EAST MIDLANDS GEOTECHNICAL GROUP
THE INSTITUTION OF CIVIL ENGINEERS

# Geotechnical Engineering of Landfills

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Department of Civil and Structural Engineering
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# Preface

The symposium on Geotechnical Engineering of Landfills and this associated publication aim to provide an opportunity for the presentation and discussion of recent developments in the design, construction and operation of landfill facilities. The specific objectives are to highlight:

- the important role played by the mechanical properties of waste in optimising barrier design and landfill operation;
- issues related to the design and testing of mineral liners, including bentonite enriched soils and colliery spoil; and
- recent developments in the assessment of geosynthetics, including barrier stability, assessment of protection materials for liners and properties of geosynthetic clay liners.

Although there have been a number of conferences and meetings both in the UK and throughout the world covering issues of landfill design, material performance and landfill operation, it was felt by the organising committee that many of these are aimed at specific sub groups of practitioners and researchers. Therefore it was considered timely to hold a symposium which covered a range of issues relevant to geotechnical engineers and associated disciplines, to highlight new areas of research and practice, and to provide a forum for discussion.

It was for these reasons that the East Midlands Geotechnical Group (EMGG) decided to organise a symposium on the subject in 1998, following the successful seminars on Groundwater Pollution in 1994 and Lime Stabilisation in 1996. The EMGG was formed in 1992 with the aim of providing the environment of a learned society on geotechnical subjects, for the benefit of civil engineers and engineering geologists living along the hinterland of the M1 from Northamptonshire to South Yorkshire. Most of the activity concerns the evening meetings programme, although site visits and symposia are organised to complement this main role. The Nottingham Trent University was considered an appropriate host for the symposium since it has been active in research in the area of landfill engineering for several years.

The editors wish to acknowledge the considerable support of both the organising committee and the full EMGG committee. They also wish to thank the contributors for their excellent papers, particularly since the time to write such a paper appears to be progressively harder to find. Meeting the deadline for camera-ready copy so that the book could be made available at the symposium requires a considerable concentration of effort, and this is much appreciated. Finally the support provided by Samantha Casterton from the Commercial Administration Centre of The Nottingham Trent University, is gratefully acknowledged.

# Contents

# Mechanical Properties of Landfill Waste

While significant attention is given to the design, construction and long-term performance of landfill side slope lining and capping systems, relatively little work has been directed towards gaining an understanding of the mechanical properties of the waste and assessing its interaction with the surrounding barrier. A knowledge of the mechanical properties of waste is required to enable cost effective, environmentally safe landfill facilities to be constructed and operated. A number of issues related to landfill design require consideration of the waste body behaviour.

- Shear strength of the waste mass are required for use in slope stability studies.
- Compressibility of waste has to be quantified to enable predictions of total and differential settlements, and of the distribution of these with time during the life of the landfill facility (e.g. this information is needed to optimise capping design).
- An understanding of the permeability of waste, and of the influence of overburden pressure and degradation, is required to improve operation of leachate collection wells and the development of leachate re-circulation systems (e.g. for potential use in flushing bio-reactors).
- The in situ stress within, and stiffness of, emplaced waste are needed to enable assessment of the support provided to vertical and near vertical side slope lining systems by the waste (i.e. leading to an understanding of the in service deformations of the lining system).

The papers in this section address aspects of household waste compressibility, permeability and stiffness.

The paper by Powrie *et al.* covers issues related to the compression of landfilled waste which occurs both during placement (due to machine compaction and overburden effects) and in the long-term (as a result of processes such as degradation and ravelling). The changes in waste density that result from compression during landfilling are considered including the affect on the mass of waste that a site can accept, and on the permeability of the waste. The authors emphasise that knowledge of the permeability of the waste is important because of its influence on leachate production and management. In addition, they stress that long-term settlement must be taken into account in the design of the landfill site closure systems, and comment that provision

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should be made for the continuing care and maintenance of restoration works. The paper summarises the various mechanisms of waste compression and discusses the implications for landfill operation, closure and aftercare.

Dixon & Jones highlight the important role that waste plays in the stability and structural integrity of vertical or near vertical lining systems. It is contended that although the landfill industry is developing and constructing novel barrier systems for steep side slopes, there is still a dearth of information, and hence limited understanding, of the factors which control both short-term and long-term deformations of the lining; these are influenced by construction techniques and waste degradation processes respectively. The paper describes a novel method for obtaining the relevant mechanical properties of waste for use in assessment of designs, based an the pressuremeter, and presents the results of initial field trials. Results from in situ tests, to measure stiffness and stresses in both fresh and partially degraded household waste, are discussed.

The work presented in this section provides a timely and significant contribution to the study of waste mechanics. While the study of waste behaviour is fraught with difficulties (i.e. studying a heterogeneous material whose engineering properties change with time), the papers demonstrate that useful results can be obtained. This area of research requires continued attention from the landfill community.

# Compression of waste and implications for practice

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## Introduction

Compression of landfilled waste occurs both during placement (due to machine compaction and overburden effects), and in the long-term (as a result of processes such as degradation and ravelling). The changes in waste density resulting from compression during landfilling will affect the as-received volume of waste that a site can accept, and also the permeability of the waste. The permeability of the waste is important because of its influence on leachate levels, production and management.

Long-term settlements must be taken into account in the design of landfill site closure systems, and may require provision to be made for the continuing care and maintenance of restoration works. In this paper, the various mechanisms of waste compression and the factors influencing them are summarized. The implications for landfill operation, closure, and aftercare are then discussed.

## Mechanisms of compression

The following mechanisms of compression of waste were identified by Edil *et al.* (1990).

- **Mechanical compression**, due to the crushing, distortion, reorientation, bending and/or breaking of waste particles as vertical stresses are increased, either during compaction or due to the self weight of the fill as further material is deposited on top. In the absence of pre-compaction, the degree of mechanical compression depends (other factors being equal) on the depth of the waste.

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- **Degradation**, due to biological decomposition and physico-chemical processes such as corrosion and oxidation of the waste in the longer term.
- **Ravelling**, which is the gradual migration of finer particles into the larger voids and which can occur during both mechanical compression and degradation.

Compression may also occur on wetting of the waste, due to the loss of strength or structure of certain components on contact with moisture, e.g. paper and cardboard. Waste settlement is conventionally classified as either primary or secondary, depending on the timescale over which it occurs.

## Primary compression

Primary compression of the waste in the landfill will probably occur within a period of days following the deposition of further material on top (Bleiker *et al.*, 1995; Beaven & Powrie, 1995).

Beaven and Powrie (1995) carried out a series of tests on a number of samples of domestic refuse to investigate the variation of density and hydraulic conductivity with vertical stress. The tests were carried out in a large purpose-built compression cell, located at Cleanaway Ltd's Pitsea landfill site in Essex, England. The cell consists of a steel cylinder, 2m in diameter and 3m high, into which refuse is placed for testing (Figure 1).

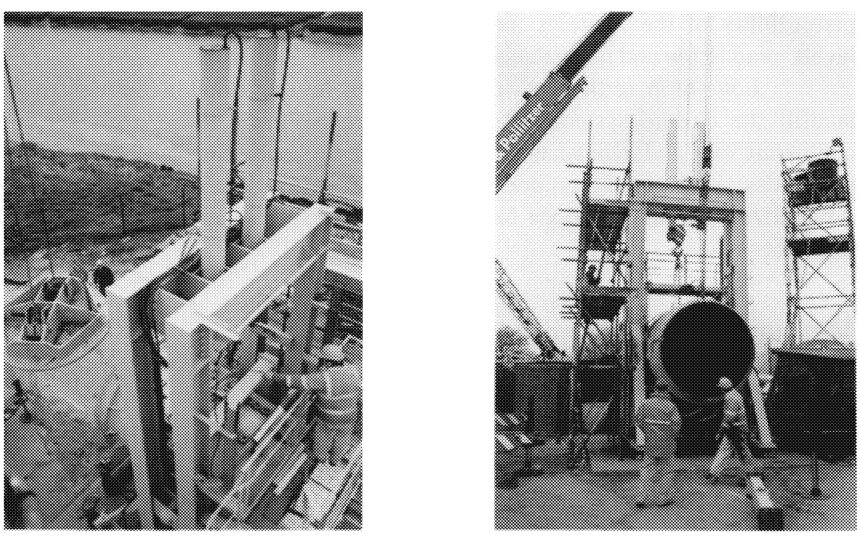

**Figure 1** The Pitsea compression cell during a recent overhaul and refurbishment

The cylinder is suspended vertically within a steel support frame. The feet of the support frame are mounted on load cells, enabling the weight of the contents of the cell to be monitored continuously. The base of the cylinder is sealed by a 2m diameter platen which is seated on an 'O' ring. Refuse in the cylinder is compressed by an upper platen, just under 2m in diameter, which can be moved vertically up and down inside the testing cylinder. The upper platen is connected to, and moved by, two 200mm diameter hydraulically operated pistons. At the start of a test the upper platen is lowered onto the refuse: a constant vertical load can then be applied by means of the hydraulic pistons. The maximum vertical load is 1900kN, giving a stress of 600kPa distributed uniformly over the plan area of the refuse sample.

Water can be allowed to flow upward through the sample, from two 450 litre water header tanks mounted on a scaffold tower up to 3 metres above the top of the testing cylinder. The tanks are connected to 12 evenly spaced 25mm diameter ports on the lower platen. Water flows out of the upper platen through similar ports, and through a 2mm annular clearance gap between the outer edge of the platen and the inner surface of the testing cylinder. In-line electromagnetic flow meters are used to record the total volumes and flow rates of water entering and leaving the sample.

At the start of each test, refuse was loaded into the cell and compacted in layers to the desired initial density, until the overall refuse depth was approximately 2.5m. Following placement of the refuse, 18 piezometers were installed horizontally through ports in the side of the cylinder, located at vertical spacings of between 150mm and 400mm. Lengths of string anchored at known points within the refuse and running out of the cell through the piezometer ports were used to measure the total compression at various depths. The upper platen was then lowered onto the sample and an initial load was applied using the hydraulic system. The compression of the refuse was monitored as a function of time by measuring the downward movement of the upper platen. Any leachate squeezed out of the refuse was collected and its volume recorded.

When compression had substantially ceased (in practice, when the rate of settlement had fallen to less than 1% of the sample height per day), the waste was saturated by introducing water through the lower platen and then allowed to drain to reach its field capacity (i.e. the moisture content of the refuse under conditions of free downward drainage conditions). The saturated hydraulic conductivity of the refuse was determined (at constant applied vertical stress) by carrying out a constant head flow test. Water from the header tanks was allowed to flow upward through the refuse, overflowing at the top of the sample. The hydraulic gradient was measured by means of the piezometer ports in the side of the column and the flow rate using the electromagnetic flowmeters. At high vertical stresses and low refuse permeabilities the flow rate of water into the sample was low, and direct measurement of the (small) fall in water level in the header tanks with time was found to be more reliable than the flowmeter reading.

The refuse was then drained, the applied stress increased and the cycle of operation and measurement repeated (i.e. the moisture content of the refuse under conditions of free downward drainage conditions). Tests were carried out on a number of samples of crude and pulverized domestic wastes. In this paper, the results from one test on crude waste (DM3) and one test on pulverized waste (PV1) are discussed.

DM3 was a crude domestic refuse obtained from the tipping face of a landfill. The water content at field capacity at the start of compression was ~102% (by dry mass). PV1 was a processed refuse, obtained by pulverizing crude domestic refuse, passing it through a 150mm filter and removing dense fines (including some putrescibles). The water content at field capacity at the start of compression was ~141% (by dry mass). The composition of these wastes is summarized in Table 1.

| | | | DM 3 | PV 1 |
|---|---|---|---|---|
| Refuse component | Dry density of component, $Mg/m^3$ | Saturated density of component, $Mg/m^3$ | % of total mass | % of total mass |
| Paper/card | 0.4[1] | 1.2[1] | 39.8 | 49 |
| Plastic film | 1.0[1] | 1.0[1] | 4.4 | 8.3 |
| Dense plastics | 1.1[1] | 1.1[1] | 6.4 | 7.8 |
| Textiles | 0.3[1] | 0.6[1] | 5.5 | 5.7 |
| Misc. combustibles | 1.0[2] | | | |
| Misc. non-combustibles | 1.8[1] | 2.0[1] | 2.4 | 1.1 |
| Glass | 2.9[1] | 2.9[1] | 7.0 | 1.3 |
| Putrescibles | 1.0[1] | 1.2[2] | 13.2 | 6.5 |
| Ferrous metals | 6.0[1] | 6.0[1] | 3.2 | 9.0 |
| Non ferrous | 6.0[2] | 6.0[2] | 1.2 | 1.6 |
| <10mm fines | 1.8[2] | 2.0[2] | 4.9 | 5.2 |
| TOTAL | | | 100 | 100 |

Note 1: From Landva & Clark (1990)
Note 2: Assumed value

**Table 1** Material classification of wastes tested

The dry density $\rho_{dry}$ is defined as the density that the waste would have if it were completely dry (i.e. had zero liquid content). The initial dry density of the refuse as loaded into the cell (prior to compression) was determined from the measured density ρ and the initial water content w obtained from a sub-sample of the refuse, using the standard soil mechanics relationship $\rho_{dry} = \rho/(1+w)$. The initial dry density of the processed refuse PV1 was 0.25Mg/m$^3$, while for the crude waste (DM3) the initial dry density was 0.33Mg/m$^3$. The corresponding initial wet densities were 0.62 Mg/m$^3$ (PV1) and 0.50 Mg/m$^3$ (DM3).

Figure 2 shows the variation in dry density and wet density at field capacity with vertical effective stress. (It has been assumed that the applied vertical stresses at equilibrium stages of the tests under discussion are effective stresses, transmitted through the refuse matrix by interparticle contact. This is because although the refuse is not dry, it is free to drain vertically downward and the sample is of substantially uniform hydraulic conductivity: thus the fluid in the voids is at zero gauge pressure). Both axes are plotted to logarithmic scales, and it has been assumed that the density and stress are uniform throughout the sample, i.e. the effects of self weight and sidewall friction have been ignored.

The final dry density of DM3 at an applied stress of 600kPa was 0.71 Mg/m$^3$, while the final dry density of the processed refuse PV1 was 0.60 Mg/m$^3$. The corresponding final wet densities at field capacity were 1.18 Mg/m$^3$ (DM3) and 0.97 Mg/m$^3$ (PV1). These may be compared with wet densities of 0.62 – 0.67 Mg/m$^3$ and 0.81 – 1.11 Mg/m$^3$ reported by Caterpillar (1995) following compaction of crude domestic wastes in layers using Caterpillar 816B and 826 compaction plant. Assuming a water content in the range of 30 - 40%, these ranges of wet density correspond to dry density ranges of 0.37 – 0.47 Mg/m$^3$ and 0.49 – 0.78 Mg/m$^3$ respectively. The implication is that, in terms of the waste density achieved, compaction at the tipping face can have a similar effect to the burial of the waste by several tens of metres of overburden.

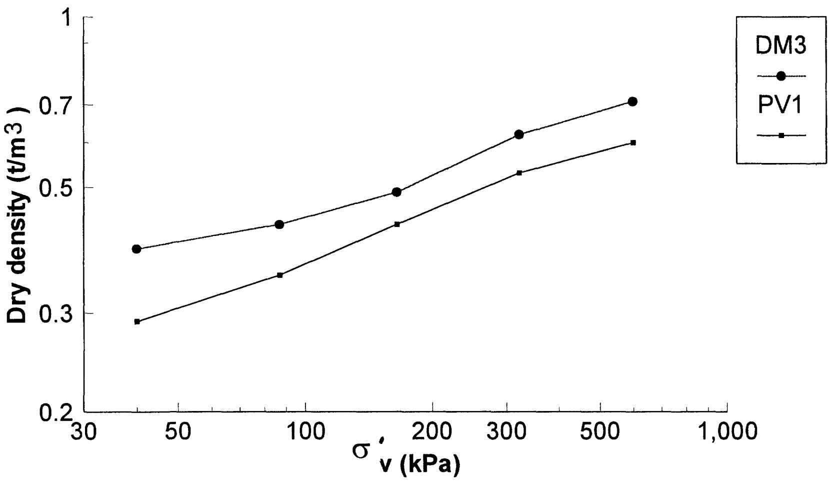


**Figure 2** Variation in refuse density with vertical effective stress

The hydraulic conductivities of PV1 and DM3 at different applied stresses are shown in Table 2. Between low and high stress states the hydraulic conductivity reduced by over 2 orders of magnitude (from $3.5\times10^{-5}$ to $10^{-7}$m/s) for DM3 and by nearly 5 orders of magnitude (from $10^{-4}$ to $3.5\times10^{-9}$m/s) for the processed refuse PV1.

| Applied stress, kPa | Saturated hydraulic conductivity, m/s | |
|---|---|---|
| | PV1 | DM3 |
| Initial | $2\times10^{-4}$ | ND |
| 40 | $3.6\times10^{-5}$ | $3.5\times10^{-5}$ |
| 87 | $7\times10^{-6}$ | $2\times10^{-5}$ |
| 165 | $2\times10^{-6}$ | $3\times10^{-6}$ |
| 322 | $9\times10^{-8}$ | $8\times10^{-7}$ |
| 600 | $3.5\times10^{-9}$ | $1\times10^{-7}$ |

ND = not determined

**Table 2** Hydraulic conductivity at different applied stresses

Hydraulic conductivity is plotted as a function of dry density for both wastes (to a double logarithmic scale) in Figure 3.

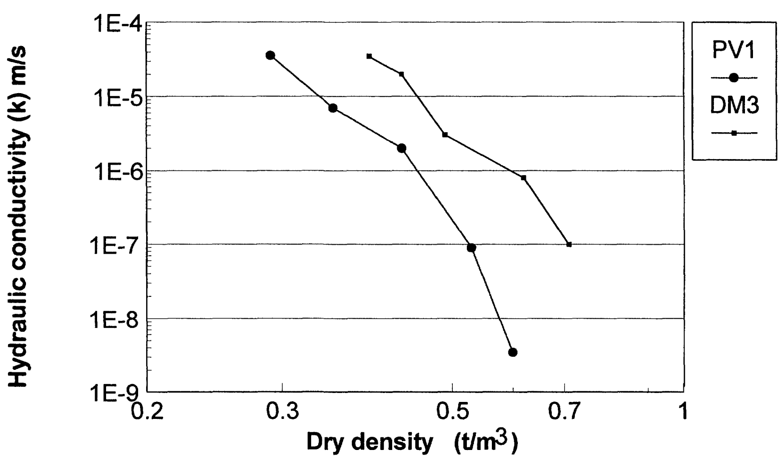


**Figure 3** Hydraulic conductivity as a function of waste density

## Secondary compression

Quantitative investigation into secondary compression of waste due to degradation is rare. Although case records are given in papers such as those by Gasparini *et al.* (1995), Hilde & Reginster (1995) and Kostantinos *et al.* (1997), there are few data concerning the likely ultimate settlements of municipal solid and industrial wastes taking the effects of degradation into account.

The UK guidance manual on landfill design and operational practice, Waste Management Paper 26B (1995), suggests a secondary settlement figure of 15-20% of the initial refuse depth on the basis of research at a limited number of UK household waste sites (Coulston & Wye College, 1993). Other research has suggested that settlements in excess of 25% of the depth of the landfill may occur (Bjarngard & Edgers, 1990; Di Stefano, 1993).

The first draft of the DoE Waste Management Paper (WMP) 26D (landfill monitoring) suggested that long-term settlements of between 25 and 50% of the original fill thickness are typical of the allowance that may need to be made in determining the final fill levels of a domestic waste landfill. The latest draft for public consultation of WMP 26D (January 1996) suggests long-term settlements in excess of 20%. The paper also notes that accelerated stabilisation will cause more of the waste to degrade during the filling phase, reducing the magnitude of settlement following closure.

The considerable uncertainty regarding the secondary compression of waste is understandable in view of the various factors that are known to influence its eventual magnitude. These include:

- the **composition** of the waste: wastes with a higher proportion of degradable (principally organic) materials will be more susceptible to long-term settlement due to decomposition than wastes containing predominantly inert materials;
- the as-placed **dry density** of the waste: for a given degradable fraction, a given mass of compacted (i.e. denser) waste would be expected to have less potential for mechanical volume loss and hence long-term settlement - typical dry densities of wastes can be found in Beaven & Powrie (1995);
- the **depth** of the fill material: for a given waste type and density, the potential loss of volume or mass on degradation is roughly proportional to the original volume or mass of waste, so that in absolute terms deeper landfills would be expected to exhibit greater settlements.

The rate of waste degradation in the long-term depends primarily on the composition of the waste, its water content and the rate at which water passes through it. Forced gas extraction may also influence the rate or pattern of degradation, e.g. larger settlements tend to occur around gas extraction wells.

