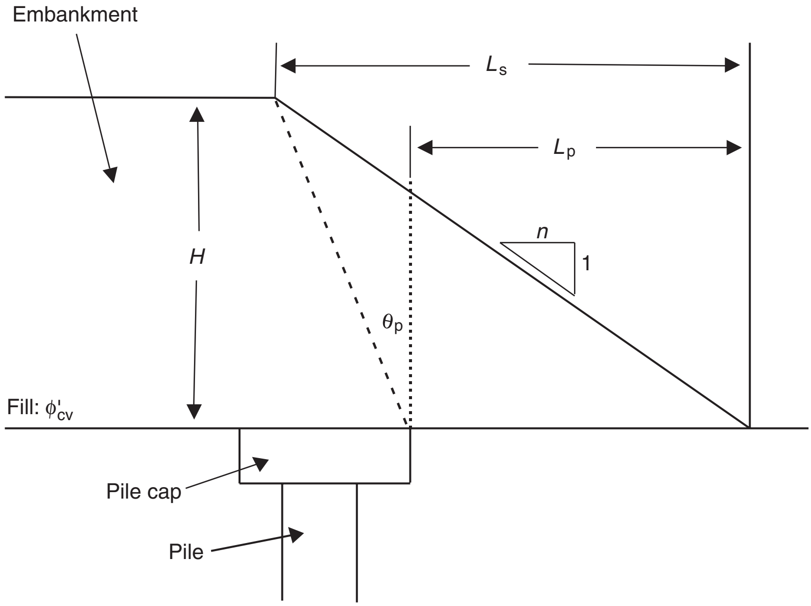
The lateral extent of the column system across the width of the embankment should extend a sufficient distance beyond the edge of the embankment to ensure that any instability or differential settlement that occurs outside the column-supported area will not affect the embankment crest [Fig. 10.4(b)]. There are several approaches that may be used to check the edge stability. The computer software ReSSA developed for the FHWA for the design of both reinforced and non-reinforced slopes and embankments is an excellent tool for checking edge stability.

BS 8006 (British Standards Institution, 1995) requires that the columns extend to within a minimum distance L_p of the toe of the embankment. Figure 10.7 defines the terms for edge stability. L_p is determined from:

$$L_p = H\,(n - \tan\theta_p) \quad [10.2]$$

where:

n = side slope of the embankment



10.7 Edge stability (British Standards Institution, 1995).

θ_p = is the angle (from vertical) between the outer edge of the outer-most column and the crest of the embankment ($\theta_p = 45 - \varphi_{emb}/2$).
φ_{emb} = effective friction angle of embankment fill.

The British method is an excellent check of the more rigorous stability analysis using limit equilibrium techniques (i.e. ReSSA). For preliminary designs and/or feasibility analysis, the simplified British approach is sufficient.

10.5.4 Lateral spreading

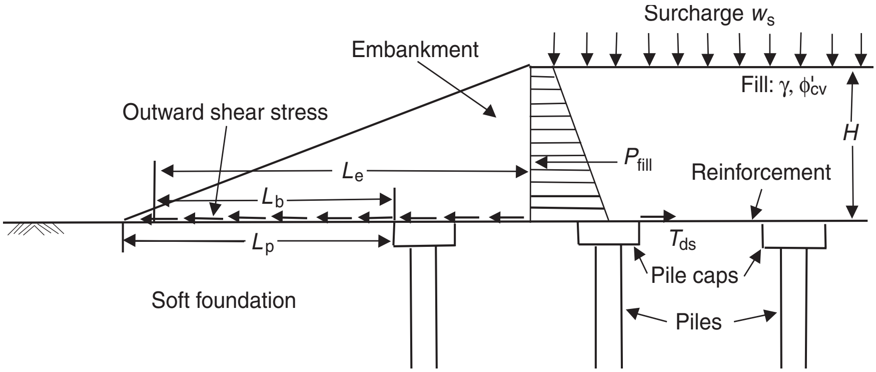
The potential for lateral spreading of the embankment must be analysed (Fig. 10.8). The geosynthetic reinforcement must be designed to prevent lateral spreading of the embankment. This is a critical aspect of the design, as many of the columns that are appropriate for CSEs are not capable of providing adequate lateral resistance to prevent spreading of the embankment without failing.

The geosynthetic reinforcement must be designed to resist the horizontal force due to the lateral spreading of the embankment. The required tensile force T_{ls} to prevent lateral spreading is determined from:

$$T_{ls} = \frac{K_a(\gamma H + q)H}{2} \qquad [10.3]$$

where:

K_a = coefficient of active earth pressure ($\tan^2(45 - \varphi_{emb}/2)$)



10.8 Lateral spreading (British Standards Institution, 1995).

The minimum length L_e of reinforcement necessary to develop the required strength of the reinforcement without the side slope of the embankment sliding across the reinforcement is determined using:

$$L_e = \frac{T_{ls}}{0.5\gamma H(c_{iemb} \tan \varphi_{emb})} \qquad [10.4]$$

where:

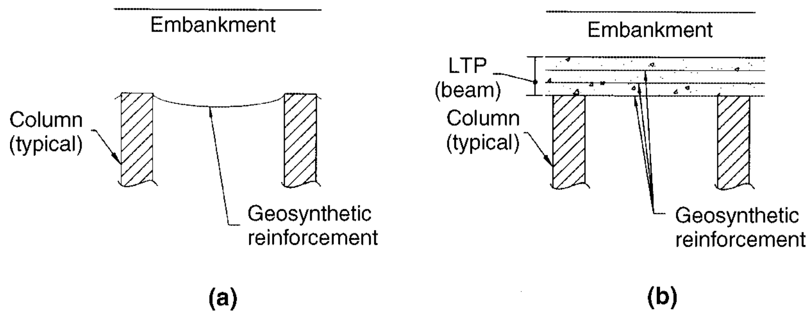
c_{iemb} = coefficient of interaction for sliding between the geosynthetic reinforcement and embankment fill.

10.5.5 Load transfer platform design

There are two fundamentally different approaches to the design of the LTP. The first approach, which is used by BS8006 (British Standards Institution, 1995), the Swedish method (Rogbeck *et al.*, 1998, 2002), and the German method (Alexiew and Gartung, 1999; Alexiew, 2003) is for the reinforcement to act as a catenary. The reinforcement transfers the load from the embankment fill to the columns through catenary tension in the reinforcement, as shown in Fig. 10.9. In essence, the reinforcement behaves as a structural element, and any benefits achieved by the creation of a composite-reinforced soil mass are ignored. The primary assumptions in the catenary theory are as follows:

1 A soil arch forms in the embankment.
2 Reinforcement is deformed during loading.
3 One layer of reinforcement is used; if more than one layer of reinforcement is used, only the tensile strength of the multiple layers is considered.

The second approach for the design of the LTP (the Collin method) is to use



10.9 Load transfer mechanisms: (a) catenary theory; (b) beam theory.

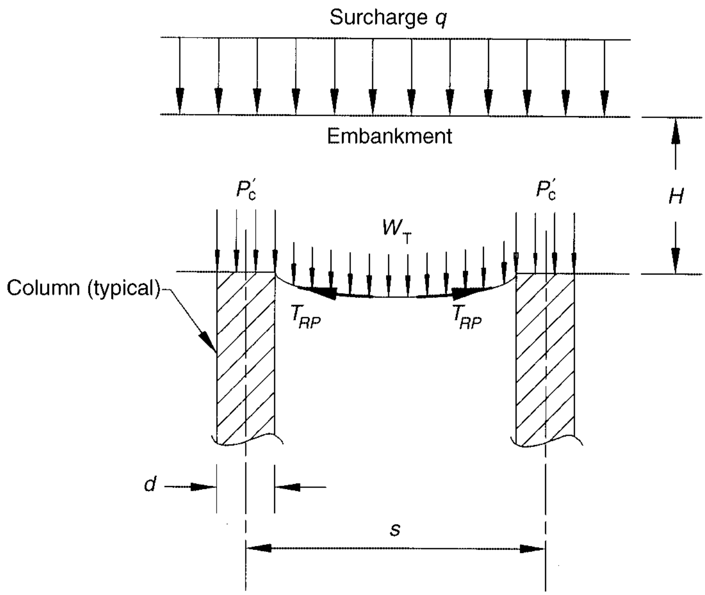
multiple layers of reinforcement to create a stiff reinforced soil mass. The Collin method is a refinement of a method sometimes referred to as the Guido method (Hewlett and Randolph, 1988; Bell *et al.*, 1994; Jenner *et al.*, 1998). The reinforced soil mass acts as a beam to transfer the load from the embankment above the platform to the columns below. The primary assumptions for the beam theory are as follows:

1 A minimum of three layers of reinforcement is used to create the platform.
2 Spacing between layers of reinforcement is 200–450 mm (8–18 in).
3 Platform thickness is greater than or equal to one half of the clear span between columns.
4 A soil arch is fully developed within the depth of the platform.

The catenary method generally requires higher strength reinforcement for the same design conditions, as opposed to the beam method (i.e. column spacing and embankment height). The beam method will generally allow for larger column-to-column spacing than the catenary method for standard geosythetics (i.e. materials available off the shelf).

All LTP design methods covered in this chapter consider soil arching. Soil arching is defined by McNulty (1965) as ‘the ability of material to transfer loads from one location to another in response to a relative displacement between locations’. Terzaghi and Peck (1967) explained arching theory using the ‘trapdoor’ analogy. If a dry cohesionless sand is placed on a platform that contains a trapdoor and the trapdoor is mounted on a scale, then, according to Terzaghi and Peck (1967): ‘As long as the trapdoor occupies its original position, the pressure on the trapdoor as well as that on the adjoining platform is equal to γH per unit area. However, as soon as the trapdoor is allowed to yield in a downward direction, the pressure on the door decreases to a small fraction of its initial value, whereas the pressure on the adjoining parts of the platform increase.’ This phenomenon of pressure transfer is known as ‘arching’.

In addition to soil arching, the LTP design includes tension membrane theory. The vertical load from the soil within the arch and any surcharge load, if the



10.10 Definition of terms.

thickness of the embankment is not great enough to develop the full arch, is carried by the reinforcement. There are several theories available to estimate the tension in the reinforcement (Fluet *et al.*, 1986; Giroud *et al.*, 1990). A detailed discussion on tension membrane theory is beyond the scope of chapter.

The symbols used by the BS 8006, the Swedish standard, the German method, and the Collin method have been standardized for ease of reference. Figure 10.10 shows the common symbols that will be used in presenting these methods. They are defined as follows:

d = diameter of the column
H = height of embankment
P'_c = vertical stress on the column
q = surcharge load
s = centre-to-centre column spacing
T_{RP} = tension in the extensible reinforcement
W_T = vertical load carried by the reinforcement.

BS 8006

The British Standard BS8600 (British Standards Institution, 1995) recommends that the embankment height be a minimum of 1.4 times the clear span between columns. This is to ensure that differential settlement cannot occur at the surface of the embankment. Soil arching between adjacent columns induces greater vertical stresses on the columns than on the surrounding foundation soil. The ratio of the vertical stress on the columns to the average vertical stress at the base of the embankment is determined from the following equation and is based on the formula given by Marston and Anderson (1913):

$$\frac{P'_c}{\sigma'_v} = \left(\frac{C_c d}{H}\right)^2 \qquad [10.5]$$

where:

P'_c = vertical stress on the column
σ'_v = the average vertical stress at the base of the embankment equal to $f_{fs}\,\gamma H + f_q\, q$
f_{fs} = partial soil unit mass load factor (equal to 1.3)
f_q = partial surcharge load factor (equal to 1.3)
C_c = arching coefficient, equal to $1.95H/d - 0.18$ for end bearing columns (unyielding), and to $1.50H/d - 0.07$ for frictional columns (normal)
d = column diameter.

The vertical load carried by the reinforcement spanning between columns for the case where H > 1.4 (s – d) may be determined as follows:

$$W_T = \frac{1.4\, s f_{fs}\, \gamma (s-d)}{s^2 - d^2}\left(s^2 - d^2\, \frac{P'_c}{\sigma'_v}\right) \qquad [10.6]$$

where:

s = centre-to-centre spacing between columns.

For the case where $0.7(s-d) < H < 1.4\,(s-d)$, the distributed vertical load carried by the reinforcement is determined from

$$W_T = \frac{s f_{fs}\, \gamma H + f_q\, q}{s^2 - d^2}\left(s^2 - d^2\, \frac{P'_c}{\sigma'_v}\right) \qquad [10.7]$$

The tension T_{rp} in the extensible reinforcement per lineal metre of reinforcement resulting from the distributed load is:

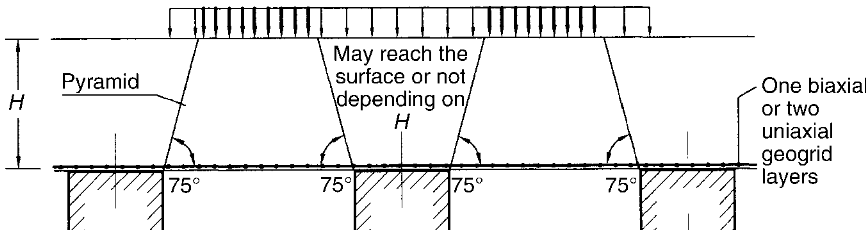
$$T_{rp} = 0.5 W_T\, \frac{s-d}{d}\left(1 + \frac{1}{6\varepsilon}\right)^{0.5} \qquad [10.8]$$

where:

ε = strain in the reinforcement.

The initial tensile strain in the reinforcement is needed to generate a tensile load. BS 8006 recommends that a practical upper limit of 6% strain be imposed to ensure all embankment loads are transferred to the piles.

The tensile load T_{rp} develops as the reinforcement deforms under the weight of the embankment. This normally occurs during construction of the embankment



10.11 Swedish method soil arch (Rogbeck *et al.*, 2002).

but, in situations where the reinforcement cannot deform during construction, the reinforcement will not carry the applied loads until the foundation settles. The above equation is appropriate for those reinforcements that can undergo deformation during loading (i.e. extensible reinforcements). For inextensible reinforcements, alternative relationships should be used to determine their required strength.

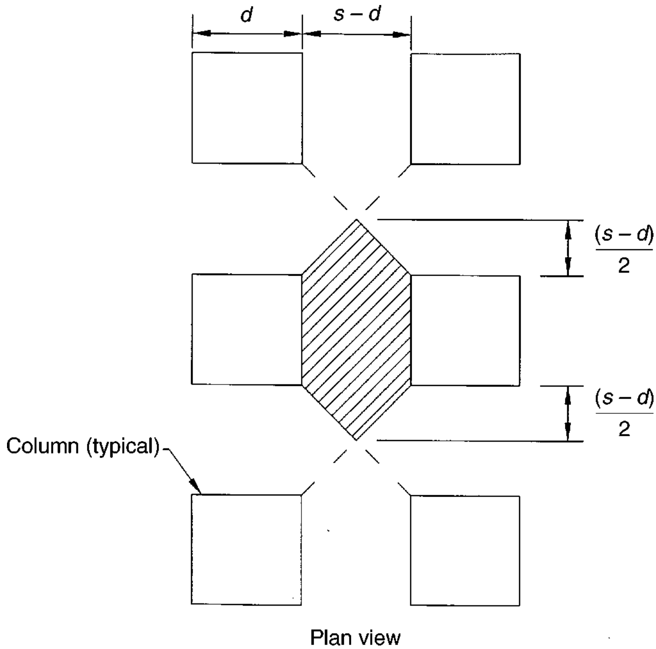
The long-term strain in the reinforcement (due to creep) should be kept to a minimum to ensure that the long-term localized deformations do not occur at the surface of the embankment. A minimum creep strain of 2% over the design life of the reinforcement should be allowed.

The Swedish method

The Swedish method (Rogbeck *et al.*, 2002) has many similarities to the BS 8006. The Swedish method is valid when the following assumptions and parameters are satisfied.

1. Arch formation occurs.
2. The reinforcement is deformed during loading.
3. One layer of reinforcement is used.
4. The reinforcement is located within 0.1 m (4 in) above the column.
5. The embankment height is greater than or equal to the clear distance between columns.
6. The ratio of column or column cap area to influence area per column is greater than or equal to 10%.
7. The embankment fill effective friction angle is 35°.
8. The initial strain in the reinforcement is limited to 6%.
9. Long-term (creep) strain is limited to 2%.
10. The total strain is less than 70% strain at failure.

Figure 10.11 shows the model used in the Swedish method to determine the vertical load carried by the reinforcement. The cross-sectional area of the soil under the arch, which is the load carried by the reinforcement, is approximated using the soil wedge shown in Fig. 10.11. This applies even when the embankment height is lower than the top of the soil wedge [i.e. $(s - d)/(2 \tan 15°)$].



10.12 Swedish method: load distribution between columns (Rogbeck *et al.,* 2002).

The two-dimensional weight W_T of the soil wedge is determined from:

$$W_T = \frac{(s-d)^2\gamma}{4 \tan 15°} \text{ per unit length in depth} \qquad [10.9]$$

The three-dimensional effects are estimated through load distribution, where the load is distributed over the surface according to Fig. 10.12 and is taken up by the reinforcement along the edge of the column. The force in the reinforcement, per lineal metre of depth, due to the vertical load in three dimensions is calculated using the equation:

$$T_{rp} = 0.5\left(1 + \frac{s}{d}\right) W_T \left(1 + \frac{1}{6\varepsilon}\right)^{0.5} \qquad [10.10]$$

The German method

The German method (Alexiew and Gartung, 1999; Alexiew, 2003) unlike either the BS 8006 or the Swedish method, considers the effect of the soft foundation soil in determining the load carried by the reinforcement. Specifically, the undrained shear strength of the foundation soil is considered to provide some resistance to the vertical load from the embankment. The German method also directly considers the shear strength of the embankment material in determining the arching within the embankment. This method is only valid when the height H of the embankment is greater than the column spacing s.

Two failure criteria are considered: failure of the embankment fill at the crown of the arch (typically controls for light surcharge loads and large column spacing); failure at the bearing point of the arch. The ratio E of the vertical load on the columns to the average load at subgrade is a function of which failure mode controls the design.

For failure to occur at the crown of the arch (this condition occurs for relatively shallow embankments with wide column spacing) E is determined from:

$$E = 1 - \left[1 - \left(\frac{d}{s}\right)^2\right](A - AB + C) \qquad [10.11]$$

where:

$$A = \left(1 - \frac{d}{s}\right)^{2(K_p - 1)} \qquad [10.12]$$

$$B = \frac{s}{1.41H}\,\frac{2K_p - 2}{2K_p - 3} \qquad [10.13]$$

$$C = \frac{s - d}{1.41H}\,\frac{2K_p - 2}{2K_p - 3} \qquad [10.14]$$

$$K_p = \frac{1 + \sin\varphi'}{1 - \sin\varphi'} \qquad [10.15]$$

and where:

φ' = effective friction angle of the embankment fill.