Increasing the water content of the fill increases its rate of decomposition (Chen & Chynoweth, 1995), while increasing the rate of water movement through waste without changing the water content has been found to increase methane generation rates by approximately 25-50% (Klink & Ham, 1982). Although current UK policy is directed towards the stabilization of landfills within a 30-50 year time period (Gronow, 1996), this cannot be achieved with "dry" containment techniques, for which stabilization times of hundreds or even thousands of years are predicted (Knox, 1996).

Measures currently being taken to reduce stabilisation times at landfill sites in the UK principally involve the recirculation of leachate. Treated and/or untreated leachate collected at the bottom of a containment cell is pumped up to the top of the landfill and is allowed to percolate down through the waste mass. This has been shown in laboratory scale experiments (Barlaz *et al.*, 1989; Reynolds & Blakey, 1995), landfill lysimeters (Buivid *et al.*, 1981; Kinman *et al.*, 1987) and controlled landfill cells (Halvadakis *et al.*, 1988; Townsend *et al.*, 1996) to enhance microbial activity within the waste and is suggested in WMP 26B (1995) as a method of achieving accelerated stabilization of a landfill. As the leachate is itself a product of the waste degradation process, it is likely to contain the micro-organisms and nutrients necessary for the degradation of the putrescible fraction of the waste mass. However, prolonged recirculation of untreated leachate could lead to high concentrations of soluble inorganic ions due to leaching and decomposition processes within the waste mass. These ions, principally sodium and chloride, could well inhibit further degradation unless the leachate is treated prior to recirculation (Knox, 1996).

## Implications for practice

### Operation

The main concern during the operational phase of a landfill is likely to be primary compression, but secondary compression may begin to be an issue especially if accelerated degradation is promoted. Primary or short-term compression will affect

- the mass or as-received volume of refuse that can be accepted;
- the permeability of the refuse, and hence the pore leachate pressures within the site and the ease with which leachate can be recirculated; and
- the integrity of structures within and adjacent to the waste mass, including gas and leachate extraction wells and steep sided lining systems.

#### *Mass input to the site*

Waste already emplaced within a landfill will undergo compression as further refuse is deposited on top. This means that the volume of waste placed will in general be greater than the final volume of the landfill, even allowing for secondary compression due to degradation etc.

The as-placed volume or dry mass of refuse corresponding to a given final fill depth can be calculated using a relationship between dry density $\rho_{dry}$ and depth z, assuming (in the absence of chemical and biological activity) that the mass of dry solids remains constant. On this basis, and taking the pore leachate pressure within the landfill as zero, Beaven & Powrie (1995) show that the volume of refuse placed at a dry density of 0.35 $Mg/m^3$ required to give a final depth of 10 m is 11.5 $m^3$ per square metre, i.e. there is an average primary compression of 15% by volume. For a final depth of 30 m, a total of 42 $m^3$ per $m^2$ of refuse is required, corresponding to an overall average primary compression of 40%.

These values of average primary compression would be reduced if the waste were compacted at the tipping face to achieve a higher as-placed dry density than 0.35 $Mg/m^3$. Using the data from DM3, the average dry density due to self-weight compression in a 10 m deep landfill would be approximately 0.4 $Mg/m^3$, and in a 30 m deep landfill 0.5 $Mg/m^3$. Thus in theory, self weight compression gives no increase in density if the waste is placed in a 10 m deep landfill at a dry density in excess of 0.4 $Mg/m^3$, or in a 30 m deep landfill at a dry density of more than 0.5 $Mg/m^3$.

In reality, compression due to chemical and microbiological activity may also occur during the operational stage: this would increase the volumetric compression during filling. However, the assumption that the pore leachate pressure is zero throughout the depth of the landfill is unrealistic as discussed in the following section: non-zero pore leachate pressures would reduce effective stresses and hence the degree of self weight compression.

### *Refuse permeability and leachate recirculation*

It has already been stated that the rate of waste degradation can be accelerated by increasing either the water content of the waste or the rate of water movement through it. Conceptually, the simplest way of promoting water movement through a landfilled waste is to introduce fresh liquid at the surface, and allow it to percolate downward under gravity through the waste mass to a basal drainage blanket. The leachate pressure within the drainage blanket is maintained at or near zero by pumping. Flushing the landfill in this way would also remove mobile non-biodegradable contaminants from the waste, which could be viewed as an essential part of the process of bringing the site to a condition whereby it is unlikely to cause pollution of the environment or harm to human health - a prerequisite for the surrender of a site licence.

It is estimated that to remove the contaminant load from a municipal solid waste landfill, each $m^3$ of waste must be flushed through with 5$m^3$ of liquid (Walker *et al.,* 1997). If this is to be achieved over a period of 50 years, the implied infiltration rate is 0.1D metres per annum, where D is the depth of the landfill (in m).

In a landfill in which the permeability of the waste is uniform with depth, a uniform downward hydraulic gradient of unity would be established and the steady-state infiltration rate (in $m^3$/s of liquid per $m^2$ of surface area of the landfill) would be equal to the Darcy permeability of the waste. A flushing rate of 0.1D m/annum would then correspond to a (uniform) hydraulic conductivity of $10^{-7}$ m/s for a 30 m deep landfill and $5 \times 10^{-7}$ m/s for a 60 m deep landfill. According to the data shown in Figure 3, acceptable flushing rates could be achieved for crude domestic wastes under a hydraulic gradient of unity for dry waste densities not exceeding about 0.6 – 0.7 Mg/m$^3$. Excessive compaction of the waste at the tipping face would lead to waste densities in excess of this range, and waste permeabilities below the required minimum.

At effective stresses in excess of 300 kPa, the permeability of the pulverized waste PV1 is significantly less than that of the crude waste DM3. Although pulverization may be desirable, in that it promotes a more even flow of liquid through the waste, the reduced permeability will limit the depth of landfill that can be flushed within the 50 year timespan.

The tests carried out in the Pitsea compression cell show that in reality the permeability of the refuse is likely to decrease with depth. In these circumstances, the downward hydraulic gradient is not uniform with depth, but increases as the permeability decreases to give a volumetric flow rate that is constant with depth.

This type of behaviour is well documented for soil deposits in which the permeability changes with depth (Vaughan, 1994; Bromhead, 1994). For refuse, the situation is more complicated than for most soils, because in addition to the permeability, the unit weight also varies significantly with vertical effective stress. The vertical effective stress depends in turn on the vertical total stress (which depends on the unit weight) and the pore water pressure (which depends on the head and hence on the permeability). This issue is discussed in more detail by Powrie & Beaven (in preparation), but the main implication as far as the current discussion is concerned is that the minimum permeability limit calculated on the basis of a hydraulic gradient of unity is likely to be a lower bound.

### *Integrity of gas/leachate wells and steep linings*

The likely compression of the waste during and after placement must be taken into account in the design of structures typically located within the landfill such as gas and leachate wells. There is at least anecdotal evidence of gas and leachate extraction wells buckling or fracturing due to the downdrag effect of the waste as settlement occurs. If this is to be avoided, the wells must be designed so as to withstand the shear stresses developed at the interface with the waste. (These shear stresses could be estimated on the basis of the estimated lateral earth pressure coefficient and the measured frictional strength of the waste: Jones & Dixon, 1997; Gotteland *et al.,* 1995; Jessberger *et al.,* 1995). Alternatively, wells can be designed to "float" or telescope in response to the settlement of the waste. These issues are discussed by Jessberger *et al.* (1995).

Downward shear stresses resulting from differential compression effects must also be considered in the design of a landfill lining system. An evaluation of the stresses induced by waste settlement in a geomembrane sideslope liner is presented by Kanou *et al.,* (1997), and the issue of lateral support provided by waste to the lining system is discussed in detail by Dixon & Jones (1998) in their paper in this volume.

## Closure and aftercare

Primary settlement would be expected to be complete before site closure, and should not therefore affect the post-closure behaviour and aftercare requirements unless there is a significant change in the post-closure groundwater (leachate) regime: this could occur in a containment site when the containment system fails.

Secondary compression due to the decay and decomposition of the waste will impinge significantly on the post-closure behaviour of the landfill and the aftercare requirements, in terms of both the eventual magnitude of the settlement and the time scale over which it develops. The effects of secondary compression will influence:

- the specified final fill level; and
- performance of and aftercare requirements for the cap.

Secondary compression of the waste will also affect the permeability and hence the flow of leachate through the waste, and the performance and integrity of steep liners and leachate/gas extraction wells, along the lines discussed above.

### *Final fill level*

The final fill level is important in the context of the site closure plan, primarily because of its influence on the ultimate (post-settlement) surface profile and the performance of any restorative cap or cover layer. In general, the final fill level at any point must be significantly higher than the desired ultimate surface level, to allow for the effects of secondary compression.

The specification of the final fill level needed to achieve a desired ultimate level is complicated by the difficulty of predicting the secondary settlement of the waste. Obviously, any underestimation of the magnitude of the post-closure settlement will result in the underfilling of the landfill and the finished contoured profile being generally too low. Troughs or hollows, which could significantly increase the likelihood of ponding and/or cap failure, may also develop: problems associated with cap integrity during continuing long-term settlement are well-documented (Morris & Woods, 1990). Overfilling the landfill site in one operation in anticipation of large long-term settlements may not be practicable, and if the expected settlement does not occur the restored site profile will be too high.

The main problem is that, although some apparently successful attempts have been made (e.g. van Meerten *et al.,* 1997), the rate and eventual magnitude of secondary compression are at present very difficult to predict. Field data on which an empirical approach might be based are scarce, not least because settlements have to be monitored over a period of years if not decades for meaningful trends to emerge. Settlement data must also be correlated with the waste type and placement history to enable the most important controlling variables to be identified.

In the absence of any proven theoretical or empirical method of estimating rates and magnitudes of secondary compression, it must be accepted that the attainment of a desired finished profile will involve a considerable amount of uncertainty. Realistically, operators and regulators must accept that there will probably be a need for a long-term management plan, perhaps involving stripping back the cap from time to time and placing further material below it.

### *Cap performance*

The restorative cap geometry should be designed so as to minimize the likelihood of cap failure by cracking or rupture as secondary settlement occurs. This is particularly important in the case of containment sites where an intact cap forms part of the leachate management strategy, as failure of the cap will tend to lead to increased infiltration and a larger volume of leachate being generated.

Magnitudes of secondary settlement are difficult to estimate with confidence, but the pattern of waste settlement can have a significant effect on whether or not a resistive capping layer ruptures. Damage is more likely to be caused by differential settlements than uniform settlements. In addition, the gauge length over which the differential settlement occurs will also be important in determining the severity of its effect on the capping layer.

Recent experimental studies carried out using a geotechnical centrifuge by Richards and Powrie (in preparation) have shown that the long-term integrity of a low permeability capping system can depend on the pattern of subsidence in the waste below it. The imposition of a displacement discontinuity (i.e. a step subsidence pattern) below a low permeability cap is much more likely to result in the rupture of the cap than the imposition of a discontinuity in slope (i.e. a ramp subsidence pattern). Also, caps with convex upward sloping edges were generally found to out-perform caps with flat edges, in terms of the degree of differential movement across the discontinuity required to cause through-rupture (Figure 4). Where possible, the depth of the landfill should be specified so that sudden changes in the depth of the waste fill (e.g. at the edge of a steep-sided pit), which could result in the imposition of a displacement discontinuity (step) below the cap as the waste degrades, are avoided. The cap geometry should be specified so that settlement of the underlying waste will tend to cause compression, rather than extension, in any resistive or low permeability layer incorporated into the cap.

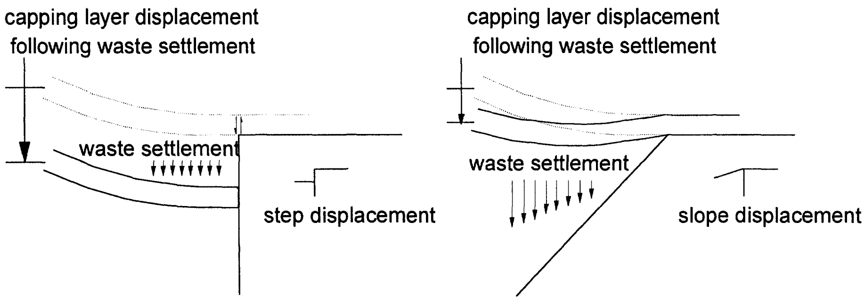


**Figure 4** Step and slope patterns of waste settlement (displacement and slope discontinuities)

The potential vulnerability of the cap edge is confirmed by data presented by Morris & Woods (1990), who suggest that severe differential settlements may occur in this zone owing to the difficulty in achieving the same degree of compaction in the waste near to the edge of the site as in the remainder of the landfill.

## Conclusions

Typical domestic or municipal solid waste is a highly compressible material. Compression can occur both in the short-term during placement of the waste, and in the long-term due to microbial and/or chemical decomposition and ravelling. Although the mechanisms of waste settlement are well understood, magnitudes and rates of compression, particularly after placement, are difficult to predict quantitatively.

Compression of waste will affect both the density and the permeability of the waste. Changes in density during placement will tend to increase the apparent capacity of the landfill in terms of mass or as-received volume. Compression during and after placement will tend to reduce the permeability of the waste, making leachate recirculation more difficult. The extent to which *in situ* compression affects the waste density and permeability may depend to some extent on the degree of compaction during placement.

Structures such as gas and leachate wells and landfill liners must be designed to resist or accommodate the shear stresses imposed by settling waste. Final fill levels and the geometry of a restorative cap must take account of the likely long-term settlement of the waste as it degrades. As this is likely to be large but difficult to predict, there may well be a need for a long-term management plan that incorporates periodic maintenance of, and placement of further material below, the cap.

## References


Barlaz, M.A., Ham, R.K. & Schaefer, D.M. (1989). Mass balance of anaerobically decomposed refuse in laboratory scale lysimeters. *J. Environmental Engineering*, ASCE, 115(6), pp 1088-1102.

Beaven, R.P. & Powrie, W. (1995). Determination of hydrogeological and geotechnical properties of refuse using a large compression cell. *Proceedings of the 5th International conference on landfills, Sardinia 95* (ed. T H Christensen, R Cossu and R Stegmann) 2, pp 745-760. Cagliari: CISA Environmental Sanitary Engineering Centre.

Bjarngard, A. & Edgers, L. (1990). Settlement of municipal solid waste landfills. *Proceedings of the 13th Annual Madison Waste Conference*, pp 192-205.

Bleiker, D.E., Farquhar, G. & McBean, E. (1995). Landfill settlement and the impact on site capacity & refuse hydraulic conductivity. *Waste Management & Research* 13, pp 533-554.

Bromhead, E.N. (1994). Interpretation of pore water pressure profiles in underdrained strata. In *Groundwater problems in urban areas* (ed W B Wilkinson), pp 149 – 158. Lonon: Thomas Telford/Institution of Civil Engineers.

Buivid, M.G., Wise, D.L., Blanchet, M.J., Remedios, E.C., Jenkins, B.M., Boyd, W.F. & Pacey, J.C. (1981). Fuel gas enhancement by controlled landfilling of municipal solid waste. *Resources & Conservation*, 6, pp 3-20.

Caterpillar (1995). *Cat 816B and 826C compaction study vs Hanomag CD 280, Bomag 601 RB*. Caterpillar Product Information Performance Report.

Chen, T. & Chynoweth, D.P. (1995). Hydraulic conductivity of compacted municipal solid waste. *Bioresource Technology* 51, pp 205-212.

Department of the Environment (1995). Waste Management Paper no. 26B. London: Her Majesty's Stationary Office.

Di Stefano A.B. (1993). Settlement of Beddingham landfill. *Proceedings of the 29th Annual Conference of the Engineering Group of the Geological Society of London* (ed S P Bentley).

Dixon, N. & Jones, D.R.V. (1998). Stress states in and stiffness of landfill waste. In *Geotechnical engineering of landfills.* London: Thomas Telford/Institution of Civil Engineers

Edil, T.B., Ranguette, V.J., & Wueller, W.W. (1990). Settlement of municipal refuse. In *Geotechnics of waste fills - theory and Practice.* ASTM STP 1070 (ed. A Landva and G D Knowles). Philadelphia, USA: American Society for Testing and Materials.

Gasparini, P., Saetti, G.F. & Marastoni, M. (1995). Experimental research on msw compaction degree and its change with time. *Proceedings of the 5th International conference on landfills, Sardinia 95* (ed. T H Christensen, R Cossu and R Stegmann) 2, pp 833-842. Cagliari: CISA Environmental Sanitary Engineering Centre.

Gotteland, P., Lemarechal, D. & Richard, P. (1995). Analysis and monitoring of the stability of a domestic waste landfill. *Proceedings of the 5th International conference on landfills, Sardinia 95* (ed. T H Christensen, R Cossu and R Stegmann) 2, pp 777-787. Cagliari: CISA Environmental Sanitary Engineering Centre.

Gronow, J. (1996). Bioreactors and landfill research. *Proceedings of the 1996 Annual Conference of the Institute of Waste Management*, Torbay, UK.

Halvadakis, C.P., Findikakis, A.N., Papelis, C. & Leckie, J.O. (1988). The Mountain View controlled landfill project field experiment. *Waste Management and Research.* London, UK, 6, pp103-114.

Hilde, J.L. & Reginster, J. (1995). Three-year study of vertical and horizontal movements in a msw landfill. *Proceedings of the 5th International conference on landfills, Sardinia 95* (ed. T H Christensen, R Cossu and R Stegmann) 2, pp 855-860. Cagliari: CISA Environmental Sanitary Engineering Centre.

Jessberger, H.L., Syllwasschy, O. & Kockel, R. (1995). Investigation of waste body behaviour and waste structure interaction. *Proceedings of the 5th International conference on landfills, Sardinia 95* (ed. T H Christensen, R Cossu and R Stegmann) 2, pp 731 - 743. Cagliari: CISA Environmental Sanitary Engineering Centre.

Jones, D.R.V. & Dixon, N. (1997). The long-term stability of landfill side slopes. *Proceedings of the 6th International conference on landfills, Sardinia 97* (ed. T H Christensen, R Cossu and R Stegmann) 3, pp 517 - 523. Cagliari: CISA Environmental Sanitary Engineering Centre.

Kanou, H., Doi, Y., Imaizumi, S. & Tsuboi, M. (1997). Evaluation of geomembrane stress on side slopes caused by settlements of wastes. *Proceedings of the 6th International conference on landfills, Sardinia 97* (ed. T H Christensen, R Cossu and R Stegmann) 3, pp 525 - 534. Cagliari: CISA Environmental Sanitary Engineering Centre.

Kinman, R.N., Nutini, D.L., Walsh, J.J., Vogt, W.G., Stamm, J. & Rickabaugh, J. (1987). Gas enhancement techniques in landfill simulators. *Waste Management and Research* 5, pp13-25.

Klink, R.E. & Ham, R.K. (1982). Effects of moisture movement on methane production in solid landfill samples. *Resources Conservation* 8, pp 29-41.

Knox, K. (1996). Leachate recirculation and its role in sustainable development. *Proceedings of the Institute of Waste Management*, March, pp 10-15.

Konstantinos, A., Papachristou, E. & Georgios, D. (1997). Settlement measurements at the msw landfill of Thessaloniki, Greece. *Proceedings of the 6th International conference on landfills, Sardinia 97* (ed. T H Christensen, R Cossu and R Stegmann) 3, pp 545 - 549. Cagliari: CISA Environmental Sanitary Engineering Centre.

Landva, A.O. & Clark, J.I. (1990). Geotechnics of Waste Fill. In *Geotechnics of Waste Fills - Theory and Practice*. (eds. A Landva & G D Knowles). ASTM STP; 1070.

Morris, D.V. & Woods, C.E. (1990). Settlement and engineering considerations in landfill and final cover design. *Geotechnics of waste fills - theory & practice*. ASTM STP 1070 (ed. A Landva and G D Knowles). Philadelphia, USA: American Society for Testing and Materials.

Powrie, W. & Beaven, R.P. (in preparation). The hydraulic properties of wastes and their implications for sustainable landfill practice.

Reynolds, P. & Blakey, N.C. (1995). *Landfill 2000*. Final report for the DoE. Report No. CWM 031/91, 192pp.

Richards, D.J. & Powrie, W. (in preparation). Geometric factors affecting the durability of landfill capping systems following subsidence induced by waste degradation.

RPS Clouston & Wye College (1993). Reclamation of landfill workings to agriculture: phase 2 Report to DOE. DOE Research Report (in preparation 1995).

Townsend, T.G., Miller, W.L., Hyung-Jib Lee & Earle, J.F. (1996). Acceleration of landfill stabilization using leachate recycle. *ASCE Journal of Environmental Engineering*, April 1996, pp 263-268.

van Meerten, J.J., Sellmeijer, J. & Pereboom, D. (1997). Prediction of landfill settlement. *Proceedings of the 6th International conference on landfills, Sardinia 97* (ed. T H Christensen, R Cossu and R Stegmann) 3, pp 535 - 544. Cagliari: CISA Environmental Sanitary Engineering Centre.

Vaughan, P.R. (1994). Assumption, prediction and reality in geotechnical engineering. 34th Rankine lecture. *Geotechnique* 44(4), pp 573 – 609.