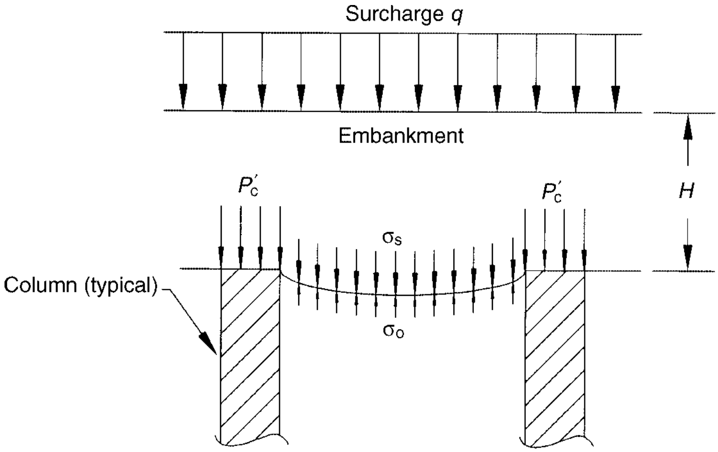
Failure at the bottom of the arch must also be analyzed with the following equation:

$$E = \frac{\beta}{(1 + \beta)} \qquad [10.16]$$

where:

$$\beta = \frac{2K_p}{(K_p + 1)(1 + d/s)}\left[\left(1 - \frac{d}{s}\right)^{-K_p} - \left(1 + K_p\frac{d}{s}\right)\right] \qquad [10.17]$$

The minimum value of E controls the stress σ_s applied to the soil between columns. The stress that is applied to the soil between columns (Fig. 10.13) is determined from:



10.13 German method.

$$\sigma_s = \frac{\gamma H + q}{(s^2 - d^2)(1 - E)s^2} \qquad [10.18]$$

The geosynthetic reinforcement is subjected to the stress σ_s on the soil less a vertical reaction stress produced by the supporting effect of the soil between columns (i.e. the bearing capacity) (Fig. 10.13). The factored (e.g. safety factor of 2) undrained shear strength c_u of the soil is used to determine the bearing capacity of the foundation soil. The equation for determining this allowable stress σ_o is:

$$\sigma_o = \frac{(2 + \pi)c_u}{FS} \qquad [10.19]$$

where:

FS = factor of safety for undrained shear strength (typically 2).

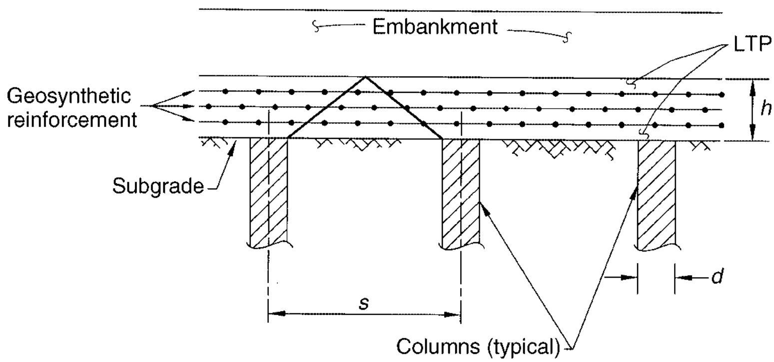
The vertical load W_T on the geosynthethic reinforcement spanning between columns is determined as follows:

$$W_T = \frac{\sigma_s(s^2 - d^2)}{2(s' - d)} - \frac{\sigma_o(s^2 - d^2)}{2(s' - d)} \quad \text{per lineal metre} \qquad [10.20]$$

where:

s' = s for a square-column pattern, and $1.4s$ for a triangular-column pattern.

For the case of more than one layer of reinforcement, the vertical load W_T may be distributed between the layers proportionally to their strain resistance. The German method, presented above, is based on the fact that the reinforcement is located vertically, less than 0.5 m (1.6 ft) above subgrade. Special procedures are provided



10.14 LTP.

by the German method for the case where the reinforcement is located between 0.5 and 1 m (between 1.6 and 3.3 ft) above subgrade (Kemfert *et al.*, 1997).

The tensile force in the reinforcement per unit length of reinforcement is determined on the basis of catenary tension and is determined by:

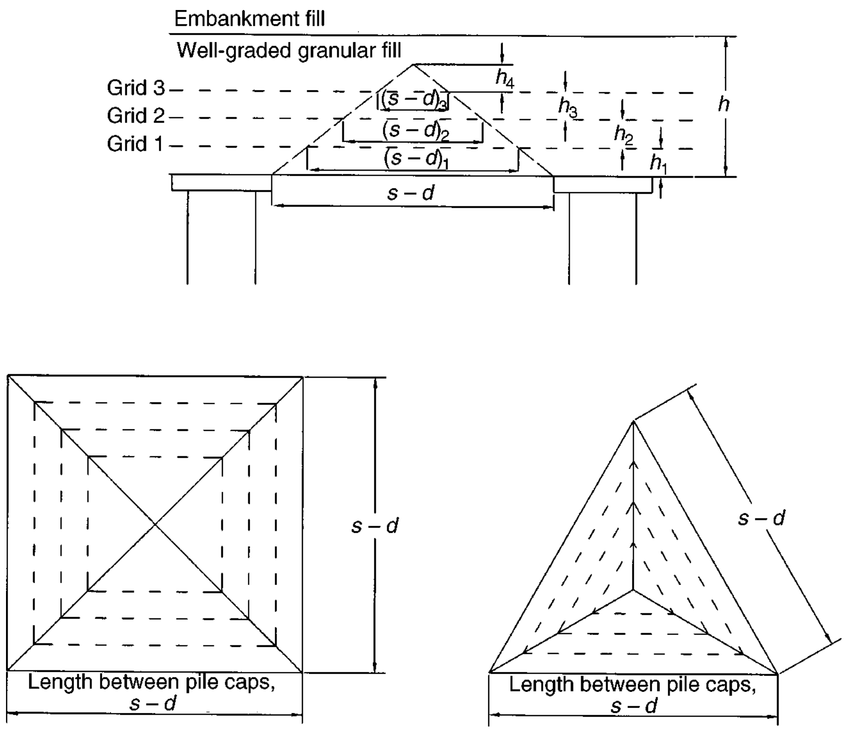
$$T_{rp} = W_T \frac{(s' - d)}{2d} \left(1 + \frac{1}{6\varepsilon}\right)^{0.5} \qquad [10.21]$$

The Collin method

The Collin method (Elias *et al.*, 2004) is fundamentally different from BS8006, the Swedish method or the German method (Collin *et al.*, 2005a). The Collin method is based on the premise that the reinforcement (minimum of three layers of reinforcement) creates a stiffened beam of reinforced soil that distributes the load from the embankment above the LTP (i.e. stiffened beam) to the columns below the platform (Fig. 10.14).

The Collin method is based on the following assumptions:

1 The thickness h of the LTP is equal to or greater than one half of the clear span $s - d$ between columns.
2 A minimum of three layers of extensible (geosynthetic) reinforcement is used to create the load transfer platform.
3 Minimum distance between layers of reinforcement is 200 mm (8 in).
4 Select fill is used in the LTP.
5 The primary function of the reinforcement is to provide lateral confinement of the select fill to facilitate soil arching within the height (thickness) of the LTP.
6 The secondary function of the reinforcement is to support the wedge of soil below the arch.
7 All the vertical load from the embankment above the LTP is transferred to the columns below the platform.
8 The initial strain in the reinforcement is limited to 5%.



10.15 LTP design for the Collin method.

The vertical load carried by each layer of reinforcement is a function of the column spacing pattern (i.e. square or triangular) and the vertical spacing of the reinforcement. Each layer of reinforcement is designed to carry the load from the platform fill that is within the soil wedge below the arch. The fill load attributed to each layer of reinforcement is the material located between that layer of reinforcement and the next layer above (Fig. 10.15).

The uniform vertical load W_{Tn} on any layer n of reinforcement may be determined from:

$$W_{\mathrm{T}n} = \frac{[(\text{area at reinforcement layer } n + \text{area at reinforcement layer } n+1)/2]\,(\text{layer thickness})\,(\text{LTP fill density})}{\text{area at reinforcement layer } n}$$

For a triangular pattern,

$$W_{\mathrm{T}n} = \frac{[(s-d)_n^2 + (s-d)_{n+1}^2]\sin 60°\, h_n\, \gamma}{(s-d)_n^2\, \sin 60°} \qquad [10.22]$$

and for a square pattern,

Table 10.1 Values of Ω

Ω	Reinforcement strain (ε) (%)
2.07	1
1.47	2
1.23	3
1.08	4
0.97	5

$$W_{\mathrm{T}n} = \frac{[(s-d)_n^2 + (s-d)_{n+1}^2]h_n \gamma}{(s-d)_n^2} \qquad [10.23]$$

The tensile load in the reinforcement is determined on the basis of tension membrane theory (Giroud *et al.*, 1990), and is a function of the amount of strain in the reinforcement. The tension in the reinforcement is determined from

$$T_{\mathrm{rp}n} = \frac{W_{\mathrm{T}n}\,\Omega\, D}{2} \qquad [10.24]$$

where:

D = design spanning for tension membrane (dimensionless factor) equal to $(s-d)_n$ for square-column spacing and to $(s-d)_n \tan 30°$ for triangular-column spacing
Ω = dimensionless factor given in Table 10.1.

The modified Collin (beam) method

Based on research recently completed (Collin *et al.*, 2005b; Han and Collin, 2005; Huang *et al.*, 2005a, 2005b) using numerical modelling, the above procedure has been modified. The modification involves the addition of one layer of reinforcement at subgrade. This layer of reinforcement is designed as a catenary to carry the load from the soil below the arch (Fig. 10.16).

The uniform vertical load W_{TC} on the catenary layer of reinforcement may be determined from:

$$W_{\mathrm{TC}n} = \frac{\text{(volume pyramid below the arch) (LTP fill density)}}{\text{area at reinforcement catenary layer}}$$

For square- or triangular-column spacing,

$$W_{\mathrm{T}n} = \frac{h_n \gamma}{3} \qquad [10.25]$$

The tensile load in the reinforcement is determined on the basis of tension

Table 10.2 LTP design method summary

Method	Design approach	Number of reinforcement layers	Angle of arch from horizontal (deg)	Subgrade support	Allowable strain ε in reinforcement (%)
British	Catenary	1[a]	70	No	6
Swedish	Catenary	1[a]	75	No	6
German	Catenary	1[a]	NA[b]	Yes	5–6
Collin	Beam	≥ 3	45	Yes	5

[a]More than one layer of reinforcement may be used, based on the required strength of the reinforcement. However, the layers are not discretely placed with a minimum of 200 mm (8 in) between layers, as in the Collin method.
[b]NA, not applicable.

membrane theory and is a function of the amount of strain in the reinforcement. The tension in the reinforcement is determined from:

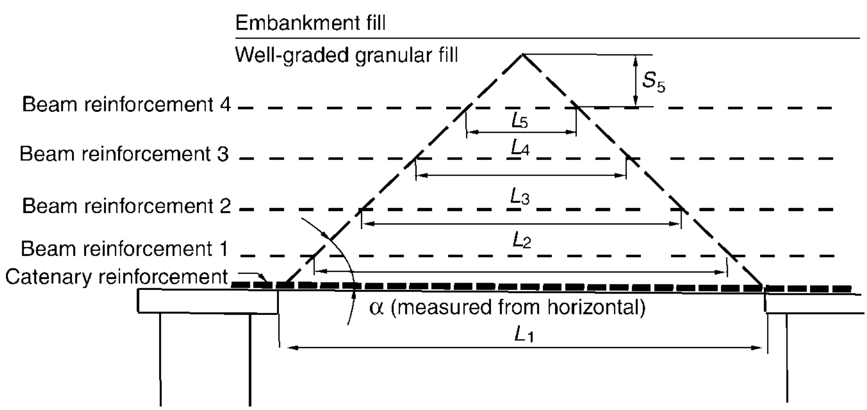
$$T_{rpC} = \frac{W_{TC}\,\Omega\,D}{2} \qquad [10.26]$$

where:

D = design span for the tensioned membrane equal to $1.41\{(s - d) - 2[\sum \text{vertical spacing}/(\tan 45°)]\}$ for square-column spacing, and to $0.867\{(s - d) - 2[\sum \text{vertical spacing}/(\tan 45°)]\}$ for triangular-column spacing
Ω = dimensionless factor from tensioned membrane theory.

The reinforcement to create the beam above the catenary layer of reinforcement is designed according to Equations [10.22]–[10.24].

The design methods summarized in this section have many similarities. However,



10.16 Modified Collin method reinforcement.

there are significant differences between the approaches that deal with the fundamental concepts of the load transfer platform. Table 10.2 provides a summary of these differences.

The use of CSEs is expanding both in the USA and abroad. Numerous design guidelines have been developed for the design. Currently, there are at least five to ten methods to design the LTP. The beam method presented here is one that has been developed by the present author and used with great success. However, the recommendations provided here cover only the basic steps in the design of the LTP. The detailing of the platform (i.e. edge detail), selection of geosynthetic reinforcement, creep characteristics of the geosynthetic, overlaps, etc., are beyond the scope of this chapter but must be considered in the design.

10.5.6 Reinforcement total design load

Independent of the method used to analyze the LTP, the total (maximum) design load T_{total} in the geosynthetic reinforcement should be determined as follows: in the direction along the length of the embankment

$$T_{total} = T_{rp}$$

and, in the direction across the width of the embankment

$$T_{total} = T_{rp} + T_{ls}$$

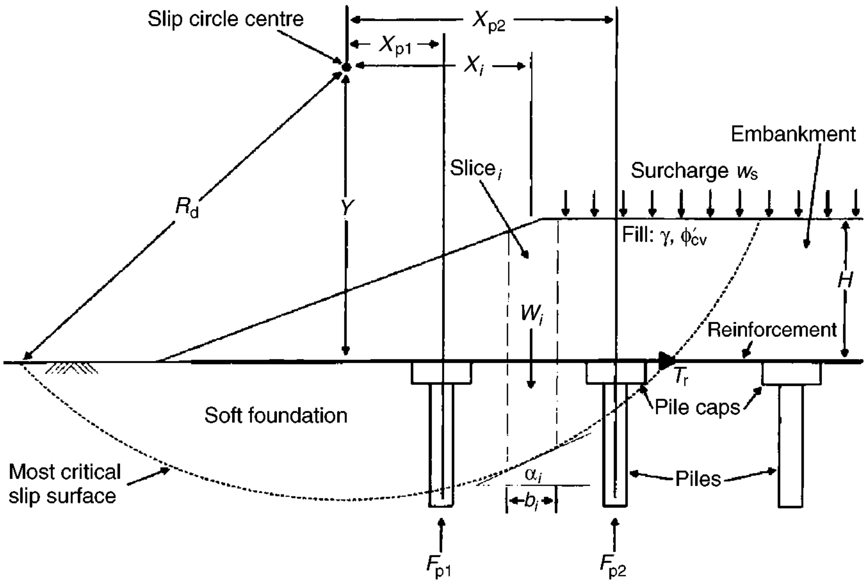
10.5.7 Global stability

Global stability of CSEs may be evaluated using limit equilibrium computer software, taking into consideration the added shear resistance of the columns and the tensile capacity of the geosynthetic reinforcement. Figure 10.17 shows the approach used in the BS 8006 for incorporating the benefit of the columns and geosynthetic.

While it is recommended that the global stability of the CSE be evaluated, it is the present author's opinion that, if the behaviour is that of a true CSE with an LTP, there is very little potential for a global stability problem.

10.5.8 Settlement

Total settlement of the CSE will be a function of the column design. For methods to estimate settlement, the reader is referred to numerous geotechnical engineering textbooks. Differential settlement between columns caused by the LTP should be less than 20–30 mm (0.8–1.2 in) when the LTP design follows the guidelines established for BS 8006, the Swedish method, or the Collin method. When using the German method, differential settlement calculations are required.



10.17 Variables used in the global stability analysis (British Standards Institution, 1995).

10.5.9 Current research needs

Gabr and Han (2005) suggested some directions for further study. The current design methods should be validated by full-scale well-instrumented field measurements investigating strains in the geosynthetic, deformation characteristics and stress distribution between column and native soil.

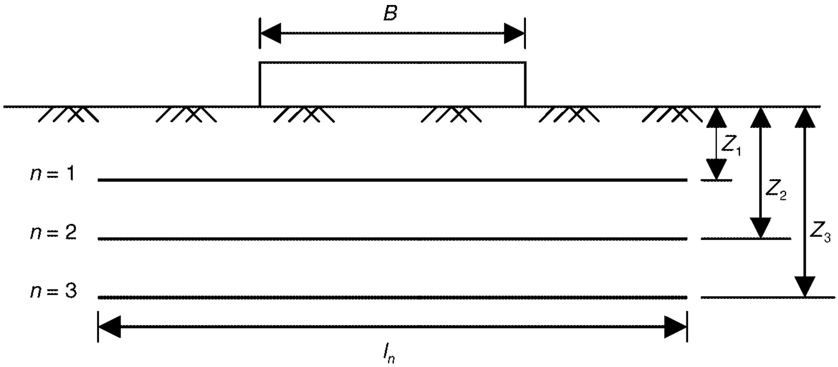
Soil-arching models currently employed assume rigid supports at the edges; for certain types of deformable soil improvement method, this assumption may be valid but, for other soil improvement techniques that result in less stiff columns, this assumption needs further verification. The current design methods do not allow the designer to estimate total and differential settlements. Finally, the effect of soil resistance between the native soil and the geosynthetics layer and the effect of geosynthetics creep within the formed earth beam are also poorly understood.

To apply tensioned membrane theory properly, sufficient strain must develop in the geosynthetic to result in some tension. As a result, there will be some displacement that occurs to generate this strain. This deformation typically occurs during placement and compaction of the embankment fill. However, further research is needed to verify this observation.

10.6 Geosynthetic-reinforced soil foundations

10.6.1 Overview

The use of geosynthetics in civil engineering applications is increasing annually.



10.18 Schematic diagram of a GRSF (Munfakh *et al.*, 2001).

One new application, the construction of a geosynthetic-reinforced soil foundation (GRSF) to support a shallow spread footing has considerable potential as a cost-effective alternative to over-excavation and replacement, consolidation, densification and chemical stabilization. In this technique, one or more layers of a geosynthetic reinforcement and controlled fill are placed below a shallow spread footing to create a composite material with improved performance characteristics over the existing foundation soils (Fig. 10.18).

Reinforcement of fill or natural soils with geosynthetics beneath shallow foundations has been explored for nearly three decades, after the pioneering soil reinforcement work of Binquet and Lee (1975a, 1975b). Das (1995) summarized results of predominantly small model strip or square footings in test boxes filled with sand or clay. Their work has identified, for the situations tested, a series of bounds on reinforcement spacing, number of reinforcing layers, total reinforced depth and reinforcement width, i.e. they have identified dimensions relative to the width of the footing where no additional benefit is gained. These tests seem to suffer from unknown scale effects, as explained by Michalowski (2004), and it is unclear whether the findings are general enough to apply to other soils.

Adams and Collin (1997) performed the first (and, to this date only) prototype-scale tests on square footings. This work was sponsored by the FHWA and was performed at Turner–Fairbank Highway Research Center in a large test pit filled with sands reinforced with geogrids and geocells. Their results seemed to confirm some of the relationships noted by Das and his colleagues and showed that an increase in bearing capacity could be obtained using reinforced soils.

In general, few design methods are available for determining the bearing capacity of shallow foundations. Huang and Menq (1997) suggested an empirical formula for reinforced soils after the 'deep footing' effect reported by Schlosser *et al.*, where the reinforcement spreads the load with depth, such that the system can be modelled as a wider footing acting at the depth of the last reinforcement layer. The increase ΔB in footing width is estimated by the Huang and Menq method using the Binquet and Lee (1975a, 1975b) laboratory-scale testing results of soils reinforced with geosynthetics, fibres, aluminium strips, etc. The same criticism of Das's work can be applied to this analysis.

Michalowski (2004) suggested a method to estimate the upper bound of bearing capacity for reinforced-soil-mass-based failure surfaces determined by plasticity theory. His results were for strip footings only, and take a form similar to a typical bearing capacity equation. This method is promising but still requires considerable calibration and refinement before it can be adopted for use in practice.

Research has also focused on optimizing the location of the reinforcement below the shallow spread footing (Guido *et al.*, 1987; Chadbourne, 1994; Omar *et al.*, 1994; Yetimoglu *et al.*, 1994; Espinoza and Bray, 1995); Adams and Collin, 1997; Adams *et al.*, 1997).

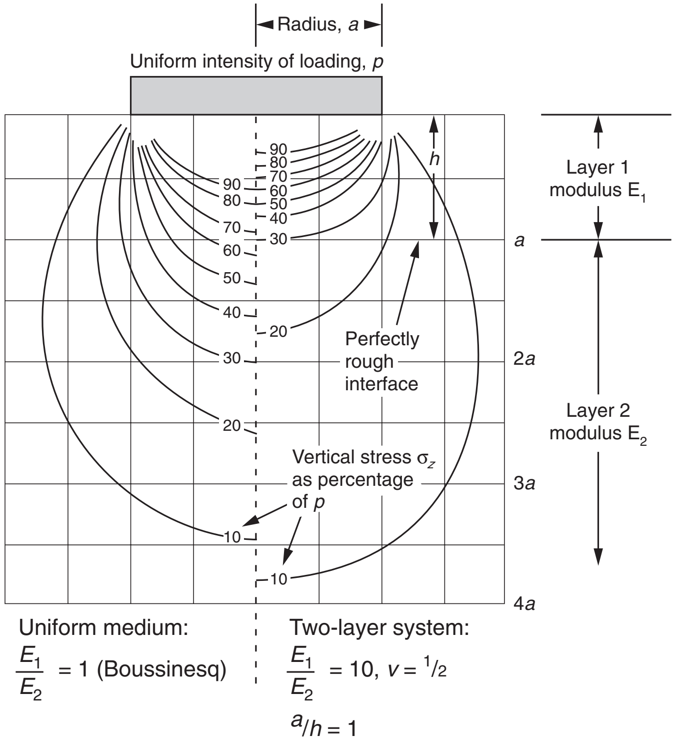
The optimum location of the geosynthetic below the spread footing has been suggested by researchers to be somewhere between 0.5*B* and 1.5*B* (where *B* is the width of the footing). The location of the reinforcement has a direct impact on the economy of a GRSF. If the reinforcement extends to a depth of 1.5*B*, the cost of the GRSF may be several times greater than a GRSF that extends to a depth of 0.5*B* below the spread footing. Work by Adams and Collin (1997) and Adams *et al.* (1997) quantified the vertical and lateral strain at several locations below shallow spread footings, both with and without a reinforced soil foundation. Recommendations were then developed with respect to the location of the geosynthetic reinforcement for optimal performance of the GRSF based on the location of the zone of maximum strain in the foundation soil.

10.6.2 Design considerations

The maximum improvement in bearing capacity at low strains ($s/B = 0.5\%$) occurs when the depth to the top layer of reinforcement is within a depth of 0.25*B* from the bottom of the footing ($z_1 < 0.25B$). The GRSF should extend to a depth of 0.5*B* below the bottom of the footing. The maximum vertical spacing between layers of reinforcement should be less than 0.5 m ($z_n - z_{n-1} < 0.5$ m) and the length of the reinforcement should be at least equal to 2.0*B* ($l_n = 2.0B$).

When using the above guidelines for minimum reinforcement spacing and length, the design of a GRSF is based on the Westergaard layered elastic theory. The Westergaard theory is based on the assumption that the stress from the footing is distributed within the stiff layer as vertical stress only (e.g. no horizontal stress or deformation). The use of many layers of tensile reinforcement (geosynthetic reinforcement) in the reinforced soil foundation reduces lateral deformations within this layer to virtually zero, thus making the Westergaard theory appropriate for design.

The GRSF acts as a stiff soil layer overlying a softer soil deposit. The stiff GRSF acts like a beam and distributes the stresses from the footing over a larger area. This reduces the unit stress on the foundation soil from the footing. Figure 10.19 may



10.19 Westergaard stress distribution for a two-layer system (Munfakh *et al.*, 2001).

be used to estimate the vertical stress exerted on the foundation soil when using a GRSF. The steps in the design of a GRSF are briefly outlined as follows:

Step 1 Perform site investigation and develop the design cross-sections for the proposed structure.

Step 2 Perform laboratory tests to evaluate the shear strength and load versus deformation characteristics for the foundation soils.

Step 3 Design the shallow foundation system for the structure without a reinforced soil foundation.

Step 4 Develop the design cross-section (using the guidelines provided above for minimum reinforcement spacing and length).

Step 5 Determine the vertical stress distribution in the foundation soil using the Westergaard elastic layer theory.

Step 6 Determine the bearing capacity of the reinforced soil foundation. In this step, treat the reinforced soil below the footing as a stiff soil with a large friction angle and check the bearing capacity of this layer. Also check the bearing capacity of the foundation soil including the effect of embedment.

Step 7 Check the settlement of the foundation soil using the vertical stress distribution determined in Step 5.

Step 8 Determine the required geosynthetic properties.

The use of a GRSF has been very limited in the USA. The above generalized design methodology is presented to provide insight into the steps that are believed to be important in the design of a GRSF. These recommendations are provided as a tool to determine the feasibility of a GRSF. However, the recommendations provided here cover only the basic steps in the design. The selection of geosynthetic reinforcement, creep characteristics of the geosynthetic, overlaps, etc, are beyond the scope of this chapter but must be considered in the design.

10.6.3 Future research needs

Clearly, the development of a relatively simple design methodology for a shallow foundation on a reinforced soil mass is important for state-of-practice implementation. The methodology presented in the previous section is based on limited research and actual implementation. Testing on a wider range of soils with either geogrids or geotextiles seems imperative, as do a wider range of instrumented full-scale tests on different footing shapes. Current methods also do not quantify how to determine the optimum size and spacing of geosynthetic reinforcement.

In most cases, bearing capacity does not control shallow foundation design. Some work has been carried out to calibrate measured strains in large-scale laboratory tests to existing settlement calculations. This must be considered for a wider range of geosynthetics to verify the assumptions of the elastic modulus increase, and for a variety of spacings.

Finally, the economics of reinforced soils should be addressed. Unless the footing is being placed over reinforced soil for a mechanically stabilized earth wall or other reinforced soil slope, the construction of the reinforcement zone requires excavation of an area to a depth where the attenuated stresses do not exceed the subgrade strength. When adding in the cost of geosynthetics and backfilling with competent material, the cost of simply constructing a larger traditional footing must be considered.

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11
Quality assurance for geosynthetics

D. SMITH
EDGE Consultants UK Ltd, UK

11.1 Introduction

When civil engineers talk about quality assurance in respect of geosynthetics, there is an inevitable immediate link with landfill engineering and, in particular, the geomembrane liner element. There have been many guidance documents produced from around the early 1980s onwards relating to the quality assurance requirements for design and construction of landfill lining systems utilizing geosynthetics.

The need for good quality assurance throughout the design and construction of a landfill site cannot be underestimated. The landfill environment is arguably the most demanding for any construction material and not just geosynthetics. However, good quality assurance in connection with other applications of geosynthetics should not be assumed nor ignored.

Geosynthetics are utilized at some stage within almost every civil engineering construction project undertaken, from the small project requiring a filter geotextile within a short length of gravel-filled drainage system to the major project incorporating massive geosynthetic-reinforced soil walls and everything in between. Whatever construction work is being undertaken, some form of quality assurance system must be in place to ensure that the geosynthetic system and its constituent parts will function as required.

This chapter provides some definitions of quality, quality assurance and quality control, some guidance on who has the responsibility for the various elements of quality assurance and some comment on design aspects. It then goes on to deal with manufacturer's quality control and construction quality control (CQC) and construction quality assurance (CQA) before outlining some of the benefits of a quality assurance system and providing some indication of the costs involved. The chapter is written generally with geosynthetic lining systems in mind because this is where most of the literature provides guidance. Where appropriate, guidance relating to other geosynthetic types is provided.

11.2 Definitions

It is useful to provide some definitions relating to quality. Quality assurance is not the only aspect of quality that should be considered. There is also the aspect of quality control.

The *Compact Oxford English Dictionary of Current English* (2005) gives the following definitions.

1 *Quality.* The degree of excellence of something as measured against other similar things.
2 *Assurance.* A positive declaration intended to give confidence.
3 *Quality control.* A system of maintaining standards in manufactured products by testing a sample against the specification.

From this, it is apparent that 'quality assurance' is meant to provide confidence that what is being 'assured' is of a required 'quality'. 'Quality control' on the other hand is intended to ensure that a particular standard has been met.

Quality assurance is not limited to the construction materials and the construction process but should be applied throughout the project process, including selection of designer, the design itself, validation of the design, selection of appropriate construction materials, construction and validation of construction.

In relation to construction materials and the construction itself, there are generally two aspects to quality control and assurance. There is the control and assurance performed by the supplier or manufacturer of the material to ensure that the product delivered to site is correct, to specification and of the required quality and also the control and assurance performed during the construction process to ensure that what is constructed is also correct, to specification and of the required quality.

In the context of geosynthetics, it is convenient to adopt terminology used within the landfill lining sector to describe quality assurance and quality control. Koerner (1997) provides definitions for manufacturing quality control, manufacturing quality assurance, CQC and CQA. Although these definitions are directed specifically at requirements for landfill lining systems, they are equally valid for all applications of geosynthetics.

The various definitions detailed by Koerner are not repeated here. Essentially, however, quality assurance can be said to embrace all activities and functions concerned with the attainment of quality, rather than the proof associated with the word 'assurance'. Thus, quality assurance includes the determination and assessment of quality. The concept of quality control is to ensure that the quality of the end product meets a predetermined standard. The term may be applied to the system of control or to the product or service being controlled.

11.3 Responsibilities

The process of quality assurance within the project is started by the client. Under normal circumstances the client who has the requirement for a particular construction project is responsible for appointing a designer to work with him or her to develop the scheme into something that meets requirements. He or she must ensure that a designer is appointed who is appropriately qualified and experienced to undertake the work. He or she is also responsible for providing the designer with an appropriate and realistic brief.

The designer is responsible for ensuring that reasonable skill and care are demonstrated in developing the design. A recognition and use of relevant published guidance and standards are essential. The designer is responsible for producing reports, drawings, specifications, schedules etc. to present all the required construction details in such a manner as to allow a construction company to undertake the works. There are some variations on this general approach, e.g. design and build, and partnering, but essentially this is the basic process.

At various stages of the process, depending on the type of project, there may be a requirement for some input from regulators and/or inspectors from environmental, planning or other stakeholders. The regulators and/or inspectors will be responsible for ensuring that the whole of the construction meets their local minimum standards. It is the responsibility of the designer to ensure that the client is aware of these minimum standards and that the design complies with them.

Both the manufacturer and constructor are responsible for implementing appropriate quality assurance and quality control procedures to ensure that the end product meets the required specification. There is a tripartite responsibility for monitoring these procedures.

1 The designer must satisfy himself or herself that the quality assurance and quality control procedures are in place – supported by the client as necessary.
2 A third-party quality assurance and quality control reviewer may be appointed to manage and control the whole quality assurance and quality control package. The reviewer will be responsible for making such records as necessary to provide the proof that the procedures have been followed. It is common for the designer to undertake the quality assurance and quality control review role in many cases. This can be an acceptable way forward; ensuring that the quality assurance and quality control reviewer is familiar with the site and the design before the works are undertaken. It may also improve communication between the designer and the quality assurance and quality control reviewer. The type of quality assurance and quality control review arrangement should be assessed on a case-by-case basis.
3 The regulator/inspector must review and approve the proof that the quality assurance and quality control procedures have indeed been implemented.

11.4 Design aspects

Designing with geosynthetics follows standard relationships of stress, strain, temperature, pressure, flow, etc. The literature contains many methodologies that have been developed from these basic principles that can be utilized in the design of geosynthetic systems. There is also a plethora of statutory guidance, national standards and case histories relating to all aspects of designing with geosynthetics. It is simply not acceptable to pull a 'standard specification' off the shelf for a particular geosynthetic application in the hope that it will perform adequately. Appropriate design will ensure that the correct geosynthetic material parameter values are determined to allow the correct specification to be prepared.

Design is best undertaken within a quality-assured system. This should ensure that all aspects of the project are taken into account within the design process. It is essential that all aspects of the construction project are designed appropriately. Geosynthetics are likely to be only part of a larger system within a construction project. Particular attention must be given to the interaction of all components of the structure system. Understanding the interaction of the individual components will enable the designer to identify where incompatibilities lie, where the weak points of the system are and to put in place measures to design these issues out. For example, it may be the case that ground conditions are such that a certain type of geosynthetic reinforcement or filter geotextile is more appropriate than another; different polymers and products may perform better under certain conditions.

There are essentially five functions that geosynthetics perform.

1 Reinforcement.
2 Separation.
3 Filtration.
4 Containment.
5 Drainage.

In some cases a geosynthetic may be required to perform multiple functions, e.g. separation and filtration, or reinforcement and drainage. In other cases, composites of geosynthetics may be formed that perform multiple functions.

In some cases, there may be a number of different methods of designing a particular system. For example, many methodologies have been published over the years relating pore opening size and particle size to designing a geotextile filter. Essentially, in all design work relating to geosynthetics, as with other construction materials, it is the designer who should ensure that the most appropriate relationship is adopted in the particular case under consideration. It is possible that this will include the testing of particular geosynthetic products together with site-specific materials to assess performance and to test the design assumptions that have been made, e.g. shear box testing of reinforcement geosynthetics together with material to be used in construction to assess interface friction values or compliance testing of geomembranes with site-specific leachate to assess compatibility.

11.5 Manufacturing quality control

Geosynthetics are manufactured from a variety of polymer resins. The selection of the resin and the method of joining the molecules together will result in different final characteristics of the geosynthetic. Quality control at this stage of the process is very important, but the civil engineering designer probably does not need to know all the intricacies of the quality control measures put in place by the manufacturer at this stage. However, the designer does need to know that the manufacturer takes the quality control process seriously and needs to be confident that the product that is being delivered to site after passing through the quality control process is suitable for use and meets specification.

Manufacturer's quality control processes will generally include random sampling and testing of the raw materials, visual and automated inspection during manufacture and sampling and testing of the final product. Sprague (1995) suggested a sampling and testing frequency for all manufacturers to follow in order to introduce some consistency in the reporting of material properties. The frequency varies with the size of the lot being manufactured. The larger the lot, the lower the frequency of testing required. Minimum frequency suggested is around 10% of the manufactured units.

It is important that the designer understands what the values quoted on the manufacturer's product literature mean when selecting products which potentially meet specification. Manufacturer's product literature quotes characteristic values for various properties of the geosynthetic. The range of characteristic values quoted depends on the intended function of the geosynthetic. In general terms the parameters quoted are the minimum average roll values (MARVs). Sprague (1995) explains that it is essential that the confidence limit associated with the MARV is also reported so that the designer can use the value with confidence.

The European Union (EU) has introduced the CE marking scheme to geosynthetic products. All geosynthetic products sold within the EU should have a CE mark. This effectively means that the manufacturer has gone through a process to sample, test and prove that the product meets certain minimum standards. The roll of geosynthetic delivered to site with a CE mark must have an accompanying certificate that details the properties of that particular roll. The properties reported are defined by the minimum standards documents. This system provides confidence to the user that the product being used has attained the standard quoted, although it does not provide any site-specific guarantees. It should be noted that at this point in time (2006) the UK Government, unlike other EU countries, is choosing not to enforce the requirement for CE marking on geosynthetic products. This potentially allows substandard geosynthetics to be sold in the UK and is despite the fact that all UK geosynthetic manufacturers have invested significant time and expense in achieving the CE mark for their products.

The designer must ensure that details of the manufacturer's quality control policy are obtained and that the details are appropriate and are being followed.

11.6 Installation and construction

Installation or construction quality assurance and quality control should be adopted to ensure that the geosynthetic products being delivered to site and utilized in the construction meet specification and are installed correctly. The degree of construction quality assurance and quality control should be proportionate to the scale, complexity and economics of the construction project under consideration. The amount of quality assurance and quality control adopted for our small project requiring a filter geotextile around a gravel-filled trench will be significantly different from that required for the major geosynthetic-reinforced soil wall and different again from the requirements within a landfill lining system.

It is possible that the properties listed on the manufacturer's product literature, together with a certificate of compliance will be sufficient evidence for the designer to be assured of the required quality. At the other extreme, a detailed CQA plan may be required to define exactly what sampling, testing, inspection and reporting regime must be followed.

To illustrate the detailed requirements of a CQA regime, it is convenient to consider a landfill lining project. Typical elements of the CQA regime are likely to include the following.

1 The lining material delivered to site (usually taken from rolls delivered to site but can be taken from the suppliers store if programme requires it and the designer can be assured that the rolls sampled and tested will be used in the works) should undergo conformance testing. Both identification and performance testing should be undertaken (Rollin and Rigo, 1991). The testing regime should be designed to suit the application.
2 The delivery, handling and storage of the lining material must be undertaken in such a way as to prevent any damage before the installation is undertaken. Visual inspection of all rolls should be undertaken and it is essential that each roll can be positively identified and related back to manufacture details such as batch number, date of manufacture and roll identification number. Certificates detailing properties for each roll will normally be required.
3 It must be ensured that the subgrade to accept the lining system is prepared to reduce the risk of puncture and settlement of the subgrade to a minimum. In some cases, it may be necessary for the liner installer to accept formally the subgrade from an earthworks contractor. This should be signed on to by all parties involved.
4 Each element of the lining system must be placed in an appropriate manner. Visual inspection of each sheet or panel of each component of the lining system is essential. Any anomalous features should be recorded and, where necessary, affected sheets or panels should be replaced or repaired. Records of actions taken to rectify any anomalies should be made.
5 The individual sheets or panels should be joined together in an appropriate manner. For the geomembrane this will require test seams to be carried out as

a minimum every day and for each combination of operator and seaming machine, with test seams being destructively tested. For other geosynthetic components of the lining system this may require sewing, lapping or other joining techniques to be undertaken. Testing of joints may be required depending on the function and requirements of the design.