Walker, A.N., Beaven, R.P. & Powrie, W. (1997). Overcoming problems in the development of a high rate flushing bioreactor landfill. *Proceedings of the 6th International conference on landfills, Sardinia 97* (ed. T H Christensen, R Cossu and R Stegmann) 1, pp 397 - 408. Cagliari: CISA Environmental Sanitary Engineering Centre.

# Stress states in, and stiffness of, landfill waste

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## Introduction

Increasing demand for landfill facilities has lead to the use of void spaces (typically rock quarries) with steep boundary side slopes. Given the present reliance in the UK on landfilling for the disposal of waste, this trend is set to continue. Structural integrity, and hence performance as a barrier to gas and leachate, of mineral, geosynthetic or composite lining systems for these vertical and near vertical slopes is controlled by the interaction between the waste and barrier system. Novel barrier systems are being developed and used for steep side slopes (e.g. reinforced earth, polystyrene face supports and buttressed clay barriers) with a limited understanding of the factors controlling both the short-term construction related, and long-term waste degradation controlled, deformations. In addition, despite the majority of present designs relying on waste in part for their stability (i.e. a heterogeneous material whose engineering properties change during degradation), there is a dearth of published records on barrier systems which have been instrumented and monitored in order to demonstrate satisfactory performance. While it is unlikely that these barrier systems will fail catastrophically with the barrier suffering deformations of several metres (i.e. because the waste provides some support), the low stiffness of household waste material will result in movement of the barrier into the waste until limit equilibrium conditions are established.

Any assessment of barrier/waste interaction requires information on the in situ stresses within the waste body, the lateral stiffness and compressibility of the waste and the time dependent variation of these parameters as the waste degrades. While there is a growing body of information on vertical compressibility of waste (e.g. Powrie *et al.,* 1998), there is still limited data on in situ stresses and stiffness of emplaced material. This paper describes a novel method for obtaining these parameters based on the pressuremeter, and presents the results of initial field trials. Results from in situ tests undertaken in both recent (1 to 3 year old) and partially degraded (11 year old) household waste at depths of 1.7 to 12 metres below ground level are discussed, and proposed further work described.

## Issues of barrier stability

Numerical modelling of typically used barrier configurations which rely on the presence of waste to provide lateral support have demonstrated, not surprisingly, that the waste properties control performance (Reddy *et al.,* 1996, Jones & Dixon, 1998b). This leads to a number of concerns:

- Deformation of the lining system is controlled by in situ stress conditions and the stiffness of the emplaced waste. Of particular importance is the strain incompatibility of traditional mineral/geosynthetic lining materials and the waste, and specifically the large strains which are likely to occur for the lining system/waste body to reach equilibrium. There exists uncertainty regarding the integrity of the lining system in the short-term under these conditions.
- Degradation of the waste with time will alter its mechanical properties, and thus influence the long-term stability and hence potentially the integrity of the lining system.
- The shear stresses mobilised at interfaces within composite barrier systems are strain dependent, and are influenced by the forces transmitted into the lining system as the waste body deforms. Research carried out by Gilbert & Byrne (1996) and Jones & Dixon (1998a) has demonstrated that such waste movements can result in interface shear strengths being reduced to residual conditions over significant areas of the side slopes and base sections. Hence, a decision on the magnitude of interface shear strengths which can be relied upon to provide the required degree of stability can not be made without understanding the interaction between the waste and lining system.
- There is a dearth of information regarding the stresses in the barrier components resulting from the barrier/waste interaction.

To date there exists only limited detailed information on the performance during construction, waste placement and operation of barrier systems used to line steep slopes. Hertweck (1997) described a large scale field trial which was undertaken in Germany to investigate the interaction between a specific design of a steep side slope barrier system and waste (the barrier design investigated was a compacted clay liner supported by a gabion wall installed on a slope of $80^{\circ}$), and compared the observed trial behaviour with the performance of the actual barrier system obtained by in situ monitoring. Findings from this detailed study included:

a) the barrier experienced significant vertical and horizontal strains, with the magnitude dependent on the stiffness of the waste body;

b) the method of construction, including the phasing of barrier construction and supporting waste lifts, had an influence on the magnitude and distribution of barrier deformations;

c) differential strains were found in the barrier components; and

d) a number of potential failure mechanisms were predicted resulting from the magnitude of deformations required for equilibrium between the barrier and waste body to be reached (see Figure 1).

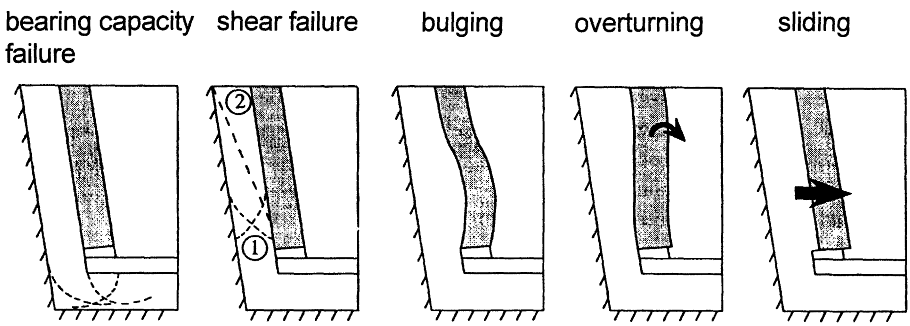


**Figure 1** Potential failure mechanisms of a clay steep slope sealing system

Hertweck (1997) concluded that ultimate limit state and serviceability limit state must be examined for each barrier design separately and should be checked by appropriate in situ measurements. The landfill barrier system investigated is one of only a very small number which have been instrumented to check the stress/strain behaviour during construction and operation.

## Waste properties

The majority of research on the mechanical properties of household waste has to date concentrated on shear strength and compressibility (e.g. Beaven & Powrie, 1995; Jessberger, 1994; Kolsch, 1995; Landva *et al.*, 1984; Van Impe & Bouazza, 1996; and Watts & Charles 1990). As with all particulate materials obtaining undisturbed samples for use in laboratory tests is problematic. The heterogeneous nature of waste also dictates that large sample sizes should be used in order that they be representative. In most cases it is not possible for large undisturbed samples to be obtained, and this has lead to the majority of laboratory studies using processed (e.g. milled) and re-compacted samples. While these can provide useful information related to the general mechanisms of waste behaviour, these measured mechanical properties are of limited use and can not be applied to field problems with any confidence.

The deficiencies of using relatively small laboratory samples has lead to the development of a limited number of large scale test facilities for assessing unprocessed wastes, although there are still problems associated with sample disturbance due to the waste having to be re-compacted in the test apparatus. Tests developed include a large shear box (Kolsch, 1995) and a compression cell (Beaven & Powrie, 1995). Studies of certain properties have been undertaken using in situ waste. These include trial failures of artificially steepened slopes to obtain shear strength parameters, and compression experiments to obtain stiffness parameters for use in settlement calculations. However, to date there is no evidence in the literature of any investigations to measure horizontal stresses in, or lateral stiffness of, in situ household waste. The small sample sizes which can be sensibly obtained, and the inevitable disturbance which will be caused, has lead to the development of an in situ testing technique based on the pressuremeter to measure these important parameters.

## Pressuremeter testing

### Justification

An approach based on in situ testing has been devised, due to the difficulties of obtaining representative laboratory test samples described above. In situ testing has been carried out using commercially available Cambridge type pressuremeters. A pressuremeter is a cylindrical device designed to apply a uniform pressure to the walls of a borehole by means of a flexible membrane. This membrane is expanded against the surrounding material by means of gas under pressure supplied from the ground surface. Outward radial deformation of the waste occurs as the membrane expands. The object of the test is to obtain the relationship between the applied pressure and the deformation of the soil, and from this information the in situ stress conditions and deformation properties of the material surrounding the pressuremeter can be obtained. Pressuremeters were used for this study for the following reasons:

- The device measures the average stress acting on the membrane and hence any large variations in stress due to the heterogeneous nature of the waste will be averaged. While this may be undesirable when using the system for accurate measurements in materials with anisotropic horizontal stresses, it is an advantage in this study where the material is non uniform and averaged values of stress and stiffness are preferable to the present lack of information.
- Relatively large strains can be achieved, with strains typically in the order of 10-15% being obtained using the standard equipment. Measurement of the large strain behaviour is important because of the high compressibility of waste material, and the pre-failure strain hardening behaviour which has been observed in laboratory tests (Jessberger & Kockel, 1993).

- Pressuremeters involve a relatively large volume of material in each test, which is again important due to the variable nature of the waste. Tests in soils have indicated that material up to 40 times the pressuremeter diameter can influence the results (Clarke, 1995).

## Borehole Formation

A critical element of pressuremeter testing is the insertion of the instrument to the desired test depth. Analyses of the test results are based on an assumption that minimal disturbance is caused by the insertion process. Given the heterogeneous nature of waste there was initially some uncertainty as to the preferred method for introducing the pressuremeter to the required depth, while causing minimum disturbance to the surrounding waste body. There are four potential methods for installing the pressuremeter, which are outlined below:

**i)** A self boring pressuremeter (SBP) can be used to form its own borehole from ground level to the required depth. The borehole is not cased and relies on the "reinforced" nature of the waste to prevent collapse behind the advancing instrument (a behaviour which has been observed in many boreholes and steep excavations formed in waste). This test method should produce the minimum amount of disturbance, and does not require the presence of additional equipment on site (e.g. a cable percussion or rotary drill rig).

**ii)** If the self boring method works in waste but drilling rates are slow, it may be preferable to form the main borehole using a drill rig, and then install the pressuremeter by self boring from the base of the hole such that it forms its own pocket.

**iii)** If the self boring technique does not perform satisfactorily in certain wastes, an alternative method entails forming a borehole to the require depth as outlined in ii) above, and subsequently forming a pocket at the base of the hole for the pressuremeter using a thin wall shell, barrel auger etc. The aim is to form a pocket with the same diameter as the instrument. A pre-bored pressuremeters (PBP) is inserted into the pocket and a test performed in the same manor as the SBP.

**iv)** An alternative approach is to push the pressuremeter into the waste, either from ground level or from the bottom of a borehole. Instruments of this type, which result in the host material being completely displaced, are known as full displacement pressuremeters (FDP), with the most commonly available being the cone pressuremeter. Due to the material around the instrument being disturbed by the insertion process, it is common for FDP to have a large strain capacity (i.e. the membrane can be expanded to a larger diameter thus straining a larger volume of material, including undisturbed ground).

It should be noted that when using any of the four methods it may be necessary to abandon some test locations due to the presence of obstructions. Initial field trials have been conducted using methods i) and ii).

## SBP test method

The SBP is about 83 millimetres in diameter and 1.2 metres long, and is a miniature tunnelling machine, the central part of which is covered with an elastic membrane. This membrane is in two parts. The inner layer, which is sealed, is made of polyurethane and is about 1.25 mm thick. This inner skin is covered by an outer layer, known as a 'Chinese Lantern' (CHL), which is formed from stainless steel strips bonded to a thin rubber skin. The CHL is used to take the frictional forces that occur when the instrument is being bored into the ground, and to provide some protection from inclusions that might otherwise puncture the inner membrane. The latter function is particularly important when using the instrument in waste. The foot of the instrument is fitted with a sharp edge. When boring, the instrument is jacked into the ground, and material is removed either by slicing it into small pieces using a rotary cutting device (soft materials) or by grinding the material using a rock roller bit (hard materials). The instrument is connected to the jacking system by a drill string. This is in two parts, an outer casing to transmit the jacking force and an inner rod which is rotated to drive the boring device. The cutting material is flushed back to the surface through the annulus between the rods and outer casing. Water is the most common flush fluid used. Disturbance is minimised by optimising the boring method, flush fluid pressure and jacking force. A schematic of a SBP is shown in Figure 2.

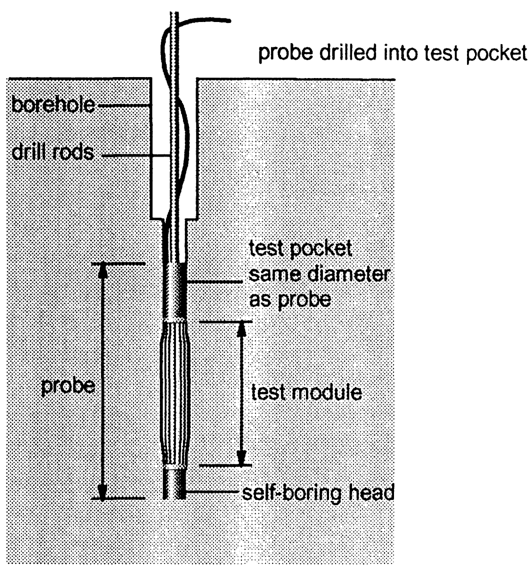


**Figure 2** Self boring pressuremeter (Clarke, 1995)

Once inserted to the test depth the pressuremeter membrane is expanded and readings of displacement against applied pressure are logged, and plotted as a loading curve. This loading curve can be solved directly using mathematical expressions for the expansion of a cylindrical cavity. The solution is conventionally quoted in terms of strength and stiffness parameters for the material tested; specifically shear modulus, shear strength and in situ lateral stress. The above description of the SBP is based on that of Cambridge Insitu (1998).

## Field trials

### Site details

Selection of an appropriate test site was an important consideration, as the aim was to develop a test procedure at the same time as obtaining preliminary data. A site with a well known construction history and relatively uniform waste type was required in order to minimise the number of factors affecting the results. The following criteria were used for site selection:

- Only sites which were formed from household waste were considered. Sites where co-disposal with commercial or industrial waste had occurred were avoided due to possible difficulties drilling through such mixed material (e.g. building rubble).
- The construction history needs to be well documented including information on the history of phased filling, placement and compaction methods used, thickness of waste layers, material used for daily cover and distance between cover layers.
- A minimum thickness in the order of 20 metres of wastes was required in order to assess the performance of the pressuremeter over a range of stress levels.
- The site should have areas of waste of different ages to enable the effects of degradation to be investigated.

The site selected was Calvert, which is located equidistant from Buckingham and Aylesbury, and operated by Shanks & McEwan (Southern Waste Services) Limited. Landfill operations, which started in 1980, are back-filling an old brick pit formed in the Oxford Clay. Waste from collection rounds is delivered to site by train from the Bristol and London areas, and this results in it being almost entirely household household material. The maximum depth of waste is in excess of 20 metres, with each cell taking approximately one to two years to fill. Waste is compacted using a dead-weight roller in approximately one metre thick layers, with typically two such layers placed before use of a 0.3 to 0.5 metre deep clayey soil cover layer. The final capping system is a 1 to 2 metre thick compacted clay layer.

### Testing schedule

In September 1997 the self boring technique was used to insert the pressuremeter from the base of boreholes formed using the barrel auger drilling technique (method ii). Tests were conducted in both recent (1 to 3 year old) waste at depths of 3.5 to 10.7 metres below ground level, and one test in partially degraded (11 year old) household waste at a depth of 11.7 metres below ground level. A second trial was conducted in May 1998 using a SBP which was operated from the ground surface (method i), with tests carried out in 2 year old waste at depths of 1.7 to 3.5 metres.

The composition of the fresh waste (1994 to 1996) as retrieved during borehole formation was approximately 40% plastics; 40% paper and organic material; 10% textiles; and 10% timber, metals and brick. The material obtained from the test depth in the partially degraded material (1987) consisted of approximately 20% plastics; 10% paper; 60% degraded material; and the remaining 10% was textiles, brick and metals.

## Analysis methods and preliminary results

Results from a pressuremeter test are presented as plots of pressure against radial displacement (i.e. pressure vs radial strain) for each of three separate sensors used to measure expansion of the cavity. However, the values of displacement obtained do not necessarily give the correct deformation of the expanding borehole wall at the sensor location as the axis of the instrument can move. Therefore, it is standard practice to use plots of pressure vs average strain. This is also required due to the analyses method assuming isotropic material surrounding the cavity. Figure 3 shows a typical pressure/displacement plot, which is for a test in one year old waste at a depth of 5.5 metres below ground level. It should be remembered that to date pressuremeters, and hence the associated analyses methods, have been developed for use in soils and rocks, and therefore the application of the standard analyses techniques to the results of tests in waste material is open to debate.

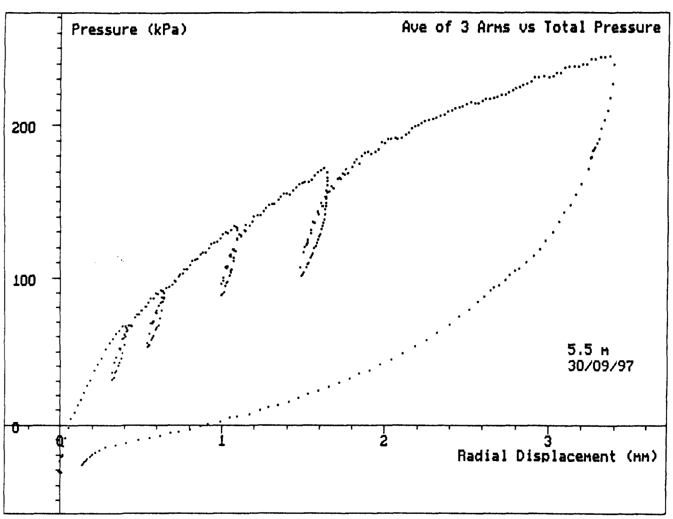


**Figure 3** Example pressure vs radial displacement plot (1 year old waste)

### *In situ horizontal stress*

In order to carry out analyses of the data it is necessary to determine the origin for expansion of the cavity (i.e. to take into account any disturbance). For the SBP it is assumed that some stress relief will occur and the origin is taken as the point where the in situ conditions are restored to the cavity. It is possible to recognise the in situ lateral stress by inspection (the lift-off method), as being the pressure required to cause movement of the membrane. Where disturbance is more pronounced it is taken as the pressure to cause significant movement, as indicated by an abrupt change in the slope of the pressure/radial strain curve. Application of this technique to the waste test data has been problematic as a result of the significant degree of disturbance caused by inserting the SBP. Therefore, a second method developed by Marsland & Randolph (1977) has also been used. This method employs an iterative approach based on identifying the onset of plastic behaviour. Although this method can be considered more robust than the lift-off approach it is still influenced by disturbance. An introduction to both these methods is provided by Mair & Wood (1987).

The estimated values of in situ horizontal stress ($\sigma_h$) can be used to calculate coefficients of earth pressure at rest ($K_0$) where:

$$K_0 = \sigma_h / \sigma_v$$

Using an assumed bulk unit weight of 10 kN/m$^3$ for the waste and a bulk unit weight of 20 kN/m$^3$ for the compacted clay cap, values of $K_0$ have been calculated and plotted against depth in Figure 4. It can be seen that there is no clear relationship between lateral stress and vertical stress. It is considered (Cambridge Insitu, 1998) that the heterogeneous nature of the waste tested resulted in the SBP being over drilled. This was probably caused by items of waste such as pieces of wood, metal or brick being pushed ahead of the SBP (i.e. into the underlying compressible waste) hence resulting in a cavity with a larger diameter than the instrument. This has resulted in the calculated values of $K_0$ showing considerable scatter, and has lead to confidence in the values being low. However of some interest is the one test in the partially degraded waste which indicates a higher value than the fresh waste. An explanation could be that as the waste degrades and settles it becomes denser with an associated increase in horizontal stress, although any conclusions will remain speculative until further tests are undertaken.

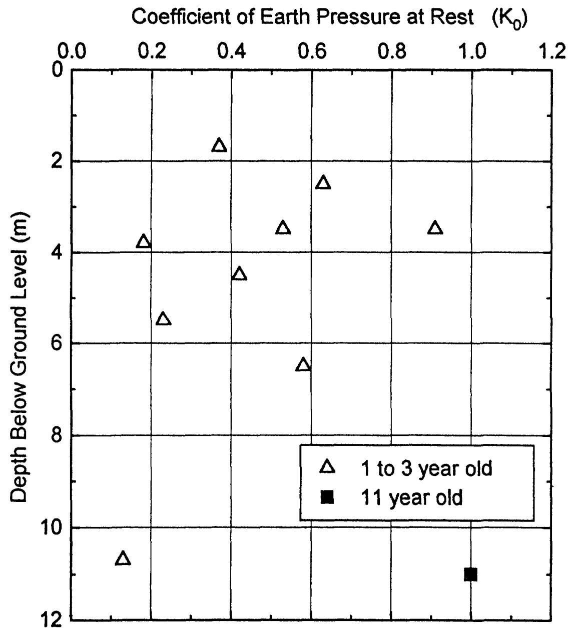


**Figure 4** Coefficient of earth pressure at rest vs depth below ground level

### *Stiffness parameters*

Stressing the material surrounding the instrument enables information on shear modulus (G) to be obtained using a number of techniques, from elements of the pressure vs radial strain curve. The preferred method, and the one used in this study, calculates the slope of the cord bisecting small unload/reload cycles (see Figure 3). This method allows the calculated shear modulus to be related to the mean strain and pressure, and the ranges of strain and pressure, during the cycle. It should be noted that there are a number of definitions of shear modulus. The values presented here are specific to the method of measurement and calculation.