6 The weather must be monitored and recorded during the installation process. An increase in wind speed can cause sheets or panels to move around suddenly and violently, making conditions very hazardous for the installers. A change in air and geomembrane temperature throughout the day may mean that the welding machines may need to be recalibrated.
7 The condition of the liner surfaces before welding should be monitored to ensure they are clean and dry.
8 Destructive and non-destructive testing of geomembrane seams must be carried out to ensure their integrity and strength. Every seam made on site should have some form of test undertaken. Locations of destructive tests should be selected carefully to reduce the number of patches required to a minimum within the operational landfill cell. For cases where there is confidence that the installation team is of good quality and are performing the seaming with care throughout, the number of destructive tests on seams can be reduced. It may be appropriate for all destructive tests to be undertaken at the ends of seams rather than the middle of seams to satisfy this requirement in this case. Where the installer's performance history is unknown, then the number of destructive seams should be increased to suit. It is recommended that the number of midseam destructive tests is kept to an absolute minimum. Some further guidance on this is provided by the International Association of Geosynthetic Installers (2004).
9 Destructive and non-destructive testing of joints should be performed on other geosynthetic sheets or panels utilised in the landfill lining system.
10 Conformance testing must be carried out on other materials utilized within the landfill lining system. This may include mineral lining, mineral protection or mineral drainage components. Note that all these functions can be met by the use of geosynthetics.
11 A leak detection survey on the installed system must be undertaken to check for holes and then any holes located must be repaired appropriately. There are a number of leak detection survey methods available.
12 It must be ensured that the first layer of waste placed within the cell does not damage the lining system.

It would be normal practice to prepare a report detailing all aspects of CQA undertaken. The CQA report would typically contain the following information as a minimum.

1 A brief description of the project, including the type of facility, name of site, location, name of owner, design engineer, liner installer and main contractor.

2 A detailed description of the lining system, including the surface area, cross-section and definition of all materials.
3 A reference to the CQA plan.
4 A copy of all subgrade inspection forms.
5 A statement that construction has been carried out in substantial accordance with the design (including modifications, if any, approved by the engineer).
6 The regulator's agreement in writing to any design modifications as sought before implementation.
7 A construction record drawing clearly showing changes from design drawings.
8 As-constructed drawings of each layer of the lining system, including basal and surface surveys of any mineral liners and drainage layers.
9 A discussion of any particular problems encountered and their solutions.
10 A copy of specifications for all elements of the lining system.
11 A copy of (or reference to) the geosynthetic manufacturer's quality control documentation.
12 A quality assurance record drawing indicating panel code numbers, seam code numbers, dates of seaming and repairs and locations and nature of all repairs.
13 A general record of activities, such as dates of performance of quality assurance operations, number and names of quality assurance monitors, names of personnel of liner installers.
14 Copies of all forms and logs filled out by quality assurance monitors.
15 Copies of all field and laboratory test results.
16 A photographic record including general photographs of the site at different phases of construction and specific construction details.

It can be seen that the CQA requirements for landfill lining systems can be onerous. They should only be carried out by qualified and experienced personnel. It is inappropriate to define frequencies of testing and exact methodologies to adopt. Adams *et al.* (2001) suggested that blindly following regulatory-driven CQA programmes in landfill lining applications is not the way forward. Making use of the advances in technology, e.g. leak location surveys and improved seaming techniques, to reduce the overall requirements of a CQA programme while retaining the confidence in the overall end product is a more appropriate methodology.

There are site-specific issues that must be referenced to the regulatory regime at the facility location and discussed and agreed with the appropriate regulator before undertaking the works. As an absolute minimum, the agreement of the CQA plan with the regulator in advance of a project must be obtained. Otherwise, there is a disaster waiting to happen.

The Environment Agency (2005) in the UK has provided draft guidance which states that best available techniques must be applied to CQA at landfills and gives general guidance in relation to specific geosynthetic elements within the system. Similar documents are available in other countries.

11.7 Benefits

The adoption of good CQA procedures during construction of landfill lining systems has been clearly demonstrated to result in less leakage from the lining system in numerous papers published over the years. The most recent of these (Koerner and Koerner, 2006) showed that the average number of leaks per hectare reduces from 22 to four when CQA is adopted. The paper by Koerner and Koerner outlined the Geosynthetic Research Institutes' CQA and inspectors certification programme. This highlights the importance of utilizing appropriately trained personnel for CQA work. In the UK, the Environment Agency introduced a training course for CQA inspectors on landfills in the late 1990s, although this does not result in a formal qualification.

Direct proof of benefits in other geosynthetic applications has not been published in detail as far as the author is aware. For example, there has not been a study into the number of reinforced soil slopes and walls that have failed with and without a quality assurance and quality control regime in place. However, it is logical that adopting a good quality assurance process when designing with geosynthetics will provide benefits in terms of increased confidence in the various elements of the structure.

11.8 Costs

As stated above, the degree of quality assurance activity required when using geosynthetics will depend on the scale and complexity of the project being undertaken and also the economics of the project.

Darileck and Laine (2001) suggested some costs for identifying defects in geomembranes (sheet or seams) following regulatory-driven CQA regimes in landfill applications in the USA. They concluded that direct conventional CQA costs would be on average about US$8000 per problem identified. They compared this with simply undertaking a leak location survey on completion of the works and concluded that leak location survey costs were around US$1200 per problem identified.

Although these figures give an indication of the general costs of CQA, they are not particularly useful as they are not related to area. They also assume that the holes in a liner are inevitable whereas the ideal result of adopting an appropriate CQA regime within a landfill lining application will be to ensure that there are no holes in the liner on completion. This is possible.

Quantifying the cost of CQA is difficult as it depends on many factors including operating area, size of facility, type of facility and regulatory regime amongst others. There will inevitably be a balance to be struck in determining the required extent of the quality assurance regime to be adopted. This will be heavily influenced by the risk associated with the failure of the geosynthetic element being used and needs to be considered carefully within the design process.

11.9 Future trends and sources of further information

Quality assurance related to the use of geosynthetics in civil engineering will always be required as with all other construction materials. In some areas of use this will inevitably be more complex than in others as detailed above.

The increasing number of directives and guidance published throughout the world, in particular in relation to environmental issues, will inevitably lead to more detailed requirements for quality assurance and quality control at the various stages of all construction projects. It is essential that this is kept in perspective, that the quality assurance and quality control requirements are appropriate for the usage of the geosynthetic and that the cost benefits are positive. As technology advances, the use of modern methodologies to assist in the quality assurance and quality control process must be encouraged where appropriate, particularly where this improves the final product and reduces the risk of failure occurring while at the same time reducing costs.

The designer should obtain guidance on quality assurance and quality control requirements from the local regulators (e.g. the US Environmental Protection Agency or the UK Environment Agency) or design standards. Further guidance can be obtained from organizations such as the International Geosynthetics Society (2006) and the Geosynthetics Research Institute (2006) and from publications such as *Geosynthetics* (www.geosyntheticsmagazine.info) (Industrial Fabrics Association International, 2006), *Geosynthetics International* and *Geotextiles and Geomembranes*.

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12
Limited-life geosynthetics

R. W. SARSBY
University of Wolverhampton, UK

12.1 Introduction

Over the past 50 years the use of man-made polymeric materials to produce high-strength durable technical fabrics has resulted in the widespread use of geotextiles to improve the engineering performance of the ground. Whilst the polymeric technical fabrics have long working lives, they are often used in practical situations where a geotextile is only needed to be fully functional for a relatively short period of time, e.g. a separator layer beneath a temporary access road, or basal reinforcement of an embankment built on soft clay. The technical requirements of such fabrics could be satisfied by geotextiles which can be designated as 'limited-life geotextiles (LLGs)' i.e. high-specification geotextiles that are designed on the basis of having a limited, clearly definable working life (Sarsby, 1997). These materials are designed so that progressive loss of their capability with time is matched by improvement in the ground conditions with time (usually due to drainage and consolidation). Vegetable fibres are natural candidates for use in the manufacture of LLGs since they are a renewable resource (and are often a waste or by-product from food production), they are environmentally friendly and their degradation with time is accounted for in the design of the LLG. Although man-made fibres dominate the technical textiles market (Byrne, 2000) a significant quantity of natural fibres are also used to make high-specification products (Table 12.1) and it is this type of product that has the potential to make a major contribution to a sustainable construction industry through enhanced use of renewable natural resources. The use of indigenous natural fibres also helps developing countries through support of agro-industry, provision of local employment and avoidance of costly imports. However, the ability of natural fibre products and components to meet technical and customer appeal requirements at a competitive price remains to be demonstrated.

12.2 Concept of limited-life geosynthetics

There are a significant number of ground engineering situations where the critical

Table 12.1 Global fibre production in 2000 (after Davis, 2001)

Fibre	Quantity ($\times 10^3$ t)
Cotton	19 800
Flax/Linen/Ramie	590
Jute and other vegetable fibres	4 000
Silk	90
Wool	1 300
Total natural fibres	**25 860**
Polyester	18 900
Polyamide	4 100
Acrylic	2 700
Polypropylene (including tape)	6 000
Elastane, aramids, etc.	310
Cellulosics	2 800
Glass	2 600
Total man-made fibres	**37 410**
Total	**63 270**

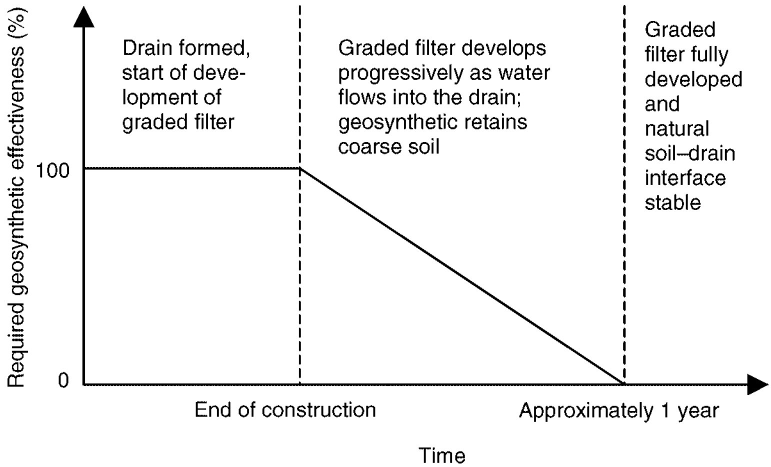
case for stability or functionality is either immediately (or very shortly) after construction and beyond this stage the stability of the system is constant or increases with time or the need for full functionality declines with time.

12.2.1 Filtration

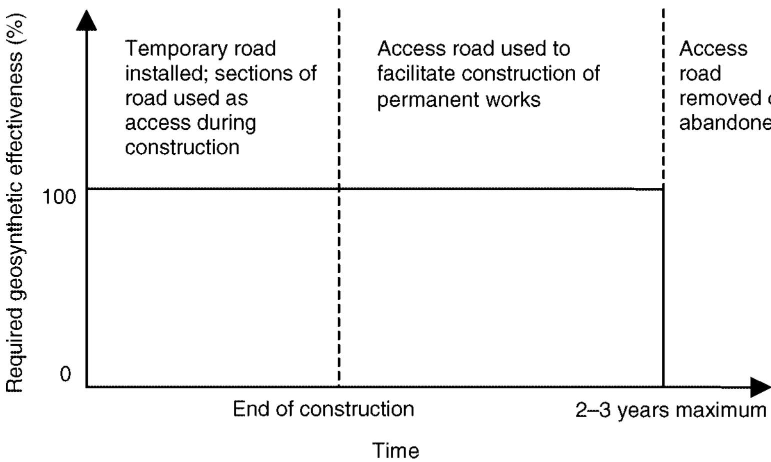
Ground drains are typically vertical-sided trenches lined with a geosynthetic and then filled with coarse gravel. The geosynthetic acts as a filter by permitting the flow of liquid and gases but prevents major passage of soil particles which could cause blockage of the drain or settlement due to loss of ground. Initial loss of fine soil particles adjacent to the geosynthetic encourages the formation of a zone in the ground wherein particles bridge over the pores in the geosynthetic. This zone retains smaller particles, which in turn retain even smaller particles. Thus, a natural graded filter is formed which will prevent additional washout of fine particles. This arrangement of particles is structurally stable and the geosynthetic becomes redundant. The generalized design life envelope, i.e. variation in required functional effectiveness with time, is illustrated schematically in Fig. 12.1.

12.2.2 Separation

A geosynthetic acts as a separator by preventing the intermixing of coarse and fine soil materials whilst allowing the free flow of water across the fabric. A typical situation is when a geosynthetic is placed between the subsoil and the granular subbase of an unpaved temporary access road. The section of fabric beneath a wheel or machine track acts like a tensioned membrane (since it is 'anchored'

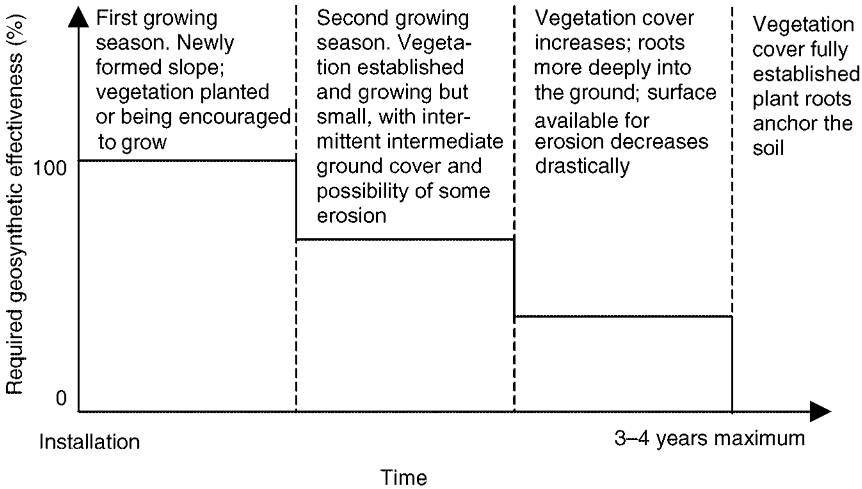


12.1 Design life envelope for a filtration geosynthetic.



12.2 Design life envelope for a separation geosynthetic.

within the fill either side of the wheel or track) and prevents the aggregate from being punched down into the soil during initial compaction and subsequent dynamic loading from vehicle axles. At the same time, the geosynthetic allows water to pass upwards through itself but prevents large quantities of fine soil from doing so. Once the permanent works are completed, the temporary haul road can be dug up and the granular material–geosynthetic mixture is disposed of. The design life envelope for a separation geosynthetic is illustrated in Fig. 12.2.



12.3 Design life envelope for an erosion control geosynthetic.

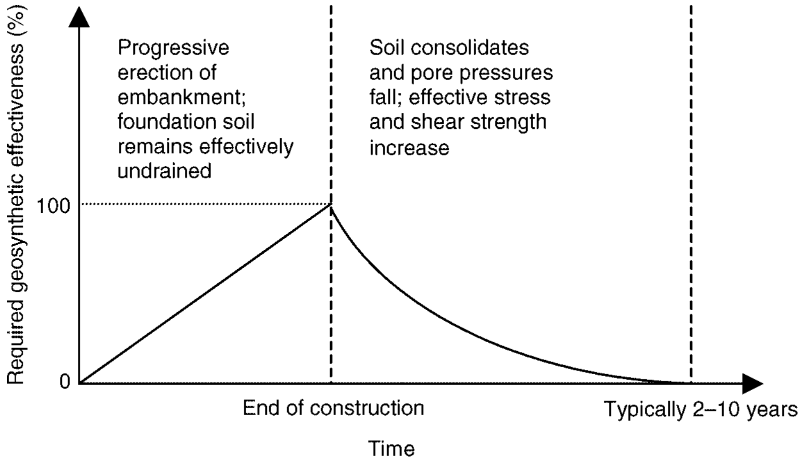
12.2.3 Erosion control

A current major area for geosynthetic utilization is in the erosion control industry. This usage differs from the other applications of geosynthetics in that these materials are laid on the ground and are not buried in the soil. The main aim is to control erosion whilst helping to establish vegetation which will then control erosion naturally. Geosynthetics can reduce run-off, retain soil particles and protect soil that has not been vegetated from the sun, rain and wind. Even when no longer needed for erosion control, the fabric may be utilized (to input nutrients into the soil as it degrades) if it is made from suitable material. The design life envelope is shown in Fig. 12.3.

12.2.4 Reinforcement

When an embankment is constructed over soft compressible ground, the load from the fill promotes foundation failure in the underlying ground without creating any immediate increase in its shear strength. The insertion of geosynthetics within the embankment and/or at its base will provide extra lateral force to prevent the embankment from failing by splitting or rotation. With time, pore water in the foundation will migrate from beneath the embankment and the shear strength of the foundation will increase. The stability of the embankment will thus improve in time as the underlying soft soil consolidates. As the underlying soil strength increases so the stabilizing force which needs to be provided by the geosynthetic diminishes (Fig. 12.4).

Tables 12.2 and 12.3 contain summaries of the relative importance of geosynthetics



12.4 Design life envelope for an embankment reinforcement geosynthetic.

properties for different engineering applications. In Table 12.2, the entries in bold type indicate those properties that are needed only until the end of the construction/installation stage. In Table 12.3, the entries in bold type indicate those properties that are needed after construction and installation but the importance of which diminishes rapidly (and even vanishes) with time. It is readily apparent that there are many ground engineering situations where geosynthetics are only required to function at full capacity for a limited time period. If a conventional geosynthetic is used in these situations, for most of its working life it will be effectively redundant. Consequently, the construction work is overdesigned and, if the geosynthetic is a costly import, it will have a detrimental effect on both the economy and the industry of the locality where the work is being undertaken. In such situations, the use of a non-conventional geosynthetic which has a limited but predictable working life, i.e. a LLG, is good engineering practice, particularly if this geosynthetic is made from local, environmentally friendly, renewable resources.

Vegetable fibres are an ideal ‘raw material’ for the manufacture of LLGs; it is accepted that they ‘degrade’ with time but there is a huge range of natural fibres available, with some having very high initial tensile strength and with some exhibiting very slow progressive loss of strength with time. Hence, it is possible to ‘tailor’ composites of natural fibres to produce a material with the required strength–time profile for a variety of engineering situations (Sarsby *et al.*, 1992). Kumar *et al.* (2001) described a classical limited-life product for use in consolidation of soil.

12.3 Natural fibres as industrial materials

The exploitation of natural fibres in construction can be traced back at least to the

Table 12.2 Functional requirements for geosynthetic applications (adapted from Pritchard *et al.*, 2000)

	Degree of importance[a] of the following properties						
	Tensile strength	Elongation	Flexibility	Puncture resistance	Creep	Permeability	Resistance to flow
Reinforcement	H	H	L	**L**	H	**L**	
Filtration	**L–M**	**L–M**	**L–M**	M		M–H	**L**
Separation	H	M	H	H	L	M–H	M
Drainage	L	**L–M**	**M**	M–H		H	L
Erosion control	M	M	**H**	**L–M**	L	M	H

[a]H, highly important; M, moderately important; L, of some importance.

Table 12.3 Post-installation functional requirements for geosynthetic applications

	Degree of importance[a] of the following properties						
	Tensile strength	Elongation	Flexibility	Puncture resistance	Creep	Permeability	Resistance to flow
Reinforcement	**H**	**H**	**L**		**H**		
Filtration				**M**		**M–H**	
Separation	**H**	**M**	**H**	**H**	L	M–H	**M**
Drainage	**L**			**M–H**		H	**L**
Erosion control	**M**	**M**			**L**	**M**	**H**

[a]H, highly important; M, moderately important; L, of some importance.

Table 12.4 Ultimate load capacities of typical ropes

Rope, diameter (mm)	12	16	20	24	28	32
Rope, circumference (mm)	41	51	64	76	89	102
Coir, tensile strength (kN)	2	3	5	7	10	13
Sisal, tensile strength (kN)	11	17	27	39	50	65
Italian hemp, tensile strength (kN)	11	18	28	41	55	71
Hardy hemp, tensile strength (kN)	14	28	44	58	92	133

fifth and fourth millennia BC when dwellings were formed from mud–clay bricks reinforced with reeds or straw. Two of the earliest surviving examples of material strengthening by natural fibres are the ziggurat in the ancient city of Dur-Kurigatzu (now known as Agar-Quf) and the Great Wall of China (Jones, 1996). The Babylonians constructed the ziggurat some 3000 years ago using reeds in the forms of woven mats (laid in horizontal beds of sand or gravel at vertical spacings of between 0.5 and 2.0 m) and plaited ropes (approximately 100 mm in diameter) as reinforcement. It is believed that it was originally over 80 m high; even today it is 45 m tall. The Great Wall of China, completed circa 200 BC, utilized tamarisk branches to reinforce mixtures of clay and gravel.

A major element in the development of all natural fibres in the past 50 years has been competition with synthetics. The intensity of this competition on the different fibres has varied according to relative prices and their respective technical properties and their function. In many cordage and twine applications, synthetics have almost completely taken the market from natural fibres. Vegetable fibres are generally perceived to have inherently low tensile strength and poor durability when in contact with the natural environment. However, for many centuries ropes made from natural fibres were used as ships' rigging, to lift large loads at docksides and mines, etc. The significant load capacity of typical natural fibre rope is displayed in Table 12.4.

The first documented engineering use of a vegetable fibre textile fabric in Civil Engineering was in 1926, when the highways department in South Carolina undertook a series of tests, using woven cotton fabrics as a simple type of geotextile or geomembrane, to reduce cracking and ravelling failure of roads (Beckman and Mills, 1957). The basic system of construction was to place the cotton fabric on a previously primed earth base and to cover it with hot asphalt. Although results obtained before 1935 appeared to indicate some improvement in road performance, especially for fabric which had been in service for 9 years, further widespread development of this fabric as a geotextile did not take place. Unfortunately, the construction made the fabric perform more like a geomembrane than a geotextile, i.e. it separated the layers of the road rather than uniting them. Furthermore, the high extensibility of the cotton fabric and its poor durability negated the slight improvement in road performance that the fabric produced.

More recent usage of vegetable fibres in construction include the following.

1 Plant fibres bonded together in large numbers (as in wood) have found an application in the production of cement-based composites. Wood-fibre-reinforced cement products are widely available and combine the high tensile strength, impact resistance and workability of wood with the fire resistance, durability and dimensional stability of cement-based materials (Askew, 2000).
2 Sisal fibres have been used to reinforce cement-roofing sheets in East Africa since the 1970s (Mwasha *et al.*, 2002).
3 Jute, coir and straw continue to be used extensively in erosion control products in the form of nets, meshes, blankets and reinforcement mats which are laid directly on the ground surface (Mandal, 1989; Mandal and Murti, 1989; Rickson, 1994).

Currently, renewed interest in the use of natural fibres as engineered tensile materials in their own right is being led by the automobile industry. When used in appropriate situations, fibres such as flax, hemp and jute are cheaper, have better stiffness per unit weight and have lower impact on the environment than man-made fibres do. This could give considerable benefits by facilitating recycling, reducing energy consumption during vehicle production and improving day-to-day fuel economy. Schuhe (1999) reported that a weight reduction of about 20% has been achieved by using a plant-fibre-reinforced material consisting of a flax–sisal fibre mat embedded in an epoxy resin matrix for the door panels of cars. Furthermore, this substitution has actually enhanced the mechanical properties of the door panels.

Hence, when natural fibres are used in appropriate situations and within a suitable framework or form they can be successfully employed as flexible high-specification tensile elements.

12.4 Agro-industrial fibres

Vegetable fibres are usually classified according to the part of the plant from which they come (Lightweight Structures BV, 2005). Five different categories can be defined.

1 *Bast or stem fibres*, which are the fibrous bundles in the inner bark of the plant running the length of the stem. Retting is employed to free the fibres from the cellular and woody tissues, i.e. the plant stalks are rotted away from the fibres. Examples include coir, flax, hemp, kenaf, ramie, rosette and rena.
2 *Leaf fibres*, which run the length of the leaves. The fibres are extracted by retting and scraping the pulp from the fibres. Examples include banana, sisal, henequen, abaca, pineapple, cantala, mauritius and phormium.
3 *Seed-hair and fruit fibres*, which are produced by a plant to give protection to seeds and fruit. Examples include coir, cotton, kapok and milkweed floss.
4 *Core, pith or stick fibres*, which form the low-density spongy inner part of the stem of certain plants.
5 *Plant fibres* not covered by the foregoing categories.

Table 12.5 Major potential fibre sources (worldwide) (after Rowell and Jacobson, 2002)

Source	Dry mass ($\times 10^6$ t)
Wood	1750
Straw (barley, flax, oats, rice, rye and wheat)	1145
Stalks (corn, cotton and sorghum)	970
Sugar cane bagasse	75
Reeds	30
Bamboo	30
Cotton staple	15
Core (coir, hemp and kenaf)	8
Papyrus	5
Bast (coir, hemp and kenaf)	3
Cotton linters	1
Esparto grass	0.5
Leaf (abaca, henequen and sisal)	0.5
Sabai grass	0.2

While individual single fibres in all the categories are quite short (except for flax, hemp, ramie, cotton and kapok), fibre bundles can be quite long. For example hemp, coir and kenaf can have fibre bundles as long as 4 m and abaca, mauritius and phormium are about half this length.

Major fibre sources with potential for usage within an agro-industrial context, such as the production of vegetable fibre geotextiles for use in a wide variety of construction scenarios, are indicated in Table 12.5. Hard fibres (such as abaca, sisal, henequen and coir) are relatively unimportant in terms of the global value of international trade. However, they provide significant economic support to the population in certain impoverished and least-developed areas of a number of producing countries. No hard fibre shares a major end use with another and for the most part, no single country produces any significant quantity of more than one hard fibre. Modes of production differ significantly between fibres and between producing regions.

1 In African countries, sisal is produced essentially on single-crop holdings.
2 Brazil produces sisal as a part of a mixed agriculture.
3 Abaca is grown largely in the Philippines as a secondary crop.
4 Coir is produced as a by-product of other coconut products.

Around the world there are copious supplies of cheap indigenous fibres which local textile industries can use to replicate common geotextile forms that are made from man-made fibres (Booth *et al.*, 2005). In many cases, these vegetable fibres are waste products, by-products or co-products of crop systems and food production (Ali, 1992). Specific advantages to be gained from the development of vegetable fibre geotextiles using indigenous materials include:

1 The raw materials employed are 'environmentally friendly' because they are a renewable resource and they can be returned to the ground without leaving a pollution residue.
2 The fibres have low unit cost (at the point of production) and the conversion costs are usually low (Azziz, 2000); however, if the finished product has to be transported any distance to its point of usage, then the expenditure on delivery becomes an excessive proportion of its value.
3 Natural fibres have significant initial strength (in some cases they are superior to man-made fibres) and they can be very resistant to harsh environments.
4 Suitable materials are often readily available in developing countries and frequently are a major contributor to the local economy (Shamte, 2000).
5 They create new markets for agricultural products by introducing additional uses of by-products or new uses for waste agricultural products and increase the range of crops (including some that are currently regarded as weeds) that farmers can grow and sell. Because the new markets are primarily close to the production point, money is attracted into rural areas and regions without incurring major expenditure on transportation.

However, if vegetable fibres are to become widely used to produce high-specification technical materials which supplant products made from man-made fibres, there are factors to be considered which would not be an issue for man-made polymeric materials:

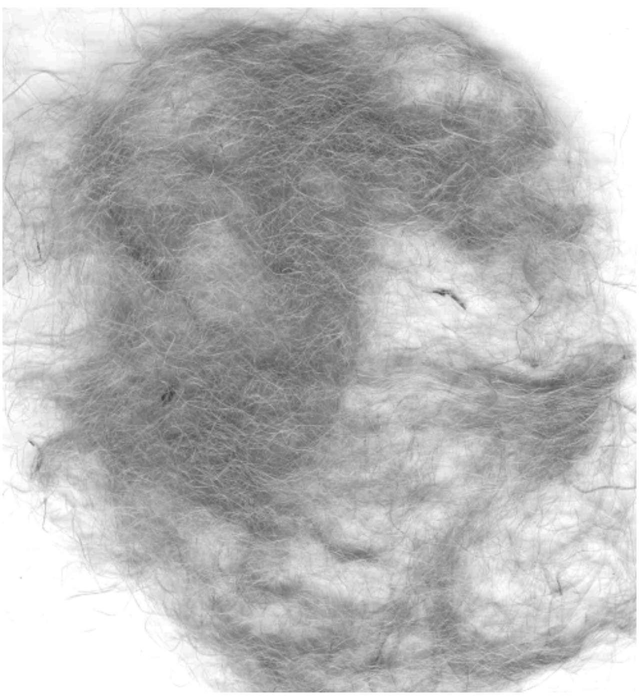
1 Any method for extracting the fibres from the parent plants must not cause serious damage (such as overstressing, cracking, weakening and permanent plastic deformation) to the fibres.
2 The relevant properties of the fibre must be equivalent or superior to those of the existing chemical fibres used for the same purpose, in terms of manufacturing capability, product quality and consistency, stability (with regard to both ambient conditions and time) and ease of utilization.
3 There must be consistent annual production of the fibre. Unfortunately, for any agricultural crop there are a large number of factors (a number of which are not under the agriculturalist's control) which can cause variability of the final product, such as:
   (a) Climatic conditions during the growing season (usually harvesting has to be done at certain, more or less fixed times of the year).
   (b) Plant diseases and insect attack.
   (c) Harvesting methods.
   (d) Conditions during the harvesting period.
   (e) Fibre extraction and separation processes.
   (f) The conditions (temperature, humidity, residence time, etc.) during fibre extraction and separation.
   (g) Storage method and conditions, and duration of storage.
   (h) Handling and shipping.

4 Inherent reluctance within industry (particularly the construction industry) to use new untried materials.

Of the 2000 or so fibre-yielding plants throughout the world there are some 15–25 plants that satisfy the criteria for commercial fibre exploitation although a number of these are only farmed on a small scale Pritchard (1999). These main fibres are are as follows.

1 Bast fibres: flax, hemp, jute, kenaf, nettle, ramie, roselle, sunn and urena.
2 Leaf fibres: abaca, banana, cantala, date palm, henequen, New Zealand flax, pineapple and sisal.
3 Seed and fruit fibres: coir, cotton and kapok.

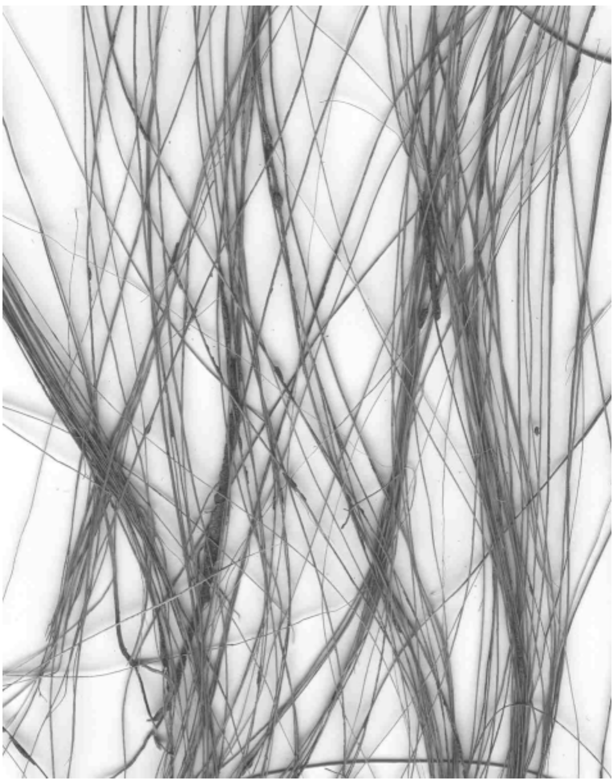
However, when consideration is given to relevant technical properties (such as strength, extensibility or stiffness, flexibility and durability), ready availability of large quantities at different times within a year, variability of products, manufacturing capacities, etc., the list of promising vegetable fibres is much smaller. Cotton, which has the highest annual production rate, has existing markets and also is not capable of satisfying high-strength requirements when buried in the ground (but it still can be used as a weft 'distribution' fibre in geotextile products). Currently the most promising fibres (in order of decreasing annual production tonnage) seem to be jute, flax, coir, sisal, hemp and abaca, (Tables 12.6, 12.7, 12.8, 12.9 12.10 and



12.5 Jute fibres.

Table 12.6 Jute fibres

Growing	Bast fibre; annual plant; stem diameter, 20 mm; stem length, 2.5–3.5 m
Production	Grown in the India subcontinent, the Far East, Africa, Asia, and Central and South America. Easily cultivated and harvested. Plants easily damaged by excessive heat, drought, rainfall and floods and by pests (semilooper, mite, caterpillar and apion). Second most important fibre worldwide in terms of cash and acreage. Fibre extraction – retting. Oil and water emulsion is added to soften the fibre for spinning into yarns
Uses	Cheap and used in great quantities. Ropes, bags, sacks and cloths. Erosion control applications include 'geojute'and 'soil-saver'
Properties	Poor tensile strength, not as strong as hemp and flax nor as durable. If kept dry, will last indefinitely. Fibre deteriorates rapidly when exposed to moisture. High initial deformation modulus, but very little recoverable elasticity; exhibits brittle fracture



12.6 Coir fibres.

12.7 Sisal fibres.

Table 12.7 Flax fibres

Growing	Bast fibre; annual plant; stem diameter, 16–32 mm; stem length, 0.9–1.2 m; harvested after 90 days
Production	Grown predominantly in Russia (80%), China, Egypt, Turkey, Philippines, Malaysia and UK. Apart from fleas and beetles, flax is not generally vulnerable to pests. Third most important fibre worldwide in terms of cash and acreage and ranks fourth for the total fibre production. Fibre extraction – retting. After retting, fibre is broken away from the stems and combed
Uses	Linen, twines, ropes, fishing nets, bags, canvas and tents. Tow fibre, and high-grade paper, i.e. cigarette paper and banknotes. Linseed oil and linseed flax fibre
Properties	Fibre strength increases when wet. Physical and chemical properties are superior to cotton. Relatively inextensible fibre; more elongation obtained when dry. One of the highest tensile strengths and moduli of elasticity of the natural vegetable fibres. Density same as polymers

12.8 Hemp fibres.

Table 12.8 Coir (coconut fibres)

Growing	Seed and fruit fibre; perennial plant; fruit picked every alternate month throughout the year; economic life, 60 years. The fibre is produced from the coconut husk
Production	Grown in India (22%), Indonesia (20%), Sri Lanka, the Far East, Brazil, the Philippines, East Africa, Latin America and throughout the Pacific regions. Fibre derives from the nut of a tree that has an economic life of 60 years. Grows on a wide range of soils; prefers high humidity and plenty of sunlight. Ranks fourth in world annual production. Fibre extraction – retting; dehusked manually or mechanically
Uses	It has many established uses, including as a filling in upholstery, mattresses and car seats and in the manufacture of doormats, rugs, brooms and brushes, ropes and cordage
Properties	Abrasive and rot resistant under wet and dry conditions and retains a high percentage of initial tensile strength. Resistant to degradation by sea water; endures sudden pulls that would snap much stronger ropes. Erosion control products from coir are well established in applications that demand limited life at low cost

Table 12.9 Sisal fibres

Growing	Leaf fibres; perennial plant; leaves, 1–2 m long, each containing about 1000 fibres
Production	Grown in Central America, Mexico, Brazil, the Philippines, East Africa and Venezuela. Prefers temperatures between 27 and 32 °C; optimum rainfall, 1200–1800 mm but can withstand droughts; requires substantial amounts of strong sunlight; waterlogging and salinity are fatal to sisal. The fibre extraction method includes retting (soaking in water), followed by beating and scraping of the leaves so that only fibres remain and all other parts are washed away by water. The fibres are then thoroughly dried and bleached in the sun or oven dried. Sisal is not generally prone to pest attack and, provided that the climatic conditions are not unfavourable, a consistent annual harvest and fibre product is obtained
Uses	Twines, ropes (widely used in marine environments), rugs, sacking, carpets, cordage and agricultural. Tow (waste product) used for upholstery. Traditionally used primarily for harvest (baler) twine and other cordage; the proportion being used for traditional purposes is decreasing and more is being used for new uses such as carpets and paper
Properties	Sisal is resistant to salt water. Shorter, coarser and not quite as strong as abaca. Also lower breaking load and tends to break suddenly without warning. Can be spun as fine as jute but stiff and somewhat inflexible

Table 12.10 Hemp fibre

Growing	Bast fibre; annual plant; stem diameter, 4–20 mm; stem length, 4.5–5 m; harvested after 90 days
Production	Grown in Russia, Italy, China. Yugoslavia, Romania, Hungry, Poland, France, Netherlands, UK and Australia. No pesticide protection required for growth. Fibre extraction – retting. Separation of the fibre from the straw can be carried out mechanically (green hemp)
Uses	Ropes, marine cordage, ships sails, carpets, rugs, paper, livestock bedding and drugs
Properties	Not weakened or quickly rotted by water or salt water. Stronger, more durable, stiffer, more rigid and coarser than most vegetable fibres. Suitable for weaving of coarse fabric

12.11, respectively). Figures 12.5, 12.6, 12.7 and 12.8 show jute, coir, sisal and hemp fibres, respectively.