In order to investigate any trends of shear modulus with depth below ground level it is first necessary to use the results from unload/reload loops, carried out at different strain levels, to values at a reference strain. Measured shear modulus values were plotted against the average radial strain during each loop. For a given test an approximate linear relationship was found between G and strain. This line was then used to obtain the shear modulus for a cavity strain of 1%. Figure 5 is a plot of shear modulus at 1% strain against the test depth below ground level. It can be seen that there is a relationship between G and depth for the fresh waste. The shear modulus decreases with depth between 1.7 and approximately 4.5 metres below ground level and then increases below this depth, except for the test at 10.7 metres which indicates a very low stiffness.

Whether this value is due to a general trend with depth, an isolated area of compressible material or a function of the test method (e.g. the introduction of water from the boring which has caused local softening), is unclear. The test in partially degraded waste indicates a relatively high shear modulus at a depth of 11 metres.

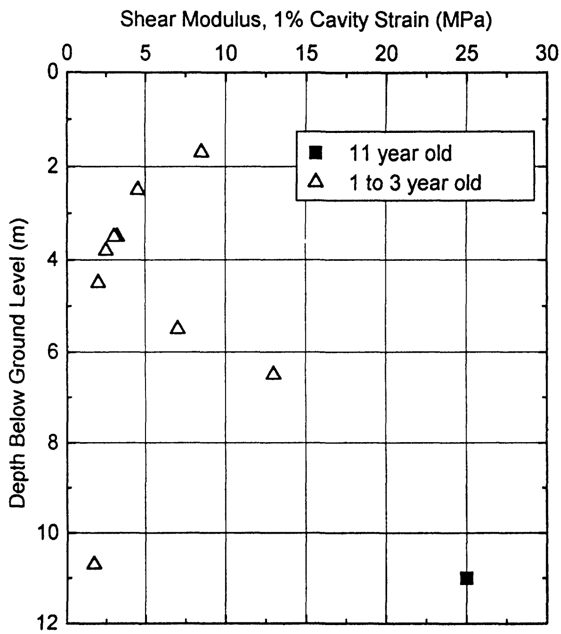


**Figure 5** Shear modulus at 1% cavity strain vs depth below ground level

It is possible to explain the observed G vs depth relationship in the fresh waste by considering the influence of cap formation. The amount of compaction used during waste placement is relatively light compared to that used to form a 1 to 2 metres thick low permeability cap. It is likely that formation of the capping layer would also result in additional compaction of the top few metres of waste, hence increasing the density and therefore stiffness of this upper layer. Below this modified zone the density and stiffness of waste would be expected to increase with depth due to the increase in vertical stress (i.e. overburden). However, despite the very clear relationship between G and depth it must be remembered that considerable variation in G should still be expected due the heterogeneous nature of the material (e.g. as demonstrated by the 10.7 metre test). This additional compaction would also be expected to increase the horizontal stresses in the layer of waste beneath the cap. Although there is considerable scatter on the $K_0$ vs depth plot (see Figure 4) it is possible to interpret the plot as showing this general trend (i.e. higher $K_0$ values in the upper layer of waste).

In laboratory unconfined compression tests on milled waste it has been observed that the stiffness increased during application of stress as a result of the waste compressing (Jessberger & Kockel, 1993). This is consistent with the stiffness increasing with depth (Figure 5), but in addition means that a relationship would be expected between the mean pressure during a unload/reload loop and the measured shear modulus (i.e. if the waste behaves similar to a compressible fully drained soil). The influence of increasing stiffness with depth is represented by the test pressure because the deeper waste will in general require larger pressures to deform the material. Figure 6 is a plot for the fresh waste of shear modulus against average pressure during the unload/reload loop used to calculate the G value. Despite there being some scatter of the data a clear relationship of increasing shear modulus with pressure, and hence lateral stress, is indicated. However, this trend may not be so pronounced for more degraded waste with higher density and moisture content, as behaviour may be closer to that of a low compressibility partially drained soil during application of stress.

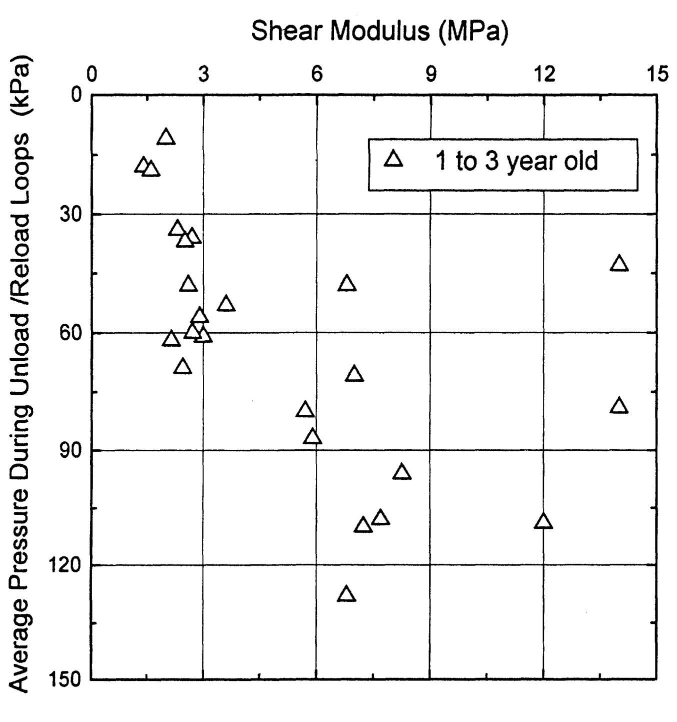


**Figure 6** Shear modulus vs average pressure during unload/reload loop

## Comparison with values from the literature

The results obtained from the small number of SBP tests indicate relationships between stiffness and depth, including effects of construction processes, and stiffness with stress level. However, it must be appreciated that the results relate to mainly fresh household waste at one site. It is important to compare the results from this investigation with measurements of waste stiffness described in the literature. The problem in undertaking such a comparison is that a number of different forms of stiffness are reported. The main parameters being Young's modulus (E) from laboratory triaxial compression tests, and constrained modulus (D) (i.e. one dimensional compression) obtained from both small and large scale laboratory tests and field studies.

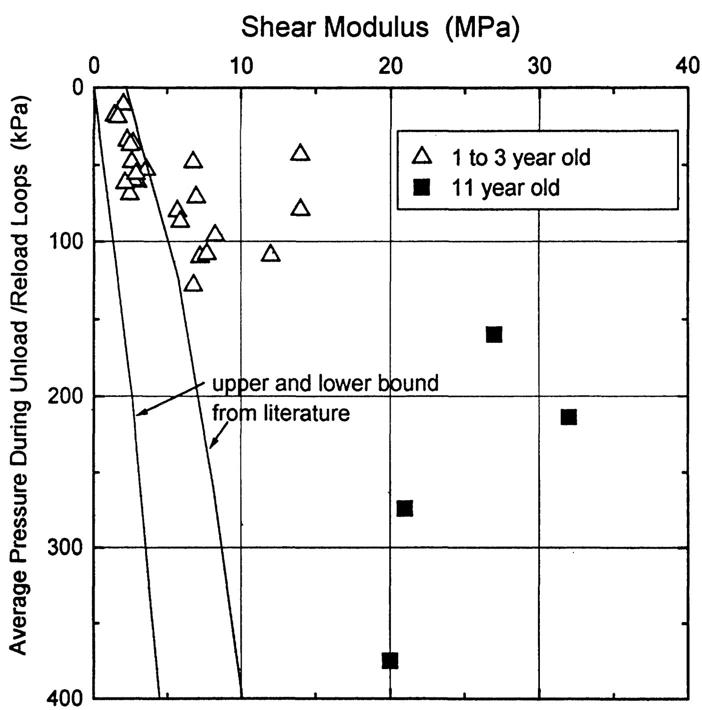


**Figure 7** Comparison of stiffness values from SBP tests and the literature

In order to compare the results from the various sources the stiffness values have been converted into shear modulus values. Based on an assumed Poisson's ratio for household waste of 0.1 (Jessberger & Kockel, 1993) it can be shown that constrained modulus (D) is approximately equal to 2.2.G. Van Impe & Bouazza (1996) summarised stiffness values available in the literature and produced lower and upper bounds for the relationship between stiffness modulus (E) and vertical stress. It should be noted that the values from the literature cover a range of waste composition, age and includes results obtained using in situ and laboratory test techniques. These upper and lower bounds are compared to the SBP values in Figure 7. Although there is a reasonable agreement for the lower stresses it can be seen that the SBP tests indicate significantly higher stiffness values for the higher stress range. This could be explained by differences in the waste type, although it is probable that the most important factor is that laboratory test results were used to define the boundaries at the higher stresses, and these could have been influenced by variations in the density and structure of the test samples resulting from re-compaction and in some cases pre-treatment. This would tend to confirm that in situ measurement of stiffness parameters is the most appropriate approach.

## Summary and future work

Optimisation of landfill barrier design can not be achieved without information on the mechanical properties of the waste, specifically in situ horizontal stresses within the waste body, lateral stiffness of the waste and the time dependent variation of these parameters as the waste degrades. The assessment of existing and new barrier designs for steep side slopes which rely on waste to provide lateral support, will be limited without this knowledge.

While there is a growing body of information on waste compressibility, permeability and shear strength there is still very limited information on in situ stresses and stiffness of as placed material. However, a method of measuring these parameters has been developed based on the pressuremeter. Preliminary results have been obtained for household waste, and these demonstrate that the technique is viable. Measurements of in situ stress and horizontal stiffness have been made in both recent (1 year old) and partially degraded (11 year old) household waste at depths of 1.7 to 12 metres below ground level. A self boring pressuremeter was used to obtain the parameters with shear modulus values calculated from unload/reload loops.

For the fresh waste the SBP tests indicate a trend of initially decreasing (1.7 to 4.5 metres) and then increasing stiffness with depth. The higher stiffness in the upper layer of waste is believed to be due the compactive effort used to construct the low permeability clay capping layer. Although there is no clear relationship between horizontal stress and depth, there is an indication that there may be higher stresses in the zone affected by cap formation. A general trend of increasing stiffness with test pressure has been found which confirms that the fresh waste is strain hardening (i.e. this is consistent with the general trend of increasing stiffness with depth of burial). As only one test was carried out in the partially degraded waste it is not possible to draw any significant conclusions on the effect degradation has on the mechanical properties.

Obviously because of the heterogeneous nature of waste, some abortive tests are inevitable due to obstructions such as pieces of metal and masonry, and the values obtained from successful tests provide only a guide to the magnitude of the parameters. However, measured against the previous background of limited laboratory and field testing on disturbed and often processed samples of waste, this study is valuable and unique.

The only cost effective method for assessing the performance of a range of barrier systems applied to different slope geometries, and subjected to varying waste support conditions and construction sequences, is to develop a numerical model. Based on the success of the preliminary investigation described in this paper it is planned to extend the pressuremeter test programme to obtain the range of parameters required for use in numerical modelling studies. It is proposed to extend the use of the SBP test method to waste of different age and depth of burial, and pre-treated wastes.

This ongoing research aims to obtain relevant waste material properties required as input parameters for numerical models, to assess current steep side slope landfill barrier designs using numerical modelling techniques, to validate the model by the instrumentation of barrier systems, and hence to produce guidance for designers.

## Acknowledgements

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## References

Beaven, R.P & Powrie, W. (1995). Determination of the hydrogeological and geotechnical properties of refuse using a large scale compression cell. *Proc. 5th Int. Landfill Symp.*, Sardinia (ed. T.H. Christensen, R. Cossu and R. Stegmann), 2, 745-760.

Cambridge Insitu (1998). *Report on Calvert pressuremeter tests* (Ref. CIR 1031/98), 1, A1-A6.

Clarke, B.G. (1995). *Pressuremeters in Geotechnical Design*. Pub. Blackie Academic & Professional.

Gilbert, R.B. & Byrne, R.J. (1996). Strain-softening behaviour of waste containment system interfaces. *Geosynthetics International*, 3, 2, 181-203.

Hertweck, M. (1997). Structural analysis of steep slope sealing systems supported by waste. *Proc. 6th Int. Landfill Symp.*, Sardinia, October, 3, 507-516.

Jessberger, H.L. (1994). Geotechnical aspects of landfill design and construction. Part 2: material parameters and test methods. *Proc. Instn Civ. Engrs Geotechnical Engineering*, 107, 105-113.

Jessberger, H.L. & Kockel, R. (1993). Determination and assessment of the mechanical properties of waste material. Proc. Geotechnics Related to the Environment, *Waste Disposal by Landfill*, Bolton, UK, 313-322.

Jones, D.R.V. & Dixon, N. (1998a). The stability of geosynthetic landfill lining systems. *Proc. Geotechnical Engineering of Landfills Symp.*, Pub. Thomas Telford, (In press)

Jones, D.R.V. & Dixon, N. (1998b). Landfill side slope stability: modelling waste/barrier interaction. (In prep.)

Kolsch, F. (1995). Material values for some mechanical properties of domestic waste. *Proc. 5th Int. Landfill Symp.*, Sardinia (ed. T.H. Christensen, R. Cossu and R. Stegmann), 2, pp 20.

Landva, A.O., Clarke, J.I., Weisner, W.R. & Burwash, W.J. (1984). Geotechnical engineering and refuse landfills. *6th National Conf. on Waste Management in Canada*, Vancouver, Nov., pp 57.

Mair, R.J. & Wood, D.M. (1987). Pressuremeter testing, methods and interpretation. *Construction Industry Research Information Association Project 335*, Pub. Butterworths.

Marsland, A. & Randolph, M.F. (1977). Comparison of the results from pressuremeter tests and large insitu plate tests in London Clay. *Geotechnique*, 27, 2, 217-243.

Powrie, W., Richards, D. & Beaven, R. (1998). Compression of waste and implications for practice. *Proc. Geotechnical Engineering of Landfills Symp.*, Pub. Thomas Telford, (In press)

Reddy, K.R., Kosgi, S. & Motan, E.S. (1996). Interface shear behaviour of landfill composite liner systems: a finite element analysis. *Geosynthetics International,* 3, 2, 247-275.

Van Impe, W.F. & Bouazza, A. (1996). Geotechnical properties of MSW. *Proc. 2nd Int. Congress on Environmental Geotechnics*, Osaka, Japan, pp 30.

Watts, K.S. & Charles, J.A. (1990). Settlement of recently placed domestic refuse landfill. *Proc. Instn Civ. Engrs*, Part 1, 88, 971-993.

# Issues Related to Mineral Liners

The papers in this section deal with the selection, use, performance, testing and validation of mineral liners which are designed and constructed to act as protectors of the aquatic environment surrounding landfill depositories. Mineral liners provide a relatively cheap, robust system which it may be argued have a proven track record. They may be constructed using traditional earthworks plant to have a low permeability and are natural attenuators of contaminates in leachate. On the down side they require careful selection, pre-treatment and compaction in order to ensure that the design requirements are achieved. The roles of clay mineralogy, soil structure, and the sensitivity of lining material are also important issues, as is the adequacy of the acceptance criteria. The control of moisture content and the conditioning of the materials, which might include the need for screening or comminution, are further matters that need to be carefully appraised.

It is essential to always bear in mind the purpose of liner construction. In this respect the wider issues of evaluating the potential impact of contaminant escape on the surrounding environment, the identification of sensitive receptors, and the mechanisms of contaminant migration and attenuation all need to be understood. These have implications for the techniques of risk assessment, and highlight the necessity of a wide appreciation of a number of scientific disciplines from geotechnical engineering and geology to chemistry, hydrogeology, hydrology and environmental engineering. All these factors and disciplines are significant to a full appreciation of the potential risks to man and the environment from the development of landfill sites.

In the first paper, Murray addresses the properties and testing of mineral lines. Though a low permeability and thus advective movement of contaminates is usually deemed to be the overriding requirement of a mineral liner, a discussion is presented on the significance of diffuse contaminant migration which also needs careful consideration. It is stressed that there are many uncertainties relating to the laboratory and field testing which are undertaken to establish design parameters and to give confidence in performance in the field. Differences of behaviour between laboratory test samples and full scale material masses lead to uncertainties. Additionally the environmental influence on lining materials and the liner/leachate compatibility cast doubt on the meaning of the test results and question whether the design parameters are achieved in the construction lining.

The paper by Hird, Smith & Cripps continues the theme of the difficulties associated with natural mineral liners to the use of colliery spoil, and exemplifies the above points. The properties of spoil from different mining processes are described and factors influencing permeability are addressed, in relation to both field permeability testing using ring infiltrometers and laboratory testing using flexible wall permeameters. In particular, the size of test sample and the need to replicate in laboratory tests the likely conditions in situ are highlighted, and issues relating to the specification and validation of such materials are outlined.

Jefferis covers the engineering of bentonite enriched soils. Bentonite exhibits a very low permeability and for this reason is an 'attractive' material for enhancing the properties of otherwise unsuitable materials. However, the swelling characteristics and the influence on such materials of the prevailing environment has cast doubts on the long-term performance of bentonite enriched soils. In this paper a simple two phase model is used to represent the bentonite enriched soil, to highlight important parameters for control of the material on site, and to identify situations which may lead to significant increase in its permeability on exposure to aggressive chemical environments.

A major conclusion from the papers presented is that, although advances are being made in our understanding of the efficacy of mineral liners to prevent contaminant escape, there are still many uncertainties which are the subject of current research. Such research must not only aim to provide a better understanding of the behaviour of mineral liners and to improve on the protection afforded but should also, where possible, aim to reduce the cost to those developing landfills.

# Properties and testing of clay liners

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## Introduction

Indigenous clays are the most widely used materials for the protection of groundwaters and surface waters surrounding landfill depositories. A considerable amount of practical experience and experimental understanding has been built up on the use of such materials. However, many uncertainties still remain in their utilisation as either the sole protector of the aquatic environment or in conjunction with proprietary lining materials. A major difficulty with the use of clays is the significant variation in permeability which can occur with relatively small changes in other properties, and a change in moisture content of only 2 or 3% can result in a permeability variation of an order of magnitude or more.

All clays have a finite permeability and potential to attenuate contaminants, and even with a well designed and constructed system, contaminant migration from landfill will occur, be it gradual. Leakage rates in the field are generally recognised as greater than the design rates because scale and structural defects are not wholly represented in laboratory testing (eg. Parkinson, 1991). The acceptability of such escapes must be balanced against the potentially prohibitive cost of attempting perfect containment. For this reason DOE (1996) advocates the concept of 'environmentally safe' landfills based on risk assessment of individual sites and the determination of safe engineering leakage criteria.

The potential of a clay to form a landfill lining is defined herein in terms of 'material suitability'. However, suitable materials may not be capable of achieving the desired permeability without conditioning and a clay is only defined as 'acceptable' following any pre-treatment and CQA testing necessary to establish and prove its compliance with the design requirements (Murray *et al.* 1996). To this end the testing undertaken may be divided into the following:

*Geotechnical engineering of landfills.* Thomas Telford, London, 1998

| | |
|---|---|
| Material Suitability Testing - | |
| Material Acceptability Testing - | Physical Design Testing |
| | Chemical Design Testing |
| | CQA Testing |

These distinctions have been found useful and are introduced to comply with the generalised staged testing and reporting procedures in the investigation, design, specification and construction of landfill linings and also to facilitate an appreciation of the role and significance of the different tests undertaken. Table A1 appended details the testing both in common use and other testing sometimes deemed necessary.

The CQA Plan forms a vital component in the construction of a lining and details the checking procedures, testing and means of ensuring that the emplaced lining achieves the desired standard. At an early stage of development it is also necessary to evaluate the risk and potential impact of pollution migration on the surrounding environment (NRA, 1992) and Regulation 15 of the Waste Management Licensing Regulations (DOE, 1994) requires that a risk assessment is undertaken detailing the potential influences on groundwaters from the discharge of List I and List II substances. NWWRO (1996) addresses this regulation and suggest that it is likely that the Waste Regulation Authority will require the risk assessment report to deal with the wider issues of contaminant escape in assessing the influence of the proposed design on the environment. This will necessitate an appraisal of the potential receptors, their sensitivity, pollution pathways and the likelihood of pollutants impacting the receptors.

In order to appreciate the significance of the testing undertaken on clay lining material in developing a landfill site, it is first necessary to outline the mechanisms of contaminant migration and the roles played by a soil's physical and chemical properties.

## Movement of contaminants through clay liners

For clay liners, the two prime mechanism of contaminant migration are -

(i) Advection (movement of contaminants as a result of permeation of water)
(ii) Diffusion (contaminant migration as a result of concentration gradient)

The influence of dispersion is usually deemed negligible for a clay lining and Rowe (1994) presents an appraisal of the relative importance of advection and diffusion based on simplistic assumptions. It is concluded that for many practical situations, for the typical range of permeabilities of landfill liners of $1x10^{-9}$ to $1x10^{-11}$ m/s, both advection and diffusion are important with diffusion becoming the dominant mechanism at lower permeabilities and advection being dominant at higher permeabilities. For advection, the mass of contaminant transported per unit area per unit time (ie. the mass flux f) is given by:

$$f = -c.k. \frac{dh}{dz} \qquad (1)$$

where, k is the permeability
c is the concentration of contaminant
dh/dz is the negative hydraulic gradient

In accordance with the foregoing argument, for the normal range of design permeabilities, advection is reduced to the same order of magnitude as diffusion and a relatively slow migration of contaminants into the surrounding environment ensues. For this reason, advection is usually deemed the prime mechanistic risk to groundwaters and surface waters surrounding landfill depositories, and low permeability is considered the overriding requirement of a clay liner. However, diffusion may be the prime means of pollution migration not only where the lining is of very low permeability but also where there is net inflow of water into a landfill as diffuse migration resulting from chemical potential can occur against the net permeation direction. The mass of contaminant transported by diffusion per unit area per unit time is given by:

$$f = -nD_e\frac{dc}{dz} \qquad (2)$$

where, n is the porosity of the clay liner
$D_e$ is the diffusion coefficient
dc/dz is the negative concentration gradient

Equations (1) and (2) are additive and constitute the advection-diffusion model in the absence of those mechanisms which remove contaminants from leachate or otherwise reduce the concentration of contamination (eg. Rowe, 1997).

Clays can not only be 'designed' to have a low permeability but can also act as an important medium for the attenuation of diffuse contaminant movement. Such buffering processes may be chemical, physical and biological with overlap between the reactions involved. The following mechanisms of contaminant attenuation are identifiable but they present only a finite buffering capacity and saturation and breakthrough may ultimately occur:

(i) Sorption by ion exchange (major ions particularly heavy metals)
(ii) Sorption by partitioning with organic matter in the soil (heavy metals and organic contaminants such as benzene and toluene)
(iii) Precipitation (heavy metals)
(iv) Oxidation/reduction (or redox reactions)
(v) Organic transformation (such as biodegradation or biological decay)

(vi) Dilution (direct result of dispersion of solutes within pore water and influences all contaminants)

The efficacy of a clay to attenuate and retard the migration of contaminants is influenced by the pollution pathways and residence times and thus by the soil micro- and macro-structure. The ability of a clay liner to retard the physical passage of the leachate not only reduces the advective movement of contaminants but increases the residence time of the permeant within the liner. This can enhance any attenuation which is time dependent. Although the majority of attenuation is likely to be ion exchange, which is rapid, it is generally accepted that the organic strength of the leachate reduces by prolonged residence in a liner material.

Although subject to uncertainties with respect to the permeability and potential attenuation properties in the field, experience would suggest that with good engineering practice and quality control, low permeability barriers can be constructed and prove effective in protecting the environment surrounding landfills. The role of soil testing is to ensure an acceptable end-product with the desired permeability and attenuation potential.

## Material suitability (or identification of possible source material for a landfill lining)

Material suitability relates to the material type and whether it could potentially form a low-permeability barrier. In order to achieve this it is usual to specify the use of a clay with suitable 'material characteristics' (B.S.5930:1981) as defined by its plasticity, material variability and clay content (see Table A1 appended). Fine soils (clays and silts) are defined as having less than 65% coarse material (sand, gravel and larger material) based on the observation that materials with more than 35% fines behave more as cohesive soils than as granular soils. The division between clays and silts on a plot of Plasticity Index (PI) against Liquid Limit (LL) is given by the A-line as shown in Figure 1. Silts plot below the A-line and are generally deemed unsuitable because of the difficulties associated with handling and working such materials, their susceptibility to significant deterioration in properties as a result of water content changes, their high dispersivity and their likely frost susceptibility.

As shown in Table 1, various criteria have been proposed in defining a clay suitable to form a mineral lining. From an examination of the table a reasonable picture emerges of the properties of a clay for it to be considered suitable There are, however, some points of note. In particular, the NRA (1992) set upper limits on clay plasticity based on criteria defined by the Department of Transport (1991) for compaction using earthworks plant. These limits preclude the use of extremely plastic clays which would exhibit very low permeability characteristics but can give rise to problems with stability, deformation, shrinkage and compaction in earthworks. Figure 1, based on

Murray *et al.* (1992), incorporates the NRA criteria and may be used to distinguish between suitable, unsuitable and marginal materials; the latter classification including the Gault Clay and, in their more weathered states, the London Clay and Fuller's Earth where they exceed the NRA limits.

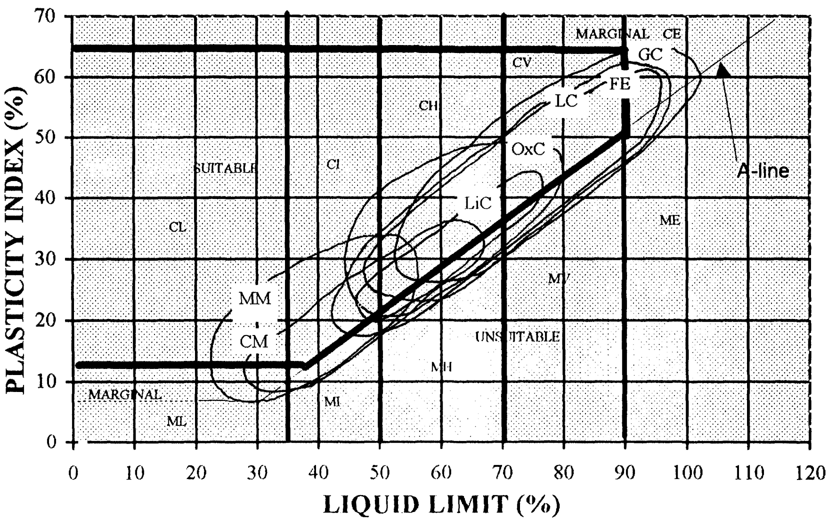
Table 1 also presents limits for the minimum plasticity index of a suitable clay. This is a direct consequence of a marked increase in permeability at low PI. As shown in Figure 1, this might influence the selection of clays such as those derived from the Mercia Mudstone and Coal Measures.

| PROPERTY | REFERENCE | LIMIT/CRITERION |
|---|---|---|
| Plasticity | DOE (1995a) | 30% > PI > 10% |
| | Daniel (1993) | PI > 7-10% |
| | NRA (1992) | LL < 90% |
| | | PI < 65% |
| | Murray *et al.* (1992) | PI > 12% |
| | Gordon (1987) | PI > 15% |
| | Williams (1987) | PI > 15% |
| Percentage fines | Daniel (1993) | clay and silt >20-30% |
| | NRA (1992) | clay particles >10% |
| | Gordon (1987) | clay and silt > 50% |
| Activity (PI/clay content) | DOE (1995a) | > 0.3 |
| Percentage gravel | Daniel (1993) | gravel (>4.76mm) < 30% |
| Maximum particle size | NWWRO (1996) | size must not affect liner integrity |
| | Daniel (1993) | < 25-30mm |

(Note: there are some differences between the ASTM and BS test methods used to determine the suitability criteria in Table 1 but these do not preclude comparison of the proposed criteria)

**Table 1** Properties of saturated clays

The variability of a deposit also influences its suitability. For example, glacial till whilst predominantly clay may exhibit significant variations in plasticity over short distances and contain pockets of sand, silt or other unsuitable materials which may not be easily segregated during excavation. Care must be taken when collating laboratory test results on a deposit to ensure that preferential sampling and testing is taken into account and the influence of material variability is fully assessed. The influence of gravel content on permeability has been examined by Shakoor and Cook (1990) and Shelley and Daniel (1993) and both report a rapid increase in permeability where the gravel content exceeds a critical value which appears to be around 50% to 60%. As shown in Table 1, Daniel (1993) suggests that the percentage gravel should be less than 30% with a maximum particle size of 25 to 30mm although NWWRO (1996) allows larger particle sizes provided they are not likely to prejudice the integrity of the liner. The prime requirement is that there is sufficient fines to fill the pores between gravel particles with low permeability material.




CM - Coal Measures; FE - Fuller's Earth; GC - Gault Clay; LC - London Clay; LiC - Lias Clay; MM - Mercia Mudstone; OxC - Oxford Clay

**Figure 1** Material suitability with typical plasticity ranges for clays from selected strata

## Acceptability of materials (or approval of lining material)

### Physical design testing

A permeability of $1\text{x}10^{-9}\text{ms}^{-1}$ or less is usually specified as the overriding requirement for a clay lining. A clay may have suitable 'material characteristics' but the variation of permeability with moisture content, degree of compaction and soil structure, defined in B.S.5930:1981 as the soil's 'mass characteristics', must also be taken into account. Acceptability relates to the excavation, handling, trafficability, conditioning and compaction of a material which is required to achieve the desired low permeability. The definition of acceptability is comparable with that adopted by the Department of Transport (1991), this specification often being used as a guide to the compaction requirements for a clay lining. It is apparent that a material which is unsuitable is also unacceptable but a material which is suitable will not necessarily be acceptable.

The requirement of ensuring a thoroughly compacted, uniform, homogeneous lining of low permeability will necessitate detailed testing, site monitoring and compaction generally in excess of normal earthworks levels. Acceptability testing encompasses those tests listed in Table A1 appended and which are deemed necessary for design purposes and CQA control.

The following discusses the significance and some important aspects of specific soil tests, and the links between measured properties, which need to be understood by those designing and controlling the emplacement and validation of lining properties.

## *Tests for construction control*

It is important that laboratory compaction tests are reasonably representative of the compaction which can be achieved in the field. Most sites are controlled by the findings from B.S.1377:1990 2.5kg rammer compaction tests or less frequently by B.S. 4.5kg rammer compaction tests. Alternatively, compaction in accordance with the Moisture Condition Value (MCV) test may be used (Parsons and Boden, 1979). Figures 2 presents the results of laboratory compaction tests on a high plasticity clay and indicates that the degree of compaction achieved during the MCV test lies between that achieved by the other two methods. The B.S. 2.5 kg and 4.5 kg tests are based upon applying a given amount of compactive effort to a soil sample whereas the MCV test is based on compacting a soil sample until no further change in density occurs. The general forms of the compaction curves are, however, similar but optimum moisture content (OMC) in the MCV compaction test tends to be closer to the zero air voids line. Experience suggests that more consistent results are obtained using the MCV test than the B.S. compaction tests.

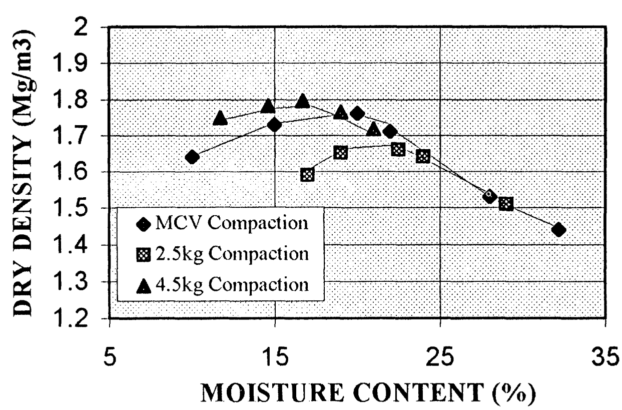



**Figure 2** Dry density against moisture content - CH soil

The degree of compaction achieved in a laboratory 4.5 kg rammer test is often difficult to replicate in the field and in many cases provides an unnecessarily strict requirement. For the higher plasticity clays the 2.5kg method is likely to be adequate but for low plasticity clays a greater degree of compaction may be required in order to achieve the permeability criterion consistently. In this latter case, compaction in accordance with the 4.5kg rammer method may be necessary. Densities obtained using the MCV test provide an intermediate level of compaction which should be achievable for most clays.

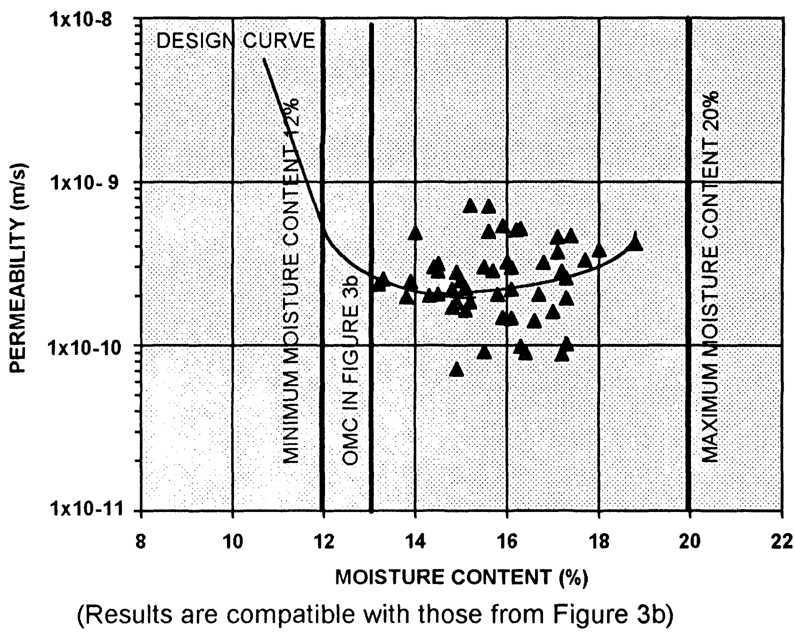



**Figure 3a** Permeability against in-situ compaction moisture content from core samples

As indicated in Figure 3a by the permeability design curve, for a given compaction criterion, dry of the OMC there is a rapid increase in permeability reflecting the lack of remoulding of a clay and the presence of fissures resulting in preferential seepage paths (Mitchell *et al.*, 1965; Murray *et al.*, 1997 amongst others). Obviously, greater compaction at these relatively low moisture contents would result in a reduction in permeability, but this is not always achievable in practice. The identification of 'clod' size as being a major factor in the presence of discontinuities or fissures goes a long way to explaining why there are often large discrepancies between in-situ permeabilities and the lower values obtained from laboratory prepared samples. Benson and Daniel (1990) show experimentally that for a soil compacted dry of optimum, the clod size significantly influences the fissuring present and thus the permeability, while wet of the OMC the clod size is unimportant. The acceptable lower limit to moisture content should therefore not be significantly less than the OMC for the compaction criterion expected to be appropriate under site conditions.

An argument may also be put forward for specifying the PL as the lower limit to moisture content for a clay as this is a measure of the onset of desiccation cracking in a clay subject to remoulding (Murray *et al.*, 1998). Indeed the PL often corresponds closely to the OMC in the B.S. 2.5 kg method where a 'clean' clay is being tested. However, the test for PL precludes material retained on the 425μm sieve and for a clay with mudstone fragments or gravel, or for greater compactive effort, the plastic limit can be significantly wet of the OMC. This is shown in Figure 3b where the material was compacted using the B.S. 4.5 kg rammer method and comprises an intermediate plasticity clay containing a proportion of fragmented mudstone.

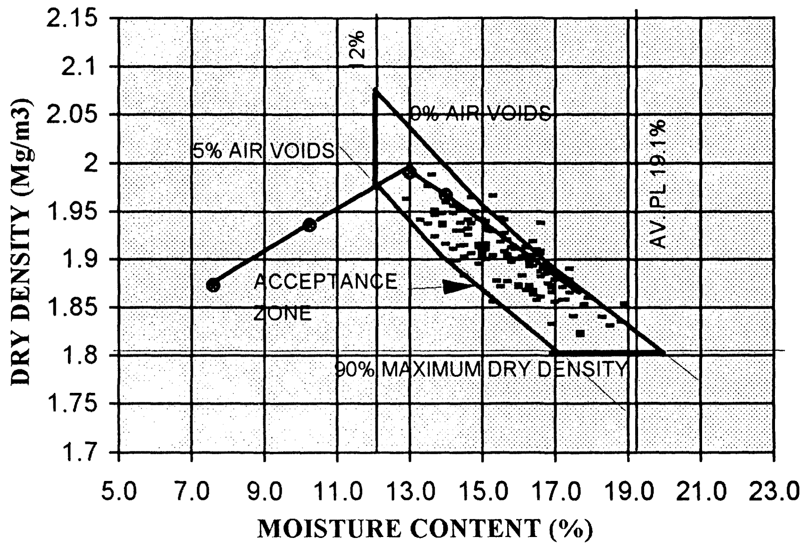



**Figure 3b** Nuclear density meter results against 4.5kg compaction curve

As liner earthworks cannot be controlled directly by permeability measurements, it is necessary to set upper and lower bounds on other material properties determined at the design stage in order to control on-site operations. The limits are usually based on moisture content determinations. If the bounds on moisture content are added to a compaction requirement of a low air voids content (generally 5 to 10%), an envelope of acceptable material and compaction is defined though it is necessary to ensure that at the maximum allowable air voids that the permeability requirement is still met. Figure 3b indicates a typical acceptance zone. Test results indicate that the acceptable lower limit for the moisture content should be dictated by the permeability requirement. However, the upper limit to the moisture content may be dictated by the shear strength of the clay because although the permeability requirement may be met, handling, compaction and trafficking become more difficult. This, in conjunction with stability considerations, dictates the requirement for a minimum shear strength. Typically an undrained shear strength ($c_u$) of no less than 40 to 50 kN/m$^2$ is required in earthworks.

### *Tests to assess permeability and advection properties*

Of particular interest in permeability testing are the findings such as those of Brunelle *et al.* (1987). Their results indicate no notable difference between permeabilities using water or leachate (from an active landfill), but do indicate significant differences between the permeabilities from the three different permeameters used. It may be concluded that, in general, fixed wall permeameters (such as the falling head method of Head, 1986) give higher permeabilities than flexible wall permeameters (such as the constant head triaxial test of B.S.1377:1990) because of side wall leakage and the influence of confining pressure. Amongst other factors, the size of the sample tested also influences the measured permeability, and Barden *et al.* (1969) report on the influence of unsaturated conditions and the significant increase in permeability as saturation is approached.

In practice the leachate levels and thus hydraulic gradients in landfills are kept low by pumping from wells. In laboratory permeability tests the hydraulic gradients are usually significantly higher than in practice for reasons of practical flow measurement. There is an argument as to whether Darcy's law is applicable at low hydraulic gradients and Mitchell and Younger (1967) present results which suggest a deviation from Darcy's law at a threshold hydraulic gradient of about 6, whereas in landfill cells the hydraulic gradient is typically around 1. This deviation may in a large part be a result of the high viscosity of the adsorbed water attached to clay particles. Other factors being equal, some comfort may be gained from the inferred greater laboratory permeabilities under elevated hydraulic gradients compared to those permeabilities which may pertain in the field under lower hydraulic heads.

### *Tests to mitigate physical damage*

Post-construction (before, during and after landfilling) the performance of a clay lining may be influenced by a number of physical factors including:

| | |
|---|---|
| Construction Factors - | Hydrostatic uplift |
| | Instability of oversteep side slopes |
| | Settlement of the ground below the basal lining |
| Environmental Factors - | Desiccation due to drying out |
| | Softening due to uptake of water |
| | Freezing |

| | |
|---|---|
| Landfilling Factors - | Disturbance due to mechanical plant<br>Punching of waste materials into the lining<br>Erosion/suffusion due to movement of free water |

These potentially disruptive influences have to be addressed at the design stage and where deemed necessary investigation and testing should be carried out to facilitate a quantitative appraisal and to evaluate the inherent risks associated with the various factors. This might include such testing for the lining as determination of the total and effective stress parameters, the influence of moisture content changes and the potential for frost heave.

Jessberger (1994) suggests the erosion/suffusion potential (dispersivity) of a clay also warrants careful consideration. These influences are more pronounced in the low plasticity, friable and fissile clays such as those derived from the Coal Measures, Mercia Mudstone and Etruria Marl. Erosion is the removal and transport of particles as a result of liquid flow whereas suffusion is the transport of the fines only, leaving a coarser skeletal structure. Three types of erosion and suffusion may be defined :

| | |
|---|---|
| External: | due to liquid flow along external faces of the clay liner |
| Internal : | due to flow along internal flow channels such as fissures |
| Contact: | movement of particles from the liner at the contact with coarse grained strata such drainage layers |

Tests to evaluate the potential degradation of a clay under conditions of water movement are given in Table A1. These tests were originally devised to address problems with earth dams, river banks and the like where water movements are likely to be far more pronounced. The tests provide a guide to material dispersivity in landfill engineering but with suitable design features and precautions, such as the use of protective membranes, the detrimental effects of erosion and suffusion may be avoided.