12.5 Vegetable fibre characteristics

Table 12.12 contains indicative data about fibre composition and physical charac-

Table 12.11 Abaca or manila hemp fibres

Growing	Leaf fibre; perennial plant; 12–30 stems per plant; leaves, 2–4 m. Plant life, 10–20 years; productive life, 1–8 years; harvest 3 stalks every 4–5 months
Production	Grown in the Philippines (85%) and Ecuador (15%). Needs a warm climate, shade, abundant moisture (rainfall of 2.5–3.0 m distributed uniformly through the year) and good drainage. Too much heat causes damage to leaves and fibres. Attacked by brown aphids, corm weevil and slug caterpillar. Relatively small annual production tonnage. Fibre obtained from the stem of the leaves and not the expanded portion of the leaf. Fibre extraction – retting. Fibre extracted by separating the ribbons of the fibre from the layers of pulp and then hung to dry
Uses	Marine cordage (naturally buoyant), fishing nets, ropes, tea bags, coffee filters and currency notes. Was traditionally used for ropes and other cordages but now is almost entirely pulped for special papers
Properties	Good water-resisting properties; hydroscopic; not affected by salt water. Superior to flax; better than hemp for marine ropes and hawsers. Very light weight. Strong and sufficiently flexible to provide a degree of give when used in ropes

teristics. It is very difficult to quantify and assign precise values to the characteristics of individual vegetable fibres because of the inherent variability of their dimensions as a result of their agricultural origins. Within the textile industry, this difficulty is overcome by measuring a form of average value, i.e. it is normal practice is to quote strength and tensile modulus in terms of tex which is the mass in grams of 1 km of fibre. Hence, any strength or modulus will be based on the behaviour of a number of fibres rather than on individual units. For instance, the tenacity (ultimate load per tex) for cotton is in the region of 0.35 N/tex (this corresponds to an ultimate tensile stress of about 0.5×10^6 kN/m^2) but individual fibres can easily have tensile strengths within ± 20% of this value. For flax, abaca and sisal the tenacity is between 0.4–0.6 N/tex (corresponding to a tensile strength of $(0.4–1.0) \times 10^6$ kN/m^2); the tenacity of ordinary chemical fibres (polyester) is around 0.4 N/tex (Pritchard, 1999). Carefully separated individual flax fibres have a strength of around 10^6 kN/m^2 and modulus of 80×10^6 kN/m (Leflaive, 1988), i.e. of the same order as Kevlar, a very-high-strength chemically modified polyamide fibre.

It is readily apparent that economically-viable vegetable fibre fabrics (a collection of numerous fibres) cannot be guaranteed to match the tensile properties of steel or solid polymeric strips or sheets (Table 12.13); synthetic fibres have a high strength, high tensile modulus and limited elongation to failure. However, natural fibre products can be manufactured which have similar technical characteristics to geotextiles made from man-made fibres (Table 12.13). On the other hand, fibres such as cotton, jute and coir (woven and fibre form) have a number of advantageous characteristics which are not possessed by man-made fibres.

Table 12.12 Dimensions and chemical composition of some common agro-fibres (from Esau, 1977; Iivessalo-Pfaffli, 1995 and Han and Rowell, 1997)

Type	Cellulose (%)	Lignin (%)	Mean length (mm)	Mean width (mm)
Abaca	56–63	7–9	6	0.024
Bamboo	26–43	21–31	2.7	0.014
Cereal straw	31–45	16–19	1.5	0.023
Coir	35–63	30–45	2.5	0.020
Coniferous wood	40–50	26–34	4.1	0.025
Corn straw	32–35	16–27	1.5	0.018
Cotton	85–90	0.7–1.6	25	0.020
Deciduous wood	38–49	23–30	1.2	0.020
Esparto	33–38	17–19	1.9	0.013
Hemp	57–77	9–13	20	0.022
Kenaf	44–57	15–19	2.6	0.020
Papyrus	38–44	16–19	1.8	0.012
Rice straw	28–36	12–16	1.4	0.008
Seed flax	43–47	21–23	30	0.020
Sisal	47–62	7–9	3.3	0.020
Sugar cane bagasse	32–37	18–26	1.7	0.020
Wheat straw	33–39	16–23	1.4	0.015

Table 12.13 Stress–strain data for some reinforcing materials

Reinforcing material	Ultimate tensile strength ($\times 10^3$ kN/m^2)	Modulus of elasticity ($\times 10^3$ kN/m^2)	Strain at failure (%)
Steel strip	410	210 000	5
Plastic grid	520	5 300	15
Geotextile	85	850	20
Jute rope	49	340	33
Coconut rope	14	16	85

1 They can absorb and store moisture.
2 Their natural flexibility allows them to conform closely to a soil profile.
3 Their bulk gives a high protective cover–weight ratio.
4 Their production is compatible with the surrounding environment and does not use scarce resources or cause pollution.

To appreciate the similarity of technical characteristics, the ultimate strength, extension at failure and tensile modulus of various natural fibres have been expressed as a percentage of the values for polyester and are presented in Table 12.14. This table clearly demonstrates the following.

1 Many vegetable fibres have similar ultimate strengths to polyester (as indicated by tenacity in Table 12.14).

Table 12.14 Natural fibre properties as a percentage of polyester values (after Bisanda and Anselm (1992), Chand *et al*. (1986, 1988) and Mukherjee and Satyanarayana (1984))

Natural fibre	Tenacity	Failure elongation	Tensile modulus
Abaca	156	12	278
Coir	33	20	33
Cotton	76	20	78
Flax	111	12	211
Hemp	100	12	222
Henequen	56	20	167
Jute	89	8	189
Ramie	118	16	167
Sisal	89	12	222
Viscose	44	80	56
Wool	33	140	22
(Polyester	100	100	100)

2 The rupture strain of vegetable fibres is much smaller (by a factor of between 5 and 8 approximately) than that of polyester.
3 The tensile moduli of many 'fresh' vegetable fibres are significantly higher than that of polyester.

However, in order to utilize the foregoing advantageous properties in engineered design, it must be remembered that all natural fibres will biodegrade in the long term as a result of the action of the micro-organisms and this must be clearly accounted for within any design methodology.

The problem of durability of natural fibres is complex and contradictory, and examples of both very fast decay and remarkable stability are cited. However, the material degradation rate is determined by the actual combination of relevant factors (ambient moisture content, degree of acidity or alkalinity, etc.) rather than just on the factors themselves and, for natural fibres, the most important factor is whether the ambient conditions favour the growth of bacteria which will feed on the fibres. Hence, natural fibres have been used extensively to make sails and fishing nets, which have performed satisfactorily under hostile conditions (saline water, periodic wetting and drying) but within a relatively sterile environment.

In the 1920s and 1930s, an extensive investigation was undertaken by the then Imperial Institute into the suitability of sisal for the manufacture of marine cordage. Numerous samples of sisal rope were subjected to cyclic wetting (with seawater) and drying over a period of 12 months. Table 12.15 shows results from a preliminary study of the percentage loss of tensile strength of three types of sisal rope (Anon., 1927). These data show the following two potential concerns with regard to the use of this type of material to create LLGs.

1 There are significant differences between the durability characteristics of the three ropes types (and each value is in fact an average of several values).

2 The ropes exhibited high rates of loss of tensile strength and integrity with immersion time.

Subsequent investigations of the influence of exposure conditions on the loss of tensile strength of sisal ropes showed similar trends for specimen subjected to cyclic wetting with seawater (Table 12.16) (Anon., 1935). However, these investigations also showed the following.

1 Sisal ropes kept in relatively dry conditions, i.e. in conventional covered storage sheds, lost very little tensile strength with time.
2 Sisal ropes subjected to cycles of wetting (with rainwater) and drying (by wind and sunlight) underwent progressive strength loss with time (at a similar rate to that determined more recently (Pritchard, 1999). However, the time-dependent strength loss was at a sufficiently slow rate to be able to satisfy economically typical durability requirements for LLGs.
3 Sisal ropes which were tarred in the usual manner of the time, i.e. by being passed through a bath containing tar, and were then immersed in seawater showed a much lower rate of strength loss than the untreated rope.

In more recent times, further research has been conducted with a view to reducing the bio-degradation rate of natural fibres. For instance, rotproofing of cotton and flax used in tarpaulins, tents, etc., could be achieved by treating the fabrics with fungicides such as pentachlorophenol esters (Hamlyn, 1990); the average life of

Table 12.15 Percentage loss of tensile strength of sisal ropes immersed in seawater

Rope type	Loss after the following exposure periods			
	4 months	6 months	9 months	12 months
East African sisal, quality 1	35	47	51	63
East African sisal, quality 2	51	60	65	75
Manila hemp	42	51	53	69

Table 12.16 Percentage loss of tensile strength of sisal ropes according to exposure

Exposure conditions	Loss after the following exposure periods			
	2 months	4 months	6 months	9 months
Immersion in seawater	16	22	57	76
In covered storage	0	4	4	4
Uncovered, on the laboratory roof	1	4	11	12

jute bags was increased sixfold when the fabric was treated with copper salts such as copper sulphate, copper ammonium sulphate, copper acetate or copper naphthenate (MacMillan, 2005); sisal fibres treated with toluene-di-isocyanate derivatives showed significantly greater durability and lower water absorption than untreated fibres (Mwasha, 2005). However, the foregoing forms of treatment vastly reduce the 'environmental friendliness' of vegetable fibres. The real key to developing geosynthetics from biodegradable natural fibres is the concept of designing by function, i.e. identifying the functions and characteristics required to overcome a given problem and then selecting appropriate fibres and manufacturing the product accordingly. When correctly designed natural fibre materials can compete with synthetic materials, sometimes they will even have superior performance.

Natural fibre geotextiles may be used where the design life of the fabric structure is relatively short or where strength or integrity requirements decrease with time – note that the definition of a short-term time scale varies from site to site and from application to application.

12.6 Erosion control

Currently, the major use of natural fibre geotextiles is in the erosion control industry (Mitchell *et al.*, 2003). Soil erosion is the removal of surface layers of soil and it involves a process of both particle detachment and particle transport by the disturbing agencies. Erosion is initiated by drag, impact or tractive forces acting on individual particles of soil. The two most common agents of erosion are rainfall and wind. This erosion is controlled by a number of soil, climatic and topographical factors including intensity and duration of precipitation, ground roughness, length and steepness of slope, inherent soil strength, and type and extent of cover (Morgan *et al.*, 1984; Gray, 1991; Rao and Balan, 2000). Surface erosion by water commences when the kinetic energy of rainfall is transferred to individual soil particles, breaking the bonding between particles and moving the particles upwards and laterally. On level ground the net transport of soil particles will be almost zero since the soil particles splash uniformly in all directions. However, on a slope, more soil will be transported downhill by the impact (Ellison, 1944).

If rainfall exceeds the infiltration capacity of a soil, continued precipitation results in overland flow which begins at a shallow depth (thin-film flow). The run-off water collects in small rivulets which may erode very small channels (rills). These rills may eventually merge into larger and deeper channels (gullies). Figure 12.9 shows gullies formed by water running down the surface of a tip of waste material (which is a difficult medium on which to establish good vegetation cover). If the soil is permeable and has a favourable structure, infiltration will be enhanced and overland run-off will be reduced. If the energy of the falling rain can be absorbed or dissipated by vegetation or by impacting on some other soil cover or

12.9 Eroded surface of a waste tip (note the sparseness of the vegetation).

surface obstruction, energy transfer to the soil particles which results in detachment will be reduced and there will be a consequent reduction in soil erosion. Slowing the flow of water downslope reduces the soil transport capacity of the thin sheet flow, thereby minimizing the displacement of dislodged soil particles.

Ground erosion is often the result of human activity, e.g. formation of cuttings and embankments when roads are built, creation of waste tips, and placement of final caps over engineered landfill sites. Promotion of vegetation on the final caps of landfill sites is very important for long-term effectiveness of the cap (prevention of water infiltration and prevention of uncontrolled gas escape, which requires the cap to be structurally intact and stable). Figure 12.10 shows a vegetable fibre net placed on top of a landfill cap to provide anchorage for vegetation.

It has been estimated that soil erosion by water and soil erosion by wind is responsible for about 56 and 28%, respectively, of worldwide land degradation (Wibisono, 2000). Furthermore, the US Army Corp of Engineers has estimated that in the USA alone the damage caused by soil erosion costs at least £140 million annually (Wibisono, 2000).

12.6.1 Surface erosion control

Erosion control measures can be classified into three broad categories (with the applicability of the method depending on the classification of the site): agronomic, soil management and mechanical (Ingold and Thomson, 1990).

In nature, vegetation (which is in effect an agronomic erosion control solution)

12.10 Vegetable fibre net on a landfill final cap.

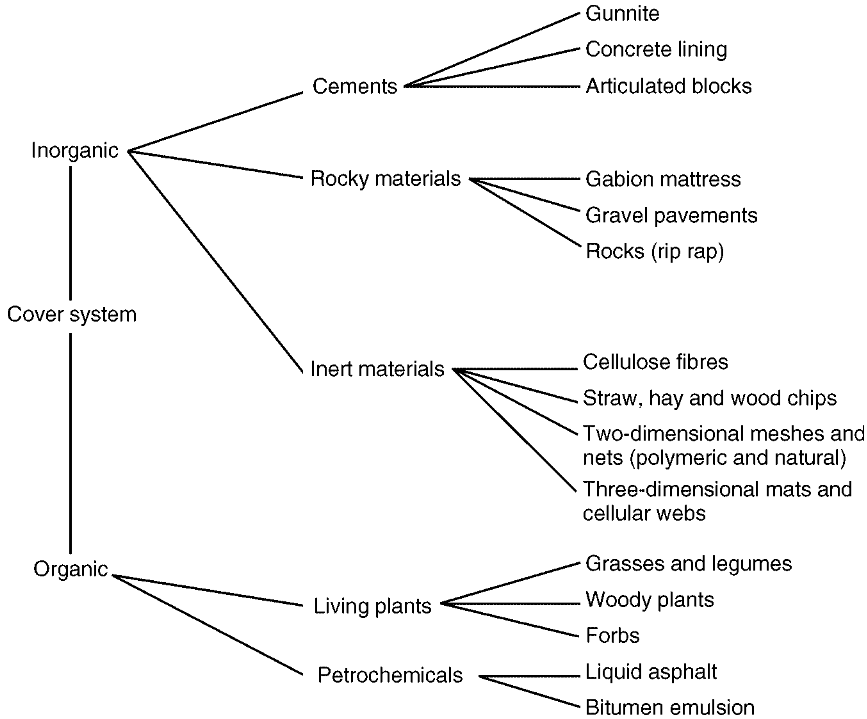
is the major factor in the prevention of erosion of soil slope by the following mechanisms.

1 Binding and restraining soil particles in place (root action).
2 Filtering soil particles out of run-off (performed by stems and organic litter).
3 Intercepting raindrops (canopy action).
4 Retarding velocity of run-off (undertaken by stems and organic litter).
5 Maintaining infiltration (root action).

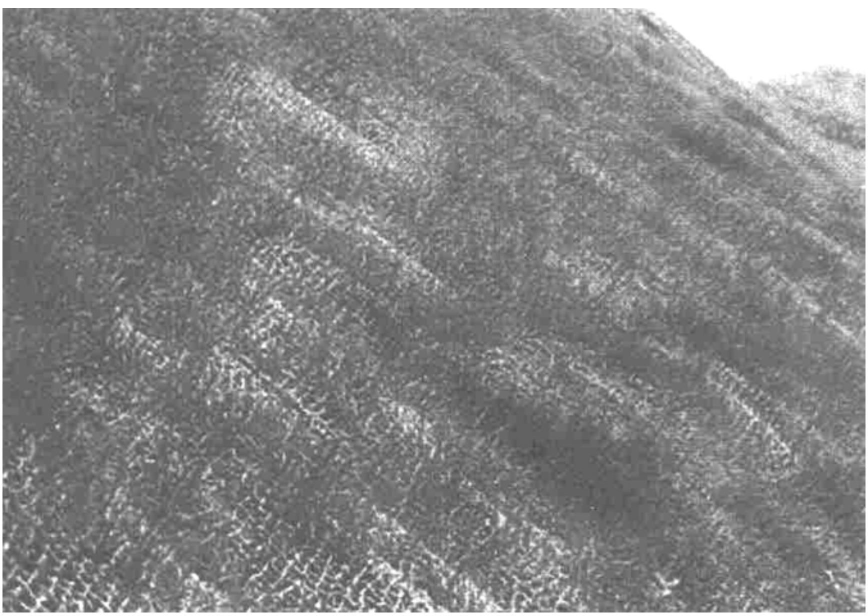
Artificial ground cover is the most common solution for 'interventional' erosion protection with dual functions of blocking out the light (thereby depriving weeds of growth potential) and providing a favourable microclimate for establishment of imported plants. This solution is a mixture of soil management and mechanical stabilization. The ground cover ranges from inert facings (concrete and stone layers), to organic mulches (hay, wood chips, straw, etc.) and live materials (seedlings and cuttings). Different types of ground cover for erosion control are outlined in Fig. 12.11 (as explained in detail by Theisen, 1992).

When geosynthetics are used as erosion control products, they are intended to simulate the properties of vegetation and also to provide assistance to vegetation to establish itself permanently. Figure 12.12 shows a new slope which has been covered with a vegetable fibre geotextile to provide anchorage and 'agricultural support' for vegetation. Different types of geotexile simulate different functions of natural vegetation cover as indicated in Table 12.17.

The prime requirements of a mulch is to provide tight ground cover to suppress weed growth and to enhance the growth of vegetation by providing a beneficial microclimate around plants. Thus, there is no need for longevity, and strength is needed primarily for handling purposes.



12.11 Surface erosion control methods.



12.12 Vegetation growing through netting covering a new slope.

Table 12.17 Simulation of functions of vegetation (adapted from Rao and Balan, 2000)

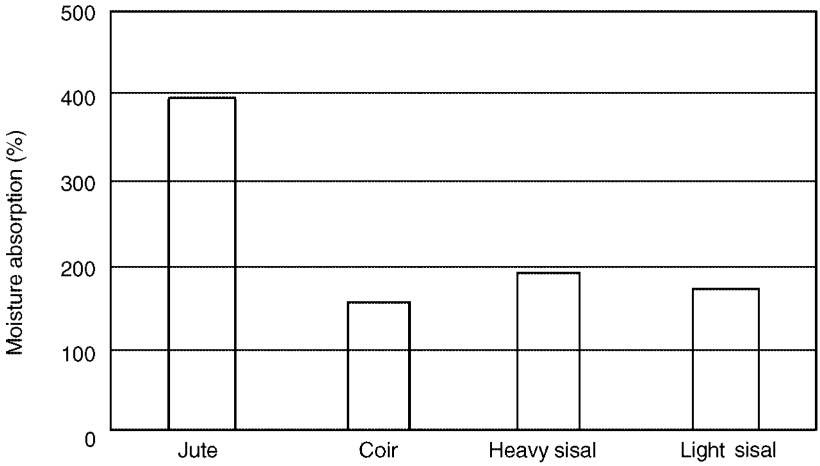
Geotextile type	Degree of similarity[a] for the following components of natural vegetation			
	Canopy	Stems	Roots	Litter
Woven meshes (often natural fibres)	H	H	L	M
Two-dimensional mats (often natural materials)	H	M	L	H
Three-dimensional meshes (often using man-made materials)	L	L	H	L
Geocells, geowebs (synthetic materials)	L	L	M	L

[a]H, high similarity to nature; M, medium similarity to nature; L, low similarity to nature.