## Chemical design testing

### *Tests to assess diffusion and attenuation properties*

In landfills comprising predominantly domestic waste it is the dissolved contaminants in the leachate that are considered of prime concern. However, clays have the potential to adsorb cations and anions from leachate, particularly the ions of heavy metals. Clays are generally deemed net negatively charged and attract positively charged cations within an adsorbed layer on the particle surfaces, the so called double layer. Rowe *et al.* (1997) suggest that the exchange may include the cations $K^+$, $Na^+$, $Pb^{2+}$, $Cd^{2+}$, $Fe^{2+}$, $Cu^{2+}$ etc which replace other cations in the adsorbed double layer such as $Ca^{2+}$, $Mg^{2+}$ etc. Other researchers (eg. Dearlove, 1995; Bright *et al.*, 1996; Mohamed and Yong, 1996) report the preferential sorption of certain cations and that an alkaline pH presents a favourable environment. Conversely, at low pH, soil particle surfaces can revert to a net positive charge and may not in this event attenuate positively charged heavy metal species. The pH of the system not only influences cation exchange but also the precipitation from solution of many metals as hydroxides and carbonates onto soil particle surfaces and into pore water (Bright *et al.*, 1996).

NWWRO (1996) suggests tests should be carried out to provide an understanding of how leachate will interact with the lining and propose determinations of Cation Exchange Capacity (CEC), Anion Exchange Capacity (AEC) and Partition Coefficient ($K_d$) as appropriate. The CEC and AEC are measures of the abundance of exchangeable ions required to be adsorbed onto the clay platelets to render them neutral, while $K_d$ may be viewed as a global measure of the sorption and precipitation attenuation potential of a clay. $K_d$ may be incorporated within a linear model to predict pollutant attenuation, where,

$$S = K_d\, c \tag{3}$$

where, S = mass of solute removed from solution per unit mass of liquid
c = equilibrium concentration of solute in pore fluid

Other more complex partitioning relationships have been proposed but seem unwarranted for general design purposes. $K_d$ may be determined from leachate column tests (eg. Rowe, 1994) or batch tests (ASTM, 1979; Griffin *et al.*, 1986, DOE, 1995b) which should ideally be carried out using a leachate of known properties or for a range of individual potential contaminants possibly with different concentrations. At the present time such tests are generally deemed research tools rather than tests in everyday usage although where specialised wastes are proposed, or for particularly sensitive sites, measurements from such tests provide diffusion coefficients (Equation 2) and a measure of the partitioning effects of the clay. In practice, however, the leachate will not be available at the design stage, the composition of the leachate will vary appreciably with time and $K_d$ for a given contaminant is likely to be influenced by the presence of other contaminants which may preferentially be attracted to the soil particle surfaces. There is no agreement at this time on what might constitute an 'indicator' solution to yield a comparative measure of the attenuation properties of a clay. It is also important to appreciate that there can be a marked change in the behaviour of some pollutants if there is sufficient loading on the system to bring about changes in clay behaviour. For metal contaminants, $K_d$ estimates are highly variable and are sensitive to soil properties including pH, clay content, organic matter content, free iron and magnesium oxide contents, and particle size distribution. The values of $K_d$ for metals are thus often presented as a range. Dragun (1988) and Rowe *et al.* (1997) presents $K_d$ values for a number of inorganic contaminants.

For organic contaminants, the organic carbon (or octanol-carbon) partition coefficient ($K_{oc}$) is used as a measure of a soils adsorption potential. There is a close relationship between the adsorption of organic pollutants to soil organic matter and the amount measured by partitioning and almost all of the adsorption of organic chemicals by soil is governed by the organic carbon content of the soil (eg. Hines and Failey, 1997). $K_{oc}$ is the ratio of the amount of chemical adsorbed per unit weight of organic carbon to the chemical concentration in solution at equilibrium and is given by,

$$K_{oc} = K_d/f_{oc} \tag{4}$$

where, $f_{oc}$ = organic carbon content

It is far easier to calculate the partition coefficient for organics than the partition coefficient for metals as the former is largely independent of soil properties. $K_d$ and $K_{oc}$ for a range of organic contaminants are presented by Hattemer-Frey and Lau (1996) and Rowe *et al.* (1997). Clays derived from the Coal Measures, Oxford Clay and similar strata are likely to have relatively high organic carbon concentrations which would be conducive to the attenuation of organic contaminants in leachate. In a complete formulation of the risks associated with contaminant migration the influence of biological degradation of organic pollutants should also be taken into account (German Geotechnical Society, 1993). This process results in analytical difficulties as it varies throughout the life of the landfill and is dependent on the presence and survival of suitable micro-organisms.

Though clay barriers are capable of sorption of chemical species from leachate over considerable periods of time (Davies *et al.*, 1996), the interaction of leachate and clays presents a complex problem and even with detailed site specific testing the situation will be far from determinate. Yet the protection of the groundwater is of prime environmental concern and aquifer vulnerability is a key factor in any assessment of landfill development and design (eg. Foster, 1998).

## *Tests to assess the influence of leachate chemistry*

The influence of the leachate chemistry arguably produces the greatest uncertainty to long term performance of a clay barrier. A number of investigations have shown that certain organic chemicals can cause shrinkage of the diffuse double layer which affects the soil structure leading to aggregation or flocculation of the clay and increased permeability (Quigley and Fernandez, 1994; Dakin *et al.*, 1997). NWRRO (1996) and DOE (1996) suggest that the chemical impact of leachate on mineral liners may be assessed by carrying out testing in accordance with ETC 8 (German Geotechnical Society, 1993). In laboratory determinations of the influence of leachate chemistry on permeability, most research seems to have been carried out using artificial leachates with high chemical concentrations, well in excess of those normally encountered in landfill. Although there appears to be no unanimity of view at this time on the influence of organic contaminants on the permeability of clays, there seems to be a consensus that the influence is small at concentrations in the leachate from normal domestic waste (DOE, 1996; Brunelle *et al.*, 1987; Daniel and Liljestrad, 1984). Nevertheless, the influence of the leachate chemistry presents uncertainties which cannot be fully addressed without further research.

Of some comfort is the suggestion by Farquhar (1994) that liner permeability often decreases with time as a result of sealing due to precipitate formation, solids accumulation and biomas growth along the upper surface of a liner and within cracks and fissures. Dakin *et al.* (1997) appear to support this contention but also highlight the uncertain influences on permeability of aerobic and anaerobic environments.

## Correlations between physical and chemical tests

In soil mechanics terms, the greater the plasticity of a clay (the greater the LL and PI) the greater the quantity of clay particles and the higher their surface activity (defined as PI/clay content after Skempton, 1953). These mineralogical properties are closely related to the physio-chemical properties of cation exchange capacity (CEC) and specific surface area (SSA) (Yong *et al.*, 1997) where SSA is a measure of the surface area of soil particles per unit mass of solids (or sometimes defined as the surface area per unit volume of solids). Generalised relationships can thus be expected between plasticity, the chemistry of the clay particles and permeability. The more highly plastic the clay the greater the CEC and SSA and the less the permeability and the advection potential. Such relationships have been shown experimentally by a number of researchers including Lambe (1954), Mesri and Olson (1971) and Benson *et al.* (1994). Thus, clays comprising mainly kaolinite (as determined by X-ray defraction) are of relatively low plasticity, CEC and SSA, and generally have a greater permeability than clays comprising illite, which in turn have a greater permeability than clays comprising smectite (which includes montmorillonite and the generic mineral species bentonite) which exhibit the highest plasticity and a high CEC and SSA (Yong and Warkenin, 1975).

## CQA testing

There have been a number of reported instances of failures of compacted clay linings and Farquhar (1994) suggests these have usually resulted from inadequate design and installation procedures. Adequate Quality Control is essential to satisfactory performance and NWWRO (1996) indicates the scope of Quality Assurance testing normally required for each layer of a clay lining.

In the field a number of key factors influence permeability and hence the acceptability of a compacted clay liner during the construction stage (eg. Elsbury *et al.*, 1990). These factors need to be addressed in compaction trials which should be designed to simulate as close as possible proposed construction.

(i) Conditioning or preparation of the clay (eg. addition of water, screening)
(ii) Compliance with the design parameters
(iii) Destruction of clods and elimination of associated fissuring
(iv) Interlift bonding
(v) Lift thickness
(vi) Type and weight of roller, number of passes and coverage
(vii) Degree of compaction and saturation
(viii) Possible construction, sampling, testing and validation difficulties
(ix) Adequacy of the CQA Plan

The close control of clay moisture content is particularly important in lining construction and often presents difficulties. Wetting up, or less frequently dying out, is sometimes required so that the clay moisture content lies within the required range. When water is added it is necessary to ensure a relatively uniform distribution and this may be achieved by spraying dispersed water from a towed bowser possibly followed by hoeing, rotivating or other mechanical means to mix the materials. In practice it is necessary to leave the material to stand in an uncompacted condition, possibly overnight or longer, to allow added water to soak into the material before compaction. Drying is more difficult and should be avoided if possible, but may be achieved in dry weather by spreading in thin layers which are periodically turning to allow natural aeration. Subsequent to liner compaction, dry or windy periods may result in desiccation of the clay and periods of precipitation may result in wetting up of the materials. In either event preventative or corrective measures need to be adopted.

Tamping (pad foot) vibrating rollers are often recommended for compacting clay liners because of the greater moulding effect achieved. However, there are benefits in employing a combination of both tamping and smooth wheeled rollers: finishing off the upper surface with a smooth wheeled roller allows a better visual indication of variations in layer thickness; the presence of zones of fragmented or gravely material with a lack of clay become more apparent; and sealing the upper surface helps to protect the material and encourages run-off in times of inclement weather. The tamping roller may subsequently be used to achieve the necessary scarification of the layers between lifts.

Earthworks are generally controlled by moisture content and dry density determinations. Figure 3b shows the results of nuclear density meter (NDM) testing against a laboratory determined compaction curve for an intermediate plasticity clay. The NDM (see Figure 4) is in common use as a means of determining both dry density and moisture content in the field as it provides immediate results as opposed to core sampling and sand replacement density determinations. However, because of inaccuracies in NDM moisture content determinations (and thus in the conversion from bulk density to dry density) normal practice is to recover soil samples to check on field measurements. Nevertheless, it is essential to undertake very careful instrument calibration for individual soil types. The instrument should preferably be calibrated against accurately measured densities in the laboratory container method (B.S.1377:1990 Part 9, Clause 2.5.5.3.1) but more frequently it is calibrated against in-situ cores or sand replacement density determinations which in themselves may be subject to errors.

**Figure 4** Use of nuclear density meter

The MCV test provides an alternative rapid means of assessing a clays acceptability at source and proves particularly useful where the source clay shows a degree of variability. For clays of differing plasticity, the acceptable MCV's show far less variability between clays than do the moisture contents. Murray *et al.* (1992 and 1996) and Jones *et al.* (1993) describe the use of the MCV apparatus in establishing acceptance limits. Measurements of moisture content (or MCV) and density cannot, however, be taken as precluding the need for further permeability testing on the compacted lining material as an assurance that the control criteria are adequate. Figure 3a presents the results of laboratory permeability tests on core samples recovered from a clay lining (compacted in accordance with Figure 3b) confirming the acceptability of the construction.

Ideally, in-situ permeability testing as detailed in Table A1 should also be carried out but such testing suffers from the disadvantages that it is time consuming, does not reflect the true confining stress conditions imposed by the landfill and can result in prolonged expose of the lining to the elements. Consideration might be given to the use of carefully prepared trial areas, isolated from lining construction, where in-situ permeability tests could be carried out prior to or at an early stage of development. Such testing would provide valuable information on the larger scale performance of lining material.

## Conclusions

The design and construction of landfills requires a detailed appraisal of site and environmental conditions encompassing a broad appreciation of a number of engineering and scientific disciplines. In assessing the risks to the aquatic environment it is necessary to have an understanding of the mechanisms by which pollution associated with leachate can escape through clay linings and an appreciation of the limitations of lining systems and the construction difficulties encountered in practice. The physical migration of contaminants due to permeation through a clay lining is generally accepted as greater than predicted from laboratory tests and advection is usually deemed the main mechanism of contaminant escape. However, for low permeability barriers, or in the case of net inflow into a landfill, diffusion cannot be ignored in assessing the influence of landfill on the aquatic environment. The current state of knowledge allows analysis of the mechanism of contaminant movement but the significance of such analysis is greatly restricted by the uncertainties inherent in testing and actual field performance. Analysis at this time is probably best limited to sensitivity studies, in particular, to determining the significance of variability in test parameters and design features.

In selecting and approving clays for landfill sites, the author finds the definitions 'material suitability' and 'acceptability of materials' useful in the distinction between source material for possible use as a liner and those materials approved, conditioned and compacted in the lining construction during a staged approval of landfill lining material. The acceptability testing may be further sub-divided into Physical, Chemical and CQA testing.

Careful drafting of a testing regime and monitoring of the earthworks operations is essential in endeavouring to ensure that the clay selected and emplaced satisfies the design requirements. It is stressed that the purpose of the physical and chemical design testing is to gain an appreciation of the overall performance of the landfill in relation to the surrounding environment, and to this end far more information is required on the actual performance of landfills in relation to that predicted at the outset of landfill development. CQA testing allows validation of a clay lining to the satisfaction of the regulatory authority and provides a check that the design parameters are achieved

With the inherent desire of the Environment Agency to improve standards and provide pressure on the industry to develop a better understanding of the potential influence of landfill on the environment, more detailed and increasingly intricate testing and analysis will be required. This must, however, be tempered with the recognition that a landfill presents a complex thermodynamic system not readily amenable to analysis, though the principles of thermodynamics are appropriate as they apply equally to solids, liquids and gases. Such principles as the conservation of energy apply to the chemical reactions within the landfill, the heat and gases generated, the stresses and volume changes, permeation and the advection and diffusion of contaminants. However, detailed analysis of a landfill as a thermodynamic system, where the energy levels within are balanced with the energy exchanges with the surrounding environment, are beyond current capabilities.

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## References

ASTM (1979). Proposed Methods for the Leaching of Waste Materials. *Annual Book of ASTM Standards*, Part 31-Water, 1258-1261.

ASTM (1990). Standard test method for field measurement of infiltration rate using a double-ring infiltrometer with a sealed inner ring. *ASTM D5093.*

Avery. B.W. and Boscomb, C.L. (1974). *Soil survey laboratory methods*. Soil Survey of England and Wales.

Barden, L., Madedor, A.O. and Sides, G.R. (1969). The flow of air and water in partly saturated clay soils. *Proc. 1st Int. Symp. on the Fundamentals of Transport Phenomena in Porous Media*, Elsevier, New York.

Benson, C..H. and Daniel, D.E. (1990). Influence of clods on hydraulic conductivity of compacted clay. *Jnl. of Geotech. Engrg*, ASTM, 111, 1231-1248.

Benson, C.H., Zhai, H. and Wang, X. (1994). Estimating hydraulic conductivity of compacted clay liners. *Jnl of Geotech. Engrg*, 21(2), 366-387.

Bright, M.I., Thornton, S.F., Lerner, D.N. and Tellam. J.H. (1996). Laboratory investigations into designed high attenuation landfill liners. *Engineering Geology of Waste Disposal*, Geological Society

Engineering Geology Special Publication No.11, Ed. Bentley, S.P., 159-164.

B.S.1377: 1990. *Methods of Tests for Soils for Civil Engineering Purposes*, Code of Practice. British Standards Institution, HMSO, London.

B.S.5930:1981. *Code of Practice for Site Investigation.* British Standards Institution, HMSO, London.

Brunelle, T.M., Dell, L.R. and Meyer, C.J. (1987). Effect of permeameter and leachate on a clay liner. *Geotechnical Practice for Waste Disposal '87*, GSP 13, Ed. Woods, R., ASCE, 347-359.

Dakin, M.G., Wright, S.P., Williams, J.B., Langdon, N.J., Sangha, C.M. and Walden, P.J. (1997). Physio-chemical and microbial factors effecting the passage of leachate through clay liners. *Geoenvironmental Engineering, Contaminated Ground: Fate of Pollutants and Remediation*, Proc. Conf., University of Wales, Cardiff, Ed. Yong, R.N. and Thomas, H.R., 68-73.

Daniel, D.E. and Liljestrand, H.M. (1984). Effect of landfill leachates on natural liner systems. *Geotechnical Engineering Report, GR 83-6*, Geotechnical Engineering Center, University of Austin, Texas.

Daniel, D.E. (1993). Geotechnical practice for waste disposal. Chapman and Hall, London.

Davies, M.C.R., Railton, L.M.R. and Williams K.P.(1996). A model for adsorption of organic species by clays and commercial landfill barrier materials. *Engineering Geology of Waste Disposal*, Geological Society Engineering Geology Special Publication No.11, Ed. Bentley, S.P., 273-78.

Dearlove, J.P.L. (1995). Geotechnical interaction processes between landfill clay liner materials and organo-metals in landfill leachate. *Waste Disposal by Landfill*, Proc. Symp. Green '93 Geotechnics Related to the Environment, Ed. Sarsby, R.W., 409-414.

DOE (1994). *Waste Management Licensing.* HMSO, London.

DOE (1995a). *Waste Management Paper 26B - Landfill Design*, Construction and Operational Practice. HMSO, London.

DOE (1995b). The technical aspects of controlled waste management - investigation of alternative materials for landfill liners. *Report No. CWM/116/94* by Cousens, T.W. and Studds, P.G.

DOE (1996). Guidance on good practice for landfill engineering - *Report No. CWM 106/94C.* The technical aspects of controlled waste management - research report.

Department of Transport (1991). *Specification for Highway Works.* HMSO, London.

Dragun, J. (1988). The fate of hazardous materials in soil (what every geologist and hydrogeologist should know), Part 2. *Hazard. Mater. Control.*, 1(3), 41-65.

Elsbury, B.R., Daniel, D.E., Sraders, G.A. and Anderson, D.C. (1990). Lessons learned from compacted clay liners. *Jnl. Geotech. Engrg.*, ASCE, 116(11), 1641-1660.

Farquhar, G.J. (1994). Experiences with liners using natural materials. *Landfilling of Waste: Barriers*, Ed. Christensen, T.H., Cossu, R. and Stegmann, R.E. Publ. F.N.Spon. 37-53.

Foster, S.S.D. (1998). Groundwater recharge and pollution vulnerability of British aquifers: a critical overview. Geological Society Special Publication No.130, *Groundwater Pollution, Aquifer Recharge and Vulnerability*, Ed. Robins, N.S., The Geological Society, London, 7-22.

German Geotechnical Society for the international Society of Soil Mechanics and Foundation Engineering (1993). *Geotechnics of Landfill Design and Remediation Works Technical Recommendations* - GLR, Ernst and Sohn, Berlin.

Gordon, M.E. (1987). Design and performance monitoring of clay-lined landfills. *Geotechnical Practice for Waste Disposal '87*. Ed. Woods, R.D., Geotechnical Special Publication No. 13, ASCE, 500-514.

Griffin, R.A., Sack, W.A., Roy, W.R., Ainsworth, C.C. and Krapac, I.G. (1986). *Hazardous and industrial Waste Testing*, ASTM STP 933, 390-408.

Hattemer-Frey, H.A. and Lau, V. (1996). Chapter 11, site-specific considerations in risk assessment. *Environmental Impact of Chemicals: Assessment and Control*, Ed. Quint, M.D., Taylor, D. and Purchase, R., The Royal Society of Chemistry, 120-146.

Head, K.H. (1981). *Manual of soil laboratory testing*, Vol. 2, permeability, shear strength and compressibility tests. Pentech Press.

Head, K.H. (1986). *Manual of soil laboratory testing*, Vol.3, effective stress tests. Pentech Press.

Hines, M. and Failey, R. (1997). Understanding pollutant behaviour to achieve best practice in site remediation. *Geoenvironmental Engineering, Contaminated Ground: Fate of Pollutants and Remediation*, Proc. Conf., University of Wales, Cardiff, Ed. Yong, R.N. and Thomas, H.R., 74-78.

Jessberger, H.L. (1994). Geotechnical design and quality control of mineral liner systems. *Landfilling of Waste: Barriers*, Ed. Christensen, T.H., Cossu, R. and Stegmann, R.E. Publ. F.N.Spon. 55-68.

Jones, R.M., Murray, E.J. and Rix, D.W. (1993). Selection of clays for use as landfill liners. *Waste disposal by Landfill*, Proc Symp. Green '93, Bolton Institute of Higher Education. Ed. Sarsby, R.W., 433-438.

Lambe, T.W. (1995). The permeability of compacted fine grained soils. *ASTM STP 163*.

Mesri, G. and Olson, R.E. (1971). Mechanisms controlling the permeability of clays. *Clays and Clay Minerals*, Pergamon Press, 19, 151-158.

Mitchell, J.K., Hooper, D.R. and Campanella, R.G. (1965). Permeability of compacted clays. *Jnl. Geotech. Engrg*, ASCE, 91 (SM4), 41-65.

Mitchell, J.K. and Younger, J.S. (1967). Abnormalities in hydraulic flow through fine grained soils. *ASTM STP 417*, 106-139.

Mohamed, A.M.O and Yong, R.N. (1996). Diffusion of contaminants through a clay barrier under acidic conditions. *Engineering Geology of Waste Disposal*, Geological Society Engineering Geology Special Publication No.11, Ed. Bentley, S.P., 279-289.