Geotextiles used in soil erosion control are often very effective in controlling rates of soil detachment by raindrop impact. Geotextiles used in soil erosion control can be natural or synthetic materials and may be installed on the surface or buried within the ground. Like vegetative canopies, geotextiles will intercept and store rainfall water. The composition of some products (especially the natural fibre geotextiles such as coir) imparts a roughness to run-off in much the same way as vegetation stems. Buried geotextiles simulate the root effect and biodegradable geotextiles act as a mulch in controlling soil erosion. Geotextiles reduce soil erosion by absorbing the impact and kinetic energy of falling raindrops and checking surface run-off. Seeds and vegetation are protected from being washed away.

The ability of natural fibres to absorb water and to degrade with time (thereby contributing nutrients to vegetation and eliminating obstruction of growth of vegetation) are the prime properties which give their products an edge over synthetic geotextiles for erosion control purposes. The 'drapability' of natural fibre geotextiles allows them to conform closely to the terrain, i.e. to follow the contours of a slope and to stay in intimate contact with the soil. Hence, vegetable fibre geotextiles can be used where vegetation is considered to be the long-term answer to slope protection and erosion control. They have a number of inherent advantages.

1 They give protection against rainsplash erosion.
2 They have the capacity to absorb several times their own weight as indicated in Fig. 12.13.
3 They reduce the velocity of run-off and hence its erosive effect. Because of their thickness and the apertures and depressions of natural-fibre geotextiles function as a series of small check dams and weirs.
4 They maintain humidity and moisture in the soil (by covering the ground) and in the air directly above the ground (by retaining water within apertures and within themselves).



12.13 Water absorption capacity of natural fibres.

5 They mitigate against extreme temperature variation of the soil (by providing shade during the day and insulating cover at night).
6 They biodegrade, adding useful mulch to the soil, and, because they have not been treated with chemicals, there is no associated ground pollution.

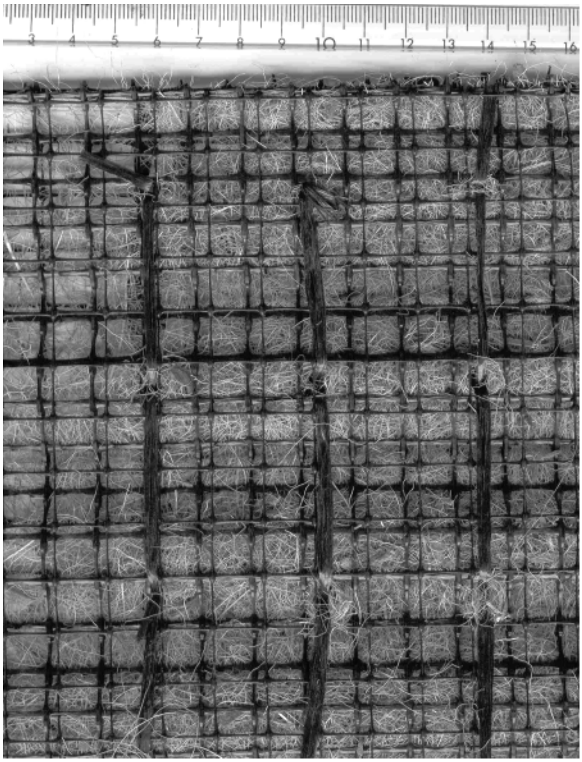
Even though synthetic geotextiles last for a long time in surface erosion control applications, studies have indicated that they are less effective than their natural fibre counterparts (Rickson, 1994). However, some jute-based geotextiles have been found to degrade completely within a year (which is usually too short a time for full development of self-sustaining vegetative cover). On the other hand, coir-based geotextiles have been found to persist and retain their erosion control functions for at least three years.

The vast majority of natural fibre erosion control products are applied to the surface of the ground but there has been some usage of these products to control riverbed erosion. Levillain (2000) described how vegetable fibres were used to protect the Pont de Pierre in Bordeaux when it was threatened by severe erosion of the riverbed adjacent to the foundations of the bridge. The protection employed involved placing prefabricated gabions on top of polymeric geotextiles (acting as a filter) laid on the riverbed. The gabions were then covered with a heavy vegetable fibre 'carpet' for a distance of 35 m upstream and downstream of the bridge foundations over the entire width of the Garonne river.

12.6.2 Erosion control materials

Products can be classified generally as follows.

1 *Nettings.* These are two-dimensional woven natural fibres or geosynthetic (polymeric) biaxial mattings (Fig. 12.14). After the proposed slope is seeded

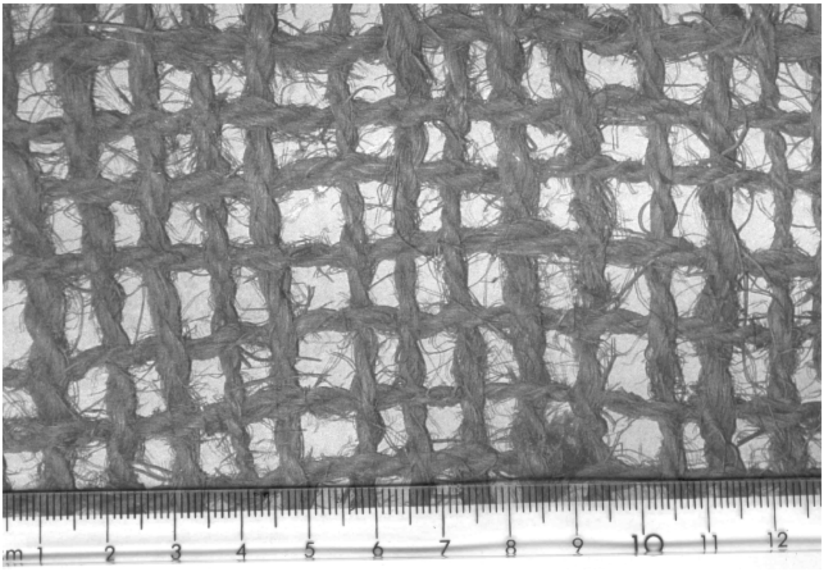


12.14 Composite natural fibre (coir) and geosynthetic erosion control matting.

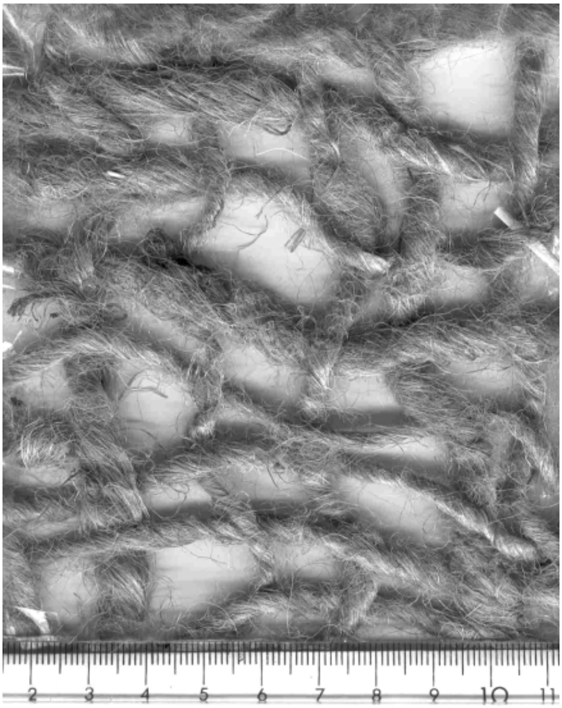
and/or mulched, these materials are laid and fixed in position. Used with loose mulch they yield good performance for moderate conditions (slope angle, slope length and run-off condition). This type of product generally has a design working life of one or two growing seasons.

2 *Meshes.* These are open weave geotextiles formed into a two-dimensional matrix (Fig. 12.15 and Fig. 12.16). The construction of these materials enables erosion control to be provided with or without underlying loose mulch layer. They generally have higher tensile strength than nettings and can be used on steeper slopes.

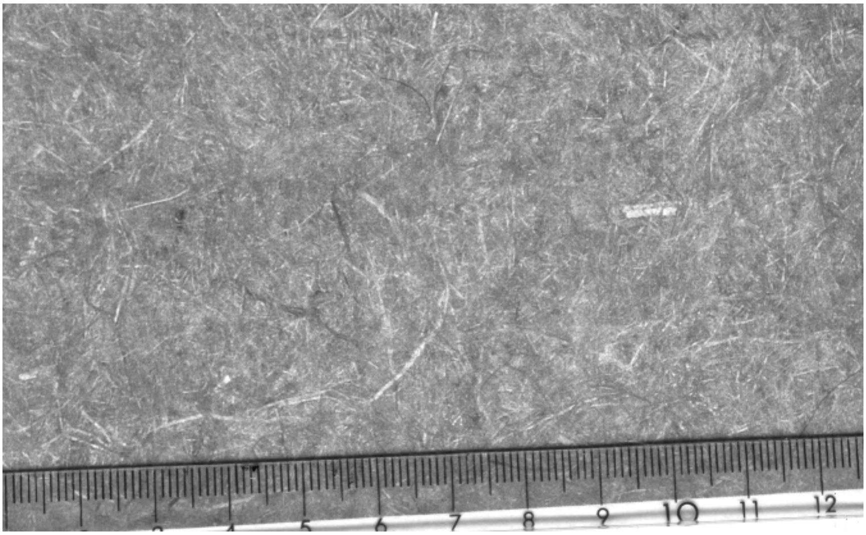
3 *Blankets.* These are constructed of various degradable organic or synthetic fibres that are woven, glued or structurally bound with nettings or meshes (Fig. 12.17). The most widely used erosion control blankets are made from straw, wood shavings, coconut fibre and polypropylene stitched or glued or glued to nettings or meshes. Some materials are available with seeds pre-incorporated into their structure. They are rolled out in intimate contact with the soil surface and anchored with staples or pegs. Applicable to sites requiring



12.15 Coir erosion control mesh.



12.16 Jute erosion control mesh.



12.17 Erosion control blanket (made from coir fibres).

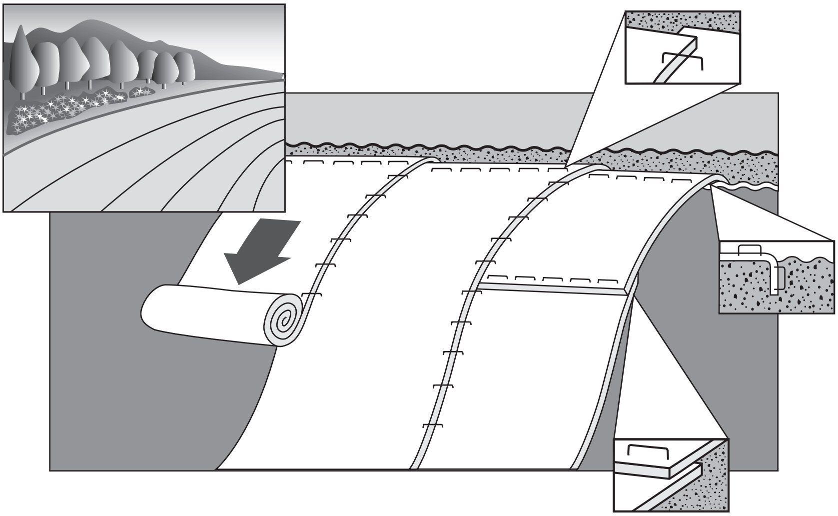
greater, more durable and/or longer-lasting erosion protection, they can be used on gentle to steep slopes. Products usually have a design functional life span of 1–5 years.

Geotextiles made purely of jute fibres (also known as geojute) have been in use since the 1950s when an open-meshed woven fabric was used in Europe and the USA to cover exposed soil surfaces with a view to promoting vegetation growth and thus arrest soil erosion (Mandal, 1988). Geojute has many limitations as a geosynthetic, primarily because its strength and durability are too limited for harsh applications such as steep slopes, high-altitude slopes or waterways; it degrades particularly quickly in very humid conditions. On the other hand, coir has only been used in geotextiles over the past 25 years approximately but it is a much more durable material; it takes coir around 15 times longer than cotton and around seven times longer than jute to degrade completely.

12.6.3 Erosion control material installation

The material is usually shipped to site in rolls. When wet, the natural fibre materials should never be stored indoors without very good ventilation as otherwise there will be promotion of microbial action, decay and loss of strength. If the material is kept outside before installation, it should be protected against rainfall and kept clear of the ground to provide air circulation.

A drawback to the use of natural fibre geotextiles is that storage for a prolonged period requires significant attention to temperature, humidity, air circulation and



12.18 Installation of erosion control materials (after Rao, 2000).

stacking to minimize risk of moisture gain. Whilst the moisture absorption or retention capacity of natural fibre geotextiles is advantageous once they are in position, it should be borne in mind that, if rolls of material are allowed to absorb water before installation, their weight will increase significantly and they will become difficult to handle and work with.

Before installation of the product, the topsoil on the slope should be worked to a fine tilth free of clods or loose stones. Seed, mulch and fertiliser should be distributed evenly over the prepared soil (an alternative is to apply a hydromulch, i.e. a wet mix of mulch and seeds which is sprayed on to the protected surface). Wherever possible the geotextile should be applied by unrolling down the slope; however, if required the geotextile can be run along the slope. The geotextile is secured with staples or pegs (at least 150 mm long) driven into the ground (Fig. 12.18). At the crest and toe of the slope the fabric is usually anchored by burying it in a trench (about 500 mm wide and 300 mm deep) and additional anchorage is achieved by driving in a row of staples or pegs along the crest and toe. Longitudinal edges of rolls are overlapped by 100 mm and stapled or pegged at 500–1000 mm centres according to the risk of lifting of the fabric (possibly due to the exposed nature of a particular site). Roll junctions are made by lapping the end of the upslope roll 200 mm over the end of the downslope roll and securing the overlap with a row of closely spaced pegs or staples (at around 200 mm centres) as illustrated in Fig. 12.18.

12.7 Basal reinforcement of embankments on soft soil

One of the standard and most effective ground improvement techniques used with soft soil subgrades is surcharging or preloading. This process usually involves constructing a temporary embankment which loads the ground and induces consolidation and settlement. The imposed load is left in place until the voids ratio of the ground has been sufficiently reduced and then it is removed. Basal reinforcement (usually in the form of geotextiles composed of man-made fibres) is frequently needed to enable an economic stable embankment to be built on such weak subgrades. As the foundation soil consolidates, owing to the weight of the embankment, it gains in shear strength. Hence, this basal reinforcement is a classic example of a decreasing need for functionality (in this situation, the input of a restraining horizontal force) of a geotextile with time. LLGs would be an extremely effective method of providing the basal reinforcement if readily available indigenous vegetable fibres could be used to manufacture these materials. A practical example of the technical, social and human benefits of the foregoing approach would be the use of coir and banana fibres (both readily available in Sri Lanka) to make reinforcing geotextiles to enable railway embankments to be rebuilt on the very weak, highly compressible soils created in the coastal areas of Sri Lanka by the tsunami of December 2004 (Sarsby, 2005; Indraratna *et al.*, 2005).

12.7.1 Analytical approach

The Factor of Safety FOS_U of an unreinforced embankment slope is equal to the ratio of the total resisting moment (due to the shear strength of the soil) to the total disturbing moment (due to the weight of the potentially unstable soil mass), i.e.

$$FOS_U = \frac{M_R}{M_D} \qquad [12.1]$$

If the potentially unstable soil mass is subdivided into a series of vertical slices, then

$$M_R = \Sigma TR \qquad [12.2]$$

and

$$M_D = \Sigma WR \sin \beta \qquad [12.3]$$

where

T = shearing resistance on the base of a vertical slice of material
R = radius of the circular failure surface
W = weight of the slice
β = inclination of the base of the slice

Thus

$$FOS_U = \frac{\Sigma TR}{\Sigma WR \sin \beta} \qquad [12.4]$$

If soil reinforcement is incorporated into the interface between the clay soil and the embankment then it will provide an additional resisting moment M_T which helps to maintain stability of the slope, so that the global factor of safety FOS_G of the reinforced situation is

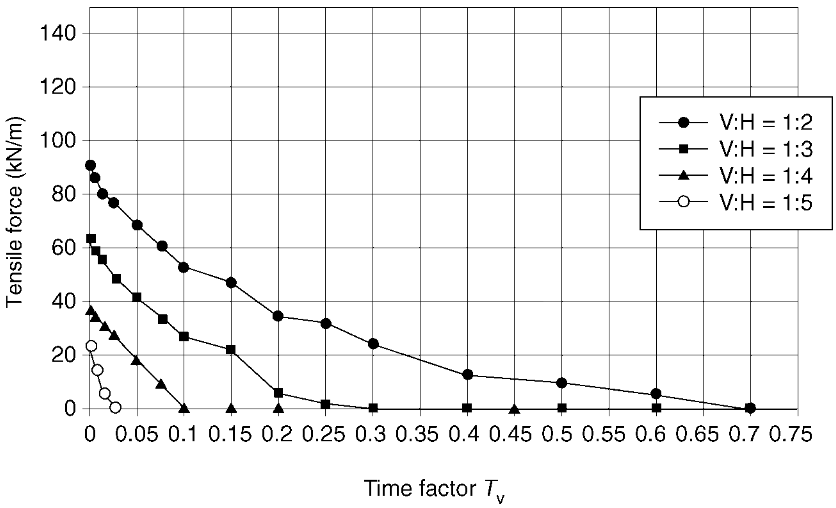
$$FOS_G = \frac{M_R + M_T}{M_D} = \frac{\Sigma TR + M_T}{\Sigma WR \sin \beta} \qquad [12.5]$$

If the horizontal force provided by the reinforcement is H and its lever arm about the centre of rotation of the potentially unstable mass is L, then

$$FOS_G = \frac{\Sigma TR + HL}{\Sigma WR \sin \beta} = \frac{\Sigma T}{\Sigma W \sin \beta} + \frac{L}{R}\,\frac{H}{\Sigma W \sin \beta} \qquad [12.6]$$

Construction of the embankment will induce excess pore pressures in the foundation soil and, as these dissipate, the shearing resistance T will increase. Consequently, if a constant global factor of safety is maintained, the force H to be provided by the reinforcement will decrease. To quantify this behaviour, a parametric study of the time-dependent reinforcement requirements of an embankment built on top of normally consolidated saturated soft clay was undertaken. The embankment was assumed to be composed of free-draining fill and a range of typical face slopes (of vertical:horizontal from 1:2 down to 1:5) was analysed. Consolidation of the foundation soil (assuming that the embankment fill was placed instantaneously) was accounted for by using one-dimensional consolidation theory and the pore pressure regime in the foundation (from the onset of loading due to the embankment until consolidation was complete) was represented by a series of isolines for each consolidation period.