Murray, E.J., Jones, R.H. and Rix, D.W. (1997). Relative importance of factors influencing the permeability of clay soils. *Geoenvironmental Engineering, Contaminated Ground: Fate of Pollutants and Remediation*, Proc. Conf., University of Wales, Cardiff, Ed. Yong, R.N. and Thomas, H.R., 229-239.

Murray, E.J., Rix, D.W. and Humphrey, R.D. (1992). Clay linings to landfill sites. *QJEG*, 25, 371-376.

Murray, E.J., Rix, D.W. and Humphrey, R.D. (1996). Evaluation of clays as linings to landfill. *Engineering Geology of Waste Disposal*, Geological Society Engineering Geology Special Publication No.11, Ed. Bentley, S.P., 251-258.

Murray, E.J. (1998). Discussion on 'Observations on soil permeability, moulding moisture content and dry density relationships' by Wright, S.P., Walden, P.J., Sangha, C.M. and Langdon, N.J. *QJEG*, 31, Part 1, 73-74.

NWWRO (1996). Earthworks on landfill sites - a technical note on the design, construction and quality assurance of compacted clay liners. *Document No. NWTECH.002.*

NRA (1992). *Policy and practice for the protection of groundwaters.* National Rivers Authority, Bristol.

Parkinson, C.D. (1991). The permeability of landfill liners. *The Planning and Engineering of Landfills*, Conf. Proc. University of Birmingham, 147 151.

Parsons, A.W. and Boden, J.B. (1979). The moisture condition test and its potential application in earthworks. *Transport and Road Research Laboratory Supplementary Report 522.*

Quigley, R.M. and Fernandez, F. (1994). Effect of organic liquids on the hydraulic conductivity of natural clays. *Landfilling of Waste: Barriers*, Ed. Christensen, T.H., Cossu, R. and Stegmann, R.E. Publ. F.N.Spon. 203-18.

Rowe, R.K. (1994). Diffusion transport of pollutants through clay liners. *Landfilling of Waste: Barriers*, Ed. Christensen, T.H., Cossu, R. and Stegmann, R.E. Publ. F.N.Spon. 219-245.

Rowe, R.K., Quigley, R.M. and Booker, J.R. (1997). *Clayey barrier systems for waste disposal facilities.* Publ. E. and F.N. Spon.

Skempton, A. (1953). The colloidal activity of clay. *Proc 3rd Int. Conf. Soil Mech and Found. Engng.*, London, England: Butterworths Scientific Publications, Vol.1, 57-61.

Shakoor, A. and Cook, B. (1990). The effect of stone content, size and shape on the engineering properties of a compacted silty clay. *Bull. Assoc. Engrg. Geologists*, XXVII(2), 245-253.

Shelley, T. and Daniel, D. (1993). Effect of gravel on hydraulic conductivity of compacted soil liners. *Jnl. Geotech. Engrg.*, ASCE, 119(1), 54-68.

Williams, C.E. (1987). Containment applications for earthen liners. In *Proceedings of the 1987 Speciality Conference on Environmental Engineering*. Ed. Dietz, J.D., ASCE, 122-128.

Yong, R.N., Tan, B.K., Bentley, S.P., Thomas, H.R., Williams, K.P., Pooley, F.D. and Zuhairi, W. (1997). Attenuation characteristics of natural clay materials in South Wales and their use as landfill liners. *Geoenvironmental Engineering, Contaminated Ground: Fate of Pollutants and Remediation*, Proc. Conf., University of Wales, Cardiff, Ed. Yong, R.N. and Thomas, H.R., 161-169.

Yong, R.N. and Warkenin, B.P. (1975). *Soil properties and behaviour.* 2nd ed., Elsevier Scientific.

# Table A1: Testing of clay liners

## Suitability testing

Plastic Limit (BS1377:1990 Part 2 Method 5.3)
Liquid Limit: Four Point Method (BS1377:1990 Part 2 Method 4.3)
Single Point Method (BS1377:1990 Part 2 Method 4.4)
Plasticity Index (BS1377:1990 Part 2 Method 5.4)
Particle Size Distribution: Wet Sieve (BS1377:1990 Part 2 Method 9.2)
Dry Sieve (BS1377:1990 Part 2 Method 9.3)
Pipette Method (BS1377:1990 Part 2 Method 9.4)
Hydrometer Method (BS1377:1990 Part 2 Method 9.5)

## Acceptability testing

### Physical design tests

#### *Tests for construction control*

Compaction Series: Light Hammer (2.5kg) rammer (BS1377:1990 Part 4 Method 3.3)
Heavy Hammer (4.5kg) rammer (BS1377:1990 Part 4 Method 3.5)
MCV Compaction (BS1377:1990 Part 4 Method 5.5)
Particle Density (BS1377:1990 Part 2 Method 8)
Moisture Content (BS1377:1990 Part 2 Method 3.2)

#### *Tests to assess permeability and advection properties*

Permeability by water on laboratory prepared samples (leachate tests may also be carried out in general accordance with the following. Other test methods are available. * indicates those tests in more common usage)

Triaxial Constant Head (BS1377:1990 Part 6 Method 6)*
Hydraulic Consolidation Cell Constant Head (BS1377:1990 Part 6 Method 4)
Triaxial Constant and Falling Head (Head 1986 Tests 20.4.1 to 20.4.4)
Falling Head Permeameter (Head 1981 Test 10.7.2)*
Falling Head Test in Sample Tube (Head 1981 Test 10.7.3)
Falling Head Test in Oedometer Cell (Head 1981 Test 10.7.4)
Falling Head Test in Rowe Consolidation Cell (Horizontal and Vertical Permeability) (Head 1986 Test 24.7.2 and 27.7.3)

### *Tests to mitigate physical damage*

Shear Strength (on recompacted samples):
- Hand Shear Vane (BS1377:1990 Part 7 Method 3)
- Undrained Triaxial Strength (BS1377:1990 Part 7 Method 8)
- Shear Box: Small (BS1377:1990 Part 7 Method 4)
  Large (BS1377:1990 Part 7 Method 5)
- Consolidated Undrained Effective Triaxial Stress
- Consolidated Drained Effective Triaxial Stress

Dispersivity:
- Pinhole Test (BS1377 :1990 Part 5 Method 6.2)
- Crumb Test (BS1377:1990 Part 5 Method 6.3)
- Dispersion Test (BS1377:1990 Part 5 Method 6.4)
- Chemical Tests (Head 1981 Test 10.8.5)

Linear Shrinkage (BS1377:1990 Part 2 Method 6.5)
Oedometer Consolidation Test (on recompacted samples) (BS1377:1990 Part 5 Method 3)

## Chemical Design Tests

### *Tests to assess diffusion and attenuation properties*

Batch Tests (ASTM 1979)
Leaching Column Tests (Rowe, 1994)
Cation Exchange Capacity
Anion Exchange Capacity
Organic Carbon Content (measured by $CO_2$ infra-red spectrometer)
Mass Loss on Ignition (BS1377:1990 Part 3 Method 4)
Carbonaceous Content by High Temperature Loss on Ignition (Avery and Boscomb, 1974)
Clay Mineralogy by X-Ray Defraction

### *Tests to assess the influence of leachate chemistry*

Tests in accordance with ETC 8 (German Geotechnical Society, 1993) to characterise the leachate and in accordance with other tests herein to assess the influence.

## CQA tests

### *Tests to check on material suitability*

Plastic Limit (BS1377:1990 Part 2 Method 5.3)
Liquid Limit:
- Four Point Method (BS1377:1990 Part 2 Method 4.3)
- Single Point Method (BS1377:1990 Part 2 Method 4.4)

Plasticity Index (BS1377:1990 Part 2 Method 5.4)
Particle Size Distribution:
- Wet Sieve (BS1377:1990 Part 2 Method 9.2)
- Dry Sieve (BS1377:1990 Part 2 Method 9.3)
- Pipette Method (BS1377:1990 Part 2 Method 9.4)
- Hydrometer (BS1377:1990 Part 2 Method 9.5)

## *Tests to check on material acceptability*

MCV (BS1377:1990 Part 4 Method 5.5)
Shear Strength (in-situ or on undisturbed samples)
- Hand Shear Vane (BS1377:1990 Part 7 Method 3)
- Undrained Triaxial Strength (BS1377:1990 Part 7 Method 8)
- Shear Box: Small (BS1377:1990 Part 7 Method 4)
  - Large (BS1377:1990 Part 7 Method 5)

Permeability by water on undisturbed samples (* indicates those tests in more common usage)
- Triaxial Constant Head (BS1377:1990 Part 6 Method 6)*
- Hydraulic Consolidation Cell Constant Head (BS1377:1990 Part 6 Method 4)
- Triaxial Constant and Falling Head (Head 1986 Tests 20.4.1 to 20.4.4)*
- Falling Head Permeameter (Head 1981 Test 10.7.2)*
- Falling Head Test in Sample Tube (Head 1981 Test 10.7.3)
- Falling Head Test in Oedometer Cell (Head 1981 Test 10.7.4)
- Falling Head Test in Rowe Consolidation Cell (Horizontal and Vertical Permeability) (Head 1986 Test 24.7.2 and 27.7.3)

Permeability (in-situ)
- Ponding Tests
- Ring Infiltrometer (eg ASTM D5093, 1990)
- Lysimeter

Density in situ or on undisturbed samples (* indicates tests in more common usage)
- Sand Replacement (BS1377:1990 Part 9 Method 2.1)*
- Cores (BS1377:1990 Part 9 Method 2.4)*
- Nuclear Density Measurements (BS1377:1990 Part 9 Method 2.5)*
- Rubber Balloon Method (ASTM Test D 2167)
- Immersion in Water (BS1377:1990 Part 2 Method 7.3)
- Water Displacement Method (BS1377:1990 Part 2 Methods 7.4

# Issues related to the use and specification of colliery spoil liners

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## Introduction

Engineered landfills require the design and construction of reliable liners capable of containing leachate, and a compacted mineral layer generally forms part of the lining system. There is a trend towards the use of composite liners comprising a geomembrane laid directly on top of a mineral layer.

Clays are the most suitable minerals for liners because, with good practice, low permeabilities (hydraulic conductivities) are readily achievable. They may also possess good contaminant absorption or attentuation properties. However, natural clays are not always available on site and other materials may need to be considered. These include bentonite modified soils and, in some parts of the UK, colliery spoils. The use of colliery spoil may be especially attractive where a plentiful local supply exists, or where a liner is being constructed as part of restoration works for abandoned collieries with associated coal by-products plants. In such circumstances, the constructive use of a waste material may yield significant economic and environmental benefits.

Although some colliery spoils have been used successfully, by comparison with clay liners, relatively little research has been conducted on colliery spoil liners and published experience is sparse. In this paper, following a background section on the engineering characteristics of colliery spoils, the findings of a research programme at the University of Sheffield are summarised and a practical approach to specification and quality control is discussed.

The paper is primarily focussed on the issue of permeability, notwithstanding the relevance of diffusion and attenuation processes. Several factors influencing permeability will be considered, as well as testing methods.

*Geotechnical engineering of landfills.* Thomas Telford, London, 1998

## Engineering characteristics of colliery spoil

Colliery spoil heaps may include both coarse rock discard, arising from the construction of underground access tunnels and galleries or opencast workings, and finer slurries and tailings separated from the coal in washery plants. Materials such as furnace ashes and a great variety of other solid and liquid wastes may also have been disposed of within a spoil tip. All the lithologies within the Coal Measures sequence are usually present within tips. Taylor and Spears (1970) report that in the East Midlands, for example, the sequence comprises the following lithologies: siltstones 40%, mudstone and shale 30%, seatearth 10-20%, sandstone 5-10% and coal 2-7%. In tips from underground workings there is liable to be a predominance of shales and seatearths and older tips usually contain a significant proportion of coal. Taylor (1984) indicates that the average organic carbon content (mostly coal) for English and Welsh tips is 13.3%, although in some old tips it may be as high as 47%. The variation in coal content, and also the variable presence of ironstone, explains a considerable variation of specific gravity (see Table 1 below).

The geotechnical properties of colliery spoils are very variable depending upon the lithologies present, the extent of weathering and, more particularly, the grading. The gradings of spoils span a large range as shown on Figure 1. Taylor (1984) notes that coarse discards range from silty sand to coarse gravel and cobbles, whereas fine discards range from clay to sandy medium gravel. The properties presented in Table 1 also show large ranges.

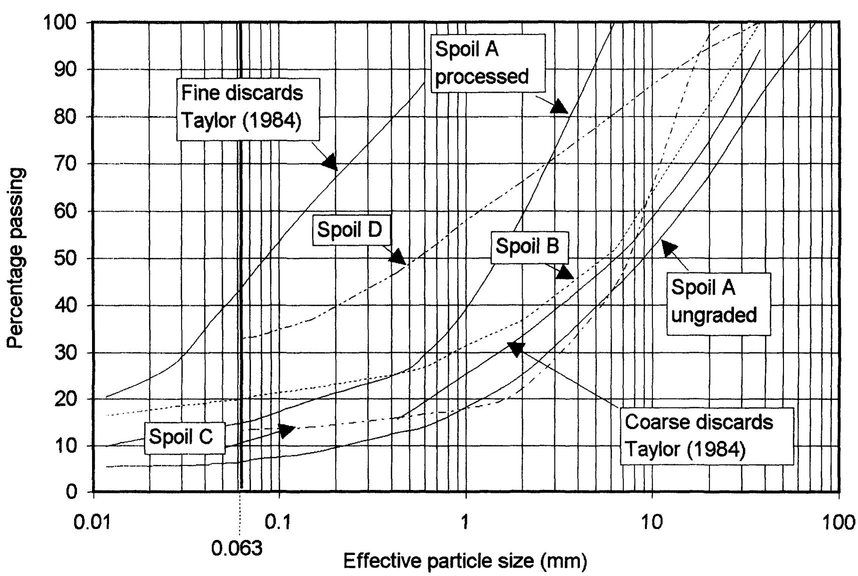


**Figure 1** Mean grading curves for UK colliery spoils

Because weathering action is a significant cause of variation in the geotechnical properties of colliery spoil, mainly through the resulting reductions in particle size, it is helpful to understand the processes involved. This is especially true in the present context, since permeability is strongly linked to particle size.

| Type of spoil | Liquid limit[2] (%) | Plastic limit[2] (%) | Specific gravity | Moisture content (%) | Optimum moisture content[3] (%) | Maximum dry density[3] ($Mg/m^3$) |
|---|---|---|---|---|---|---|
| Coarse | 36.8 (6.9) | 21.4 (4.0) | 2.31 (0.20) | 12.0 (6.0) | 13.2 *10.1* (4.2) *(3.2)* | 1.74 *1.81* (0.16) *(0.16)* |
| Fine | 38.8 (7.6) | 23.5 (4.5) | 1.94 (0.24) | 34.8 (12.9) | No data | No data |

Note 1: standard deviations are given in brackets.
Note 2: of material passing a 0.425mm sieve.
Note 3: for standard Proctor (2.5kg rammer) compaction in first column and modified Proctor (4.5kg rammer) compaction in second column, in italics.

**Table 1** Mean values of some geotechnical properties of UK spoils (from Taylor, 1984)

Although weathering effects in a tip might be expected to be most pronounced near its surface, tips may contain material that has been exposed to weathering for protracted periods of time before being subsequently tipped over or capped. Also, the upper parts of the sequence stripped in the course of opencasting operations are liable to have suffered some effects of weathering degradation in situ. The more vulnerable lithologies, such as the mudrocks, are liable to show evidence of surface derived weathering action down to depths of 7m or so (Taylor & Spears, 1972).

A regional variation of the geotechnical character of colliery spoil occurs in response to differences in the materials originally deposited and also because of differences in the depth of subsequent burial. Geothermal heating and increased pressure lead to increases in the density of the deposited materials, the precipitation of mineral cements in pore spaces and the conversion of swelling clay minerals, including in mudrocks any montmorillonite and mixed layer illite-smectite, to more stable illite and micas. Hence the rocks become stronger and more resistant to degradation. In parallel with these changes the vegetable material becomes transformed, as volatile components are driven off, into coal of successively higher rank.

Taylor (1988) notes that, although some coals in Scotland have low rank, the mudrocks contain more kaolinite and less unstable mixed layer clay than is found in the Yorkshire and Midlands coalfields. Mudrocks originating from North Derbyshire, Nottingham and the Western Region, which includes the South Staffordshire coalfield, are of lower strength than those from Yorkshire, Scotland, South Wales and North-East England.

Coal Measures rocks may be subject to degradation due to stress relief and other physical weathering processes but, as most of the mineral components were derived by weathering processes, they are quite stable chemically in present day weathering environments. However, certain minerals, of which pyrite ($FeS_2$) is the most important, are unstable in such environments. Pyrite occurs most commonly in coals and dark coloured shales and constitutes about 2% of fine discard (Taylor, 1984); traces, at least, are usually present in coarse spoil. Slow chemical oxidation can be considerably accelerated by the activities of bacteria, leading to the rapid removal of pyrite and the production of acid. The latter may attack carbonates, clay minerals and other components, and give rise to sulphate rich solutions. In engineering operations the possible generation of aggressive sulphate bearing solutions due to weathering of the material needs to be borne in mind. On the other hand, if low permeability is achieved, the processes will be limited by the rates at which reactants can be transported to and from reaction sites. Furthermore, it is reported by Taylor (1984) that 71% of the unburnt spoils he studied possessed water and acid soluble sulphate contents of less than 2.0g/l and 1% respectively.

It may be possible to encourage degradation of an unweathered spoil prior to placement in a liner in order to improve its performance. The rapid physical breakdown of Coal Measures rocks is favoured by the presence of swelling clay minerals, including mixed layer illite-smectite, and small-scale compositional laminations. The latter may include slight changes in the composition or texture of the rock. However, Czerewko (1997) has shown that these factors are not the sole controls on the breakdown of such rock. In some cases, degradation due to shrink-swell effects only occurs after the removal of cements by appropriate weathering action. Breakdown due to slaking action typically produces coarse sand or gravel sized particles which then undergo slower disintegration.

As mentioned above, the coal content of spoil can be appreciable. Depending on coal rank and the extent of weathering, humic and non-humic substances with a capacity for complexation with leachate components may be present. This, coupled with the capacity for cation exchange possessed by some clays, can render spoil capable of attenuating contaminants in leachates. This capability is liable to decrease in higher rank and less weathered spoils. In a study on spoils from Yorkshire, Cousens & Studds (1996) observed rather limited attenuation of various cations commonly found in spoil but $Cl^-$, $Na^+$ and $SO_4^{2-}$ were leached from the material.

## Testing methods for permeability

### Laboratory versus field testing

Most permeability testing on mineral liner materials, certainly in the UK, is conducted in the laboratory using rigid or flexible-walled permeameters. These test methods are reviewed by Daniel (1994). Laboratory tests can be well controlled and, in general, are relatively easy to interpret. Unfortunately, the results may not provide a reliable measure of field permeability for the following reasons:

(1) a nominally undisturbed specimen taken from the field for testing in the laboratory may be disturbed or damaged by the sampling process.

(2) an undisturbed specimen tested in the laboratory may be too small to represent the fabric (i.e. pore structure and hydraulic defects) of the soil in situ.

(3) a laboratory compacted, or recompacted, specimen may have a significantly different fabric (and perhaps grading) from that of the soil in situ.

(4) artificially high confining pressures and hydraulic gradients are often used in laboratory tests to reduce test durations.

Experience with clays (e.g. Daniel, 1984; Elsbury *et al.,* 1990) has shown that, for one or more of these reasons, the field permeabilities are likely to be higher than the laboratory values, and the differences can be very substantial. Evidence is presented below to show that similar differences are potentially observable for colliery spoil. Therefore, field testing should be contemplated at some stage in the verification process for mineral liner performance. This is well recognised, for example, in the USA where field testing of a test pad, constructed in the same manner as the actual liner, is a regulatory requirement (Shackleford, 1994).

Generally, field tests are harder to control and more difficult to interpret than laboratory tests. The available methods are reviewed by Sai & Anderson (1990) and Trautwein & Boutwell (1994). Some, though not all, of the field tests overcome the above limitations of laboratory tests. The single most important factor influencing reliability is the scale of the test; in order to obtain representative measurements, a sufficiently large volume of soil must be tested. The sealed double ring infiltrometer (SDRI) has gained wide acceptance in the USA, and SDRI tests on colliery spoil will be described below.

Large scale field tests, such as the SDRI, are time consuming and ill-suited for use as a quality control tool during the construction stage. In most cases, during construction a direct assessment of permeability must rely on laboratory testing, with all its potential errors. Experience with clay liners (e.g. Trautwein & Williams, 1990; Benson *et al.,* 1994) suggests that, if the liner is well constructed, small (say 100mm diameter) undisturbed specimens could give reliable results but, for a poorly constructed liner, undisturbed block specimens of about 300mm diameter would be required. Some upward adjustment of these sizes might be required for colliery spoil, depending on its maximum particle size. On the basis of the above experience, which is quite extensive, the use of the smaller specimens can only be defended if there is other evidence of good construction (e.g. consistently high dry density and degree of saturation). Ideally, a correction factor for the results of small scale laboratory tests should be established from comparisons with larger scale tests.