A rotational stability analysis was used to backcalculate the reinforcement force H required to achieve a particular Factor of Safety at any given time after construction of an embankment. To undertake the back calculation, a target global Factor of Safety was first selected. For each specific embankment configuration, i.e. slope angle, typical values of geotechnical parameters, consolidation time, etc., a value of the basal reinforcement force H was assumed and a rotational failure analysis was undertaken (using computer software incorporating the Bishop simplified method) to determine the most critical failure circle and the resultant global Factor of Safety. If this backcalculated Factor of Safety was not equal to the defined target value, then the initially assumed basal reinforcement force was altered and the failure analysis was performed anew. This cycle of assumption and analysis was repeated until the assumed reinforcement force provided the target global Factor of Safety. The plot of this force against time then defines the 'time–strength envelope' required of reinforcement to be used in this situation. Global Factors of Safety of 1.0, 1.2 1.5 and 2.0 and four different slopes of vertical:horizontal equal to 1:2, 1:3, 1:4 and 1:5 were considered (Mwasha, 2005).



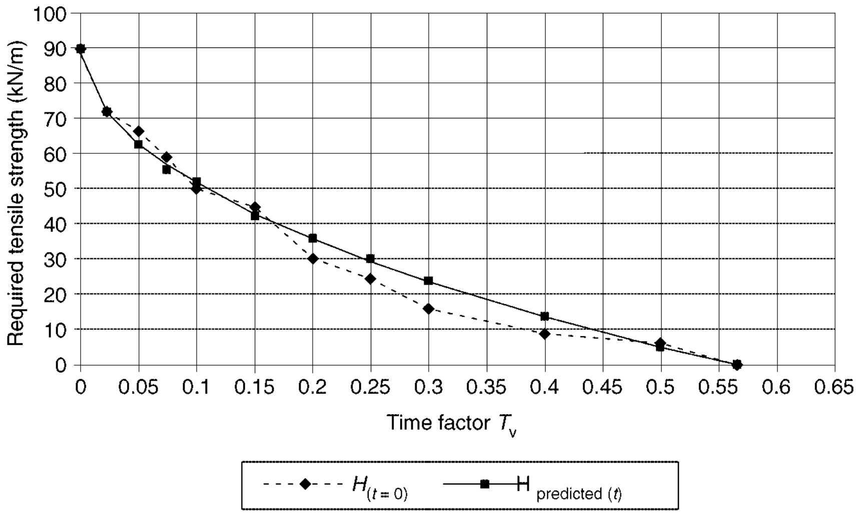
12.19 Time–strength envelopes for a global factor of safety of unity: V, vertical; H, horizontal.

12.7.2 The time–strength envelope

For each case analysed, the tensile force in the basal reinforcement shows a progressive decrease with time after embankment construction; Fig. 12.19 shows the variation in computed horizontal reinforcement force needed to achieve a global Factor of Safety of unity for slopes ranging from 1:2 to 1:5. The curves shown in this figure represent the 'time–strength envelopes' that have to be satisfied by any LLG used as basal reinforcement. For a global Factor of Safety of unity, the following points are evident:

1 The stabilizing force to be provided by the geotextile drops rapidly with time as the foundation soil consolidates.
2 For the steepest slope (1:2), the tensile force to be provided by the geotextile falls to 50% of its initial value within a time factor of 0.15 approximately (for a typical clay foundation with a C_v of 1 m^2/year this would correspond to just under 1½ years).
3 For the steepest slope, the basal reinforcement becomes totally redundant at a time factor of about 0.7 (equivalent to around 6.5 years of consolidation). For flatter slopes, the reinforcement only has to have a very short working life, i.e. around 2.5 years, 0.9 years and 0.3 years for 1:3, 1:4 and 1:5 slopes, respectively.

All the time–strength envelopes in Fig. 12.19 indicate an approximately exponential decrease in the required horizontal force with time factor (which is directly proportional to the time that has elapsed since the start of consolidation of the



12.20 Comparison between a backcalculated time–strength envelope and the empirical equation.

foundation soil). Consequently, it has been proposed (Sarsby, 2006) that for practical purposes these envelopes could be represented by an equation of the form

$$H = H_{(t=0)} - ST_v^n \qquad [12.7]$$

where

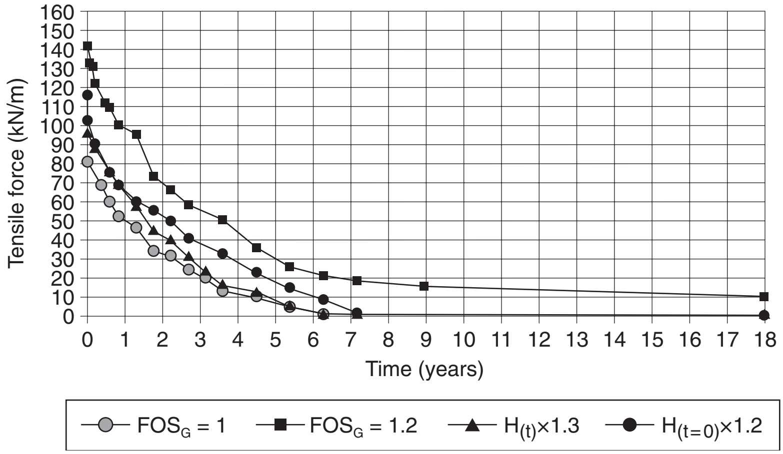
$H_{(t=0)}$ = force required before any consolidation
S, n = factors which may be affected by slope angle, strength properties of the embankment fill and the foundation soil, etc.

From the computer parametric study, it was found that accurate upper bound curves to all the individual time–strength envelopes could be represented by one empirical equation, i.e.

$$H = H_{(t=0)} - 120T_v^{0.5} \qquad [12.8]$$

The similarity between the time–strength envelope produced by back analysis and the curve represented by Equation [12.8] is shown in Fig. 12.20 (for a 1:2 embankment slope). The actual effect of slope angle and material properties was accounted for by the value of $H_{(t=0)}$ whilst S and n were essentially constant for the conditions considered in the parametric study.

Whilst the foregoing points support the concept of LLGs, consideration has to be given as to how to incorporate a suitable Factor of Safety into the definition of a design time–strength envelope. For instance, the global Factor of Safety may be defined as



12.21 Influence of the Factor of Safety on the time–strength envelope.

$$\mathrm{FOS_G} = \frac{M_R + HLF_R}{M_D} \qquad [12.9]$$

where F_R is a partial factor applied to geotextile strength.

For the steepest slope likely in practice, i.e. 1:2, the back analysis was applied to several design philosophies and combinations of global Factor of Safety and partial factor and the results are shown in Fig.12.21.

Method 1. $\mathrm{FOS_G} = 1$ with $F_R = 1$ at all time values (the initial case shown in Fig. 12.19).

Method 2. $\mathrm{FOS_G} = 1.2$ (this is a typical practical global Factor of Safety) with $F_R = 1.0$ at all time values. The form of the time–strength envelope is very similar to that determined for an $\mathrm{FOS_G}$ value of unity and, within 2 years of embankment construction, the force that the reinforcement needs to provide has fallen to 50% of the original value. However, the basal geotextile is needed to continue to provide some permanent stabilizing force even after the foundation soil has fully drained (18 years after the end of construction for a C_v value of 1 m^2/year). Although it is recognized that the magnitude of this permanent force is very small (around 10 kN/m and only about 8% of the initial tensile force needed), it is needed permanently and so does not accord completely with the concept of LLGs.

Method 3. $\mathrm{FOS_G} = 1$ with $F_R = 1.3$ at all time values. With this method of analysis, the stability of the slope is generally greater than is suggested by an $\mathrm{FOS_G}$ of unity because there would be a ‘reserve’ of strength within the reinforcement. However, this ‘reserve’ would disappear abruptly at the end of the working life of the geotextile and the slope would then be only just stable for some period of time (until further consolidation and strengthening of the foundation had occurred).

Method 4. The time–strength envelope designated as $H_{(t=0)} \times 1.2$ was generated to incorporate both a partial factor and a global Factor of Safety. The initial step was to calculate the value of H, before any consolidation occurred, for a global Factor of Safety of unity. This value of H was then increased by 20% and the actual global Factor of Safety FOS^*_G that this would represent at time zero was calculated. The full time–strength envelope was then back-calculated for this value of FOS^*_G.

It is felt that Method 4 is the best way of incorporating the Factor of Safety into the reinforcement design as it ensures stability throughout the consolidation period and yet permits the reinforcement to cease to contribute any restraining force within a reasonable time span.

12.7.3 Reinforcement selection

Pritchard *et al.* (2000) reported the outcome of work undertaken at the Bolton Institute (UK) to develop commercial forms of vegetable fibre geotextile for use as soil reinforcement. In developing these forms, a prime requirement was that they should be capable of being manufactured on conventional textile machines (with a minimum of modification of the machine). A variety of novel geotextile structures was created using flax, sisal and coir fibres (Table 12.18).

The novel materials created at Bolton were intended specifically for soil reinforcement purposes and so they were designed to provide the following:

Table 12.18 Novel vegetable fibre geotextiles (adapted from Pritchard *et al.*, 2000)

Fabric	Values at maximum load			Surface mass (g/m^2)	Thickness (mm)
	Strain %	Stress (MN/m^2)	Load/width (kN/m)		
Knitted; flax, sisal inlay (strength direction)	8.2	39	207	1753	5.3
Knitted;flax grid, sisal (strength direction)	7.4	33	144	1614	4.4
Plain weave; sisal warp, flax weft (warp direction)	9.6	50	180	1290	3.6
Plain weave; sisal warp, coir weft (warp direction)	16.3	15	113	1895	7.6
6 x 1 woven weft rib; sisal warp, coir weft (warp direction)	8.4	14	171	3052	12.1

1 The highest possible strength in one direction, since practical soil reinforcement situations such as slopes, retaining walls, etc., usually correspond closely to a plane strain condition (Sarsby, 2000).
2 Very high efficiency of transfer of shear stress within the surrounding fill into the reinforcement (where it would be balanced by the tensile strength of the reinforcement). The ability of the reinforcement to impart a confining tensile force to the soil fill is the key component in the principle of soil reinforcement. The efficiency of the 'bond' is readily indicated by the coefficient of interaction, α, i.e. the ratio of the effective friction coefficient tan δ' of a soil–reinforcement interface to the effective friction coefficient tan ϕ' of the

Table 12.19 Comparison of required and available time–strength envelopes

Slope		Tensile force (kN/m) required for the following times after embankment construction					
		0 years	1 years	2 years	3 years	4 years	5 years
1:2	Required for global Factor of Safety of unity	90	54	32	22	12	8
	Knitted; flax grid, weft sisal inlay; two layers	115	58	32	22	12[a]	9[a]
	Plain weave, sisal warp, flax weft; two layers	135	75	38	25	17[a]	14[a]
1:3	Required for global Factor of Safety of unity	65	25	5	0		
	Woven weft rib; sisal warp, coir weft; one layer	72	37	18	9		
	Knitted; flax grid, weft sisal inlay; one layer	83	43	23	12		
1:4	Required for global Factor of Safety of unity	38	0				
	Plain weave; coir warp, flax weft; two layers	48	42				
	Knotted; coir grid; two layers	44	42				
1:5	Required for global Factor of Safety of unity	21	0				
	Plain weave; coir warp weft; two layers	24	20				
	Knotted; coir grid; one layer	22	21				

[a]Extrapolated value.

surrounding soil. For soil sliding on a solid surface, the value of α is typically around 0.5–0.6. For soil sliding on an efficient conventional geosynthetic reinforcement (such as a geogrid), the α value would be unity, or even slightly above unity (Sarsby and Marshall, 1984). The value of α depends very much on the degree of interlock between the fill and the reinforcement and this is controlled primarily by the relative size of the fill particles and apertures or depressions in the reinforcement (Jewell *et al.*, 1984.)

3 Some degree of protection (against on-site installation damage) to the fibres which are intended to provide the high tensile strength of the reinforcement. The construction work on a real site involves the placement and compaction of a variety of particle sizes and shapes on top of the fibres through the use of heavy plant. This is likely to lead to crimp, shear, tearing, etc., of some of the geotextile fibres.

Figures 12.22 and Fig. 12.23 show two of the vegetable fibre geotextile forms made by Pritchard (1999).

The strengths of the novel structures (given as load per unit width in Table 12.18) are in the same range as strengths provided by 'lower strength polyester woven geotextiles used for soil reinforcement' (Rankilor, 2000).

After the reinforcement time–strength envelope has been back calculated, geotextiles can be selected that will satisfy this strength versus time profile. Table 12.19 contains data on the measured time–strength envelopes of prototype vegetable fibre geotextiles (as manufactured by Pritchard, 1999) which can satisfy the reinforcement strength requirements for embankment slopes from 1:2 to 1:5. For shallow slopes (1:4 and 1:5), the time periods for which basal reinforcement is needed are very short and many vegetable fibre geotextiles are capable of providing the required strength–time profile. For the steepest embankment considered (1:2), the basal reinforcement needs to provide some restraining force for a number of years. Nevertheless, the required time–strength envelope can be easily achieved by using more than one layer of vegetable fibre geotextile.

Values of the coefficient of interaction for the novel structures created at Bolton were determined in accordance with the provisions of Part 8 of BS 6906 (British Standards Institution, 1991) using a 300 mm by 300 mm direct shear box with the geotextiles confined within sand or gravel. The applied normal stress range was up to 200 kN/m^2 (which corresponds to approximately 11 m of fill on top of the reinforcement). The values of measured friction angle and coefficient of interaction α given in Table 12.20 demonstrate clearly that the surface of the vegetable fibre geotextiles was very efficient at transmitting stresses between the fill and the reinforcement. Ali (1992) had previously obtained α values close to unity for coir and jute ropes used as reinforcement of tropical soil.

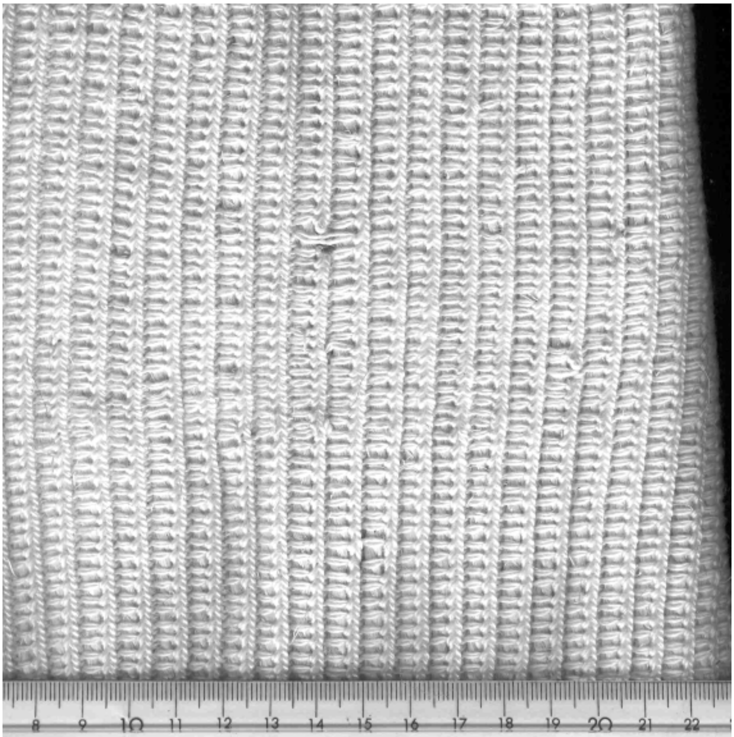
The foregoing data confirm both the concept of LLGs and the proposition that suitable geotextiles can be made using readily available vegetable fibres, in this case, sisal, coir and flax.

12.22 Novel geotextile with the form of a collection of linear reinforcements.

Table 12.20 Shearing interaction between soil and novel vegetable fibre geotextiles (adapted from Pritchard *et al.*, 2000)

Fabric	Maximum friction angle[a] with sand (deg)	α for sand fill	Maximum friction angle[b] with gravel (deg)	α for gravel fill
Knitted; flax, sisal inlay	40.9	1.01	50.5	0.86
Knitted; flax grid, sisal	38.8	0.94	50.9	0.87
Plain weave; sisal warp, flax weft	40.0	0.98	49.8	0.84
Plain weave; sisal warp, coir weft	42.1	1.06	53.4	0.95
6x1 woven weft rib; sisal warp, coir weft	42.0	1.05	50.9	0.87

[a]Maximum friction for sand fill, 40.5°.
[b]Maximum friction angle for gravel fill, 54.7°.
*Max friction for sand fill = 40.5°, max friction angle for gravel fill = 54.7°



12.23 Novel geotextile with the form of a 'corrugated sheet'.

12.8 Conclusions

The use of natural fibres in composite construction can be traced back many centuries. Since the creation of man-made polymeric fibres, perceptions that natural fibres have low apparent tensile strength and a very short working life (particularly when in contact with soil and water) have led to the virtual demise of their use in construction. However, there have been sporadic attempts at their use as technical materials within construction.

The key to developing geosynthetics from natural fibres is the concept of designing by function, i.e. identifying the functions and characteristics required to overcome a given problem and then manufacturing the product accordingly. When correctly designed, natural fibre materials can compete with synthetic materials and sometimes they will even have superior performance.

There are numerous advantages to be gained from the development of vegetable fibre geotextiles using indigenous materials.

1 The raw materials employed, i.e. vegetable fibres, are 'environmentally friendly'.

2 They have low unit cost at the source of production.
3 Natural fibres have significant initial strength and are resistant to harsh environments
4 They introduce additional uses of by-products or new uses for waste products.
5 They represent a renewable resource.

It has been shown that the basal reinforcement requirements for an embankment built on soft clay can be satisfied by vegetable fibre geotextiles. These geotextiles can be made from fibres (such as coir, sisal and flax) which are readily available in developing countries. These new materials represent an extremely cost-effective and socially beneficial alternative to geotextiles made from synthetic compounds because they utilize indigenous renewable resources and provide widespread local employment.

There is a need to develop an assurance quality ethos in the use of bio-based products in world markets especially where they are replacing traditional products made from other resources. This requires the need to develop codes and specifications of each desired bio-based composite product. Such codes will assure the user of the composites that the product will perform in a certain way and they will develop consumer confidence.

12.9 References

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Index