## Comparison of testing methods

Some research into the effect of testing method on the measured permeability of colliery spoil has been carried out at the University of Sheffield. The spoil tested was from a deep anthracite mine in South Wales and had been stored for over 10 years in a tip. However, the spoil was relatively resistant to weathering and the fines (silt plus clay) content had remained very low at 4-11%. The mean grading is indicated by the curve labelled "Spoil A- ungraded" in Figure 1.

Two test pads were constructed, one from ungraded material (with some particles in excess of 50mm) and one from material which had been crushed and graded to less than 6mm, so that its fines content was increased to 11-20%. The mean grading curve of this material is labelled "Spoil A - processed" in Figure 1. Compaction data for each test pad are summarised in Table 2. Test Pad 1 was formed by compacting the ungraded material at its natural water content in six 150mm lifts using a smooth vibrating roller. This produced fairly good results. Test Pad 2 was formed by spreading the crushed and graded material, mixing it in situ with water to bring the moisture content to slightly wet of optimum, and compacting it in a single 150-200mm lift on top of a base of coarser compacted spoil. A smooth vibrating roller was again used but in this case the compaction was relatively poor and also more variable.

| Test pad | Water content (%) | Dry density ($Mg/m^3$) | Maximum dry density[1] ($Mg/m^3$) | Optimum moisture content[1] (%) | Relative compaction (%) | Degree of saturation (%) | Air voids (%) |
|---|---|---|---|---|---|---|---|
| 1 | 5.5-7.5 | 2.13-2.21 | 2.22 | 6.0 | 96-100 | 75-94 | 1-5 |
| 2 | 6.0-8.0 | 1.69-2.06 | 2.14 | 6.0 | 79-96 | 34-68 | 7-23 |

Note 1: for modified Procter (4.5kg rammer) compaction.

**Table 2** Summary of compaction data for test pads

On each test pad a set of three SDRI tests was carried out in accordance with ASTM D5093 (1990). The infiltrometers were of a circular shape with inner ring diameters of 0.5m, 1.0m and 1.5m. The tests lasted for 4 to 8 weeks and employed a hydraulic head, applied at the ground surface, of between 0.21m and 0.34m. Unfortunately, because of either the coarse particles present in the spoil (Test Pad 1) or its limited depth (Test Pad 2), it was not practicable to install tensiometers to monitor the depth of infiltration. From each test pad undisturbed block samples were extracted and tested in large flexible-walled permeameters. The 300mm diameter by 200mm high blocks were carefully trimmed down to 250mm diameter to form the test specimens. Low effective confining pressures of about 20kPa were applied and back pressures were used to saturate the specimens. The hydraulic gradient was initially set to 11 but later it was changed to 16 and 21, and then back to 11. Finally, in some tests the direction of flow was reversed. Full details of the experimental techniques are given by Norton (1998).

In addition to the these tests, small scale permeability tests were carried out by commercial laboratories on 100mm diameter specimens in flexible-walled permeameters. Disturbed samples were taken from Test Pad 1, graded to 20mm, recompacted and tested using a hydraulic gradient of about 90 and an effective confining pressure of about 60kPa. It is likely that the fines content of these test specimens was increased as a result of the recompaction. Tube samples were taken from Test Pad 2 and tested using a hydraulic gradient of about 110 and an effective confining pressure of about 190kPa. Back pressures were employed to ensure saturation of all the specimens.

The results of these various tests were first presented by Hird *et al.* (1997a) and are summarised here in Figure 2. The test data were reprocessed and reinterpreted by Norton (1998) but, although some of the permeability values differ from those given previously, the numerical differences do not affect the overall research findings. For the SDRIs, the permeability was determined using the virtually constant rate of infiltration achieved after a period of 54 or 14 days for Test Pads 1 and 2 respectively. It was assumed that the flow had then penetrated the whole thickness of Test Pad 1 or the single final lift of Test Pad 2. This was consistent with the transit times of flow in laboratory infiltration tests which were also carried out (Hird *et al.,* 1997a); on Test Pad 2 flow was actually observed emerging from the base of the pad. When calculating hydraulic gradients in order to apply Darcy's law, the influence of suction was therefore ignored. In the large scale permeameter tests, the permeability was usually observed to decrease markedly (by an order of magnitude or more) in the early stages of the test. This was attributed (Hird *et al.,* 1997b) to particle migration and clogging of pores within the specimen and is discussed further below. The permeability values shown in Figure 2 were the steady values attained under the initial hydraulic gradient of 11 after a period of about 10 days.

The results of Figure 2 display some unexpected as well as some expected features. Considering first the SDRI results alone, on both test pads the smallest SDRI gave a distinctly higher permeability than either of the two larger ones. This is against the expectation that the measured permeability increases with the volume of soil tested and was attributed to the effects of installation. Excavation of a circular trench in the spoil to receive the inner ring, achieved using hand tools, was thought to have led to a peripheral disturbed zone. Cracks in this zone could have remained open even after cement bentonite grout was poured into the trench to seal the inner ring to the spoil and could therefore have provided preferential flow paths. The effect of such disturbance would have increased as the size of the SDRI decreased and is considered to have invalidated the results from the smallest SDRI. The similarity of the results from the two larger SDRIs suggests that the disturbance was relatively unimportant for inner rings of 1.0m diameter or larger.

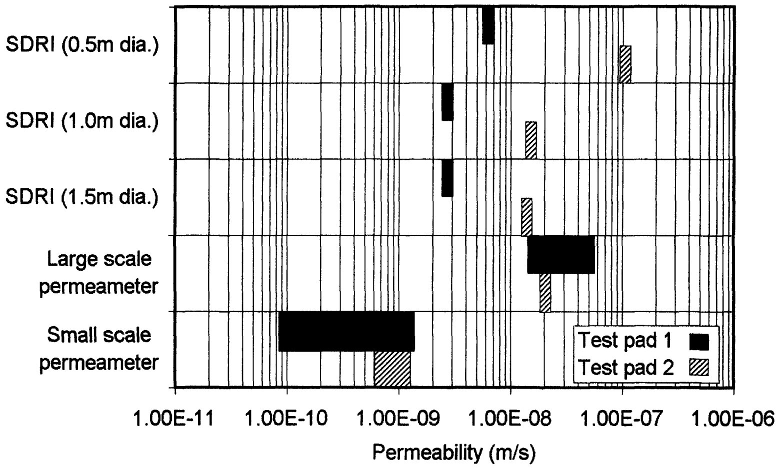


**Figure 2** Summary of permeability tests (after Norton, 1998)

On Test Pad 1 the large scale laboratory permeameter tests yielded values (4 results shown as a range in Figure 2) well above those from the two larger SDRI tests. Clearly, this goes against the generally expected trend of laboratory values being lower than field values. When the test pad was dissected beneath the SDRI test locations it was found that, except in the uppermost 250mm or so, no significant increase in average moisture content had occurred. Certainly, no distinct wetting front could be defined. However, isolated pockets of moisture arising from seepage were seen throughout the thickness of the pad and, taken together with evidence from laboratory infiltration tests referred to above, this suggested that flow had taken place to the base of the pad via preferential pathways, leaving most of the material in its original unsaturated condition. In contrast, in the laboratory tests the spoil was fully saturated. The difference in saturation was considered to be mainly responsible for the discrepancy between the SDRI and large scale permeameter results.

On Test Pad 2 there was much better agreement between the two larger SDRI tests and large scale permeameter tests carried out on blocks with representative dry densities (2 results shown as a range in Figure 2). However, in this case the relatively thin spoil layer being tested became much wetter beneath the SDRI test locations and, at the time of the permeability measurement, was probably approaching full saturation, although accurate measurements of moisture content proved difficult to make. The close agreement between the above test types is therefore understandable and also demonstrates that the scale of the laboratory tests was sufficient. Two other large scale permeameter tests were carried out on blocks which appeared to have unrepresentatively high dry densities and these measured significantly lower permeabilities (Hird *et al.,* 1997a).

Finally, as expected, for each test pad the large scale permeameter tests gave higher permeabilities than the small scale permeameter tests (ranges of 21 and 3 results shown in Figure 2 for Test Pads 1 and 2 respectively), the difference being over an order of magnitude. The use of higher confining pressures and hydraulic gradients in the small scale tests would have contributed to this; for Test Pad 1 an additional contributory factor would have been the use of laboratory recompacted material with a different grading (20mm down, more fines) in the small scale tests. In both cases there may have been a large scale fabric which was present in the 250mm diameter specimens but which was not represented in the 100mm diameter specimens. It should be noted that the SDRI tests on both test pads also gave significantly higher permeabilities than the small scale permeameter tests, despite the lack of saturation on Test Pad 1.

As already mentioned, there was a substantial reduction of permeability in the early stages of the large scale permeameter tests. Subsequently, increases of permeability occurred when the hydraulic gradient was increased or when the flow direction was reversed. Sometimes the changes were temporary or reversible, but not always, and hence the behaviour was complex. Kenney & Lau (1985) proposed a criterion based on grading for the internal stability of granular (or cohesionless) soils. While the spoils from the test pads would not be described as cohesionless, their fines content was low and it is interesting that they classified as unstable according to the Kenney and Lau criterion (Hird *et al.,* 1997b). It was therefore concluded that internal particle migration was responsible for the above changes and that such effects may complicate the interpretation of permeability tests conducted on similar colliery spoil. On the basis of limited evidence (Norton, 1998), it appears that a modest increase in the fines content can promote stability.

## Parameters influencing permeability

It is generally accepted that major factors influencing the permeability of compacted mineral liners are: grading, compaction moisture content, compactive effort and confining stress (e.g. Daniel, 1984 and 1993; Elsbury *et al.,* 1990; Mitchell *et al.,* 1965). Studies of these factors for colliery spoil permeability have been carried out at the University of Sheffield, as reported by Smith *et al.* (1997) and Norton (1998).

The colliery spoil investigated by Smith *et al.* (1997) originated from an opencast mine in South Wales. The site was mined during the 1970's and back filled in 1980, leaving the surface material to weather for over a decade. The spoil was initially sieved to remove particles greater than 50mm, resulting in a mean fines content of 20%, as indicated by the curve labelled "Spoil B" in Figure 1.

Parametric studies were carried out on specimens of this spoil compacted in a CBR mould and subsequently tested in a triaxial cell with back pressure saturation and a hydraulic gradient of 16. Each parametric study was conducted using material quartered and riffled from a bulk sample. Material consistency may be assumed within each study but not necessarily across studies. Specimens with a maximum particle size of up to 50mm were prepared and tested, notwithstanding the usual limit of 37.5mm for compaction in a CBR mould (BS 1377, 1990).

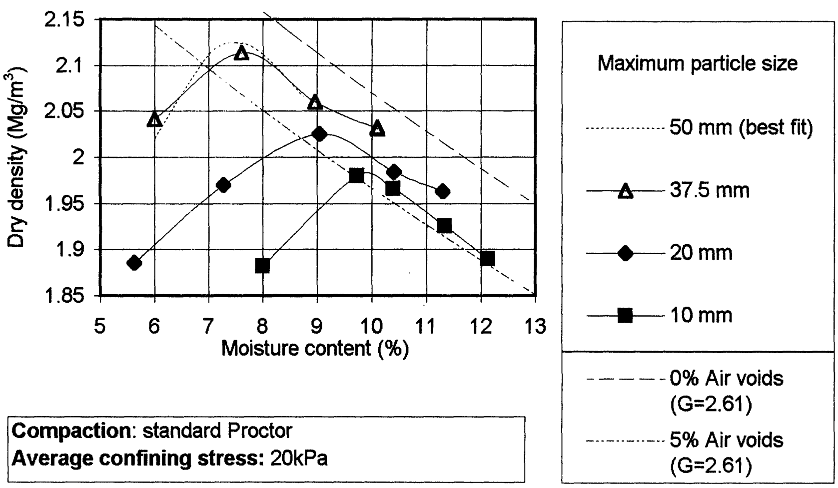


**Figure 3** Compaction results for spoil with various maximum particle sizes

## Effect of grading

The effect of adjusting maximum particle size only was investigated. The compaction curves for four different particle size ranges are given in Figure 3. The evident increase in dry density and reduction in air voids at optimum moisture content with increasing maximum particle size indicates that the large particles are replacing clusters of smaller particles without particle interference, which would otherwise generate additional voids. This is reflected again in the permeability test results, shown in Figure 4, which indicate that the maximum particle size has an insignificant effect on the measured permeabilities. The same conclusion was reached independently by Norton (1998). Data presented in the current paper for the various spoils whose gradings are shown in Figure 1 indicate that, not surprisingly, the fines content strongly controls the overall permeability of the material. The coarser material can be expected to pack efficiently regardless of its maximum particle size as long as the spoil is well mixed and well graded.

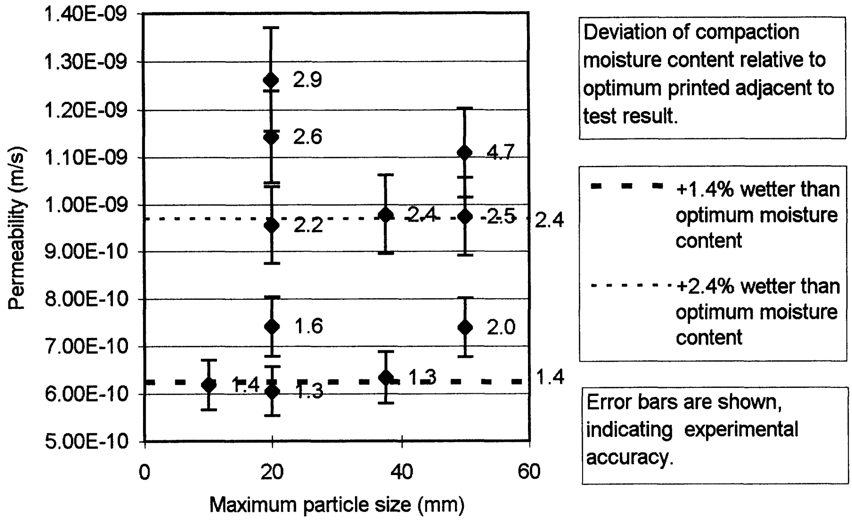


**Figure 4** Relationship between permeability and maximum particle size

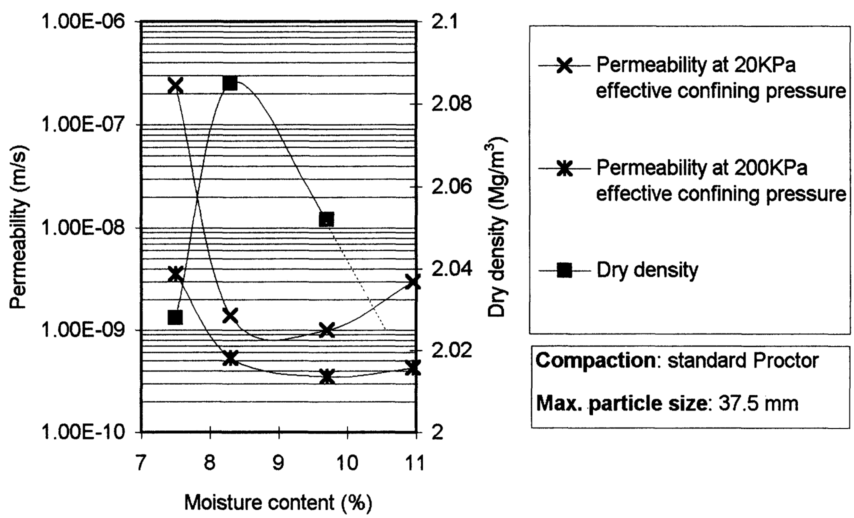


**Figure 5** Variation of permeability with compaction moisture content

## Effect of compaction moisture content and compactive effort

The variations of permeability with compaction moisture content at two different (average) consolidation pressures are presented in Figure 5. These results, and similar ones obtained by Norton (1998), indicate that the lowest permeability is achieved at moisture contents 0-1.5 % wetter than optimum. If these data are compared with data for clays (e.g. Mitchell *et al.*, 1965), the trends are similar but the permeability of spoil appears to be less sensitive to moisture content changes. Despite this, as the compaction moisture content falls below optimum, large increases in permeability still occur in compacted spoil.

Smith *et al.* (1997) report laboratory studies into the effect of different compactive efforts. As would be anticipated, maximum dry density increases and optimum moisture content reduces with increasing compactive effort. There is limited evidence, requiring further experimental support, that there is a modest reduction in permeability with an increase in compactive effort for comparable moisture conditions wet of optimum (that is at a given deviation of moisture content from optimum).

## Effect of confining stress

Consolidation of compacted colliery spoil under a confining stress will result in a decrease in permeability. Figure 6 indicates significant reductions in permeability with increasing confining stress, particularly at the lower stress levels. The range of pressures, 20kPa to 200kPa, might be considered representative of the overburdens experienced by a typical liner at the start and finish of landfilling. Permeability data for several clays given by Trast and Benson (1995) display a similar degree of sensitivity to confining stress.

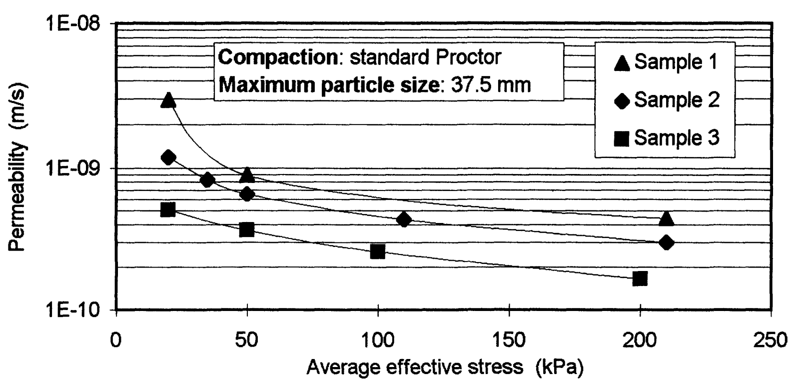


**Figure 6** Effect of confining stress on permeability

## Approach to specification and quality control

A general framework for the specification of compacted clay liners has been established involving consideration of shear strength, desiccation shrinkage and permeability (e.g. Daniel, 1993). Acceptable combinations of dry density and compaction moisture content satisfying all three criteria must be identified. This framework can equally well be applied to colliery spoils, although colliery spoils are less susceptible to shrinkage than most clays and the strength and permeability criteria are likely to be dominant. Attention here is focussed solely on permeability.

The accepted compaction target for achieving the lowest permeability of a given material is the zone lying roughly between the line of optimum moisture contents for different compactive efforts and the zero air voids line on the diagram of dry density versus compaction moisture content (referred to below as the "compaction diagram"). As shown in Figure 5, it is advantageous to compact colliery spoil at or a little above the optimum moisture content. At the design stage a combination of laboratory compaction and permeability tests can be used to define the limits of this zone that will satisfy the design requirement (usually that the permeability is less than $10^{-9}$m/s). Assuming that sufficiently low permeabilities can be achieved under favourable conditions, the limits will be dictated by the increases of permeability at compaction moisture contents dry of optimum and at low dry densities.

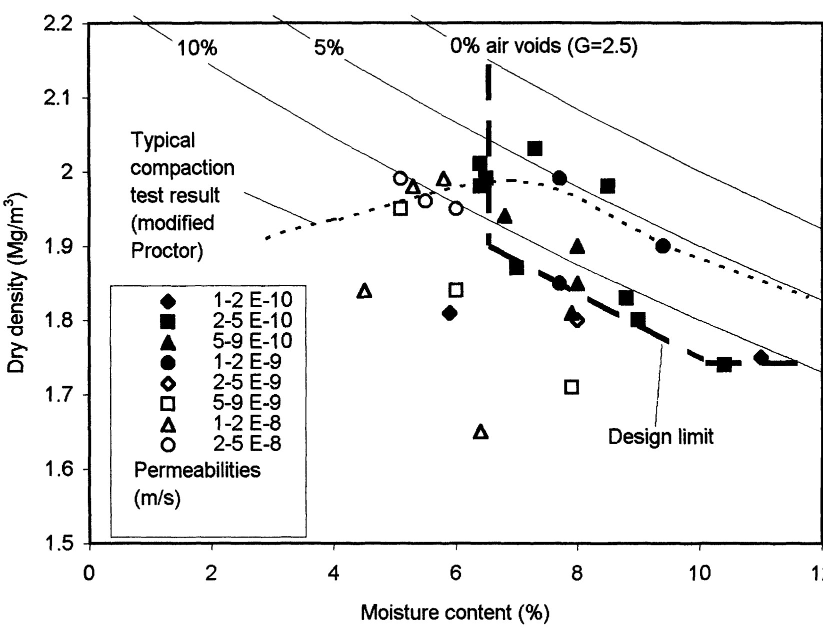


**Figure 7** Example of determination of acceptable compaction states

In order to illustrate this approach, Figure 7 shows an example of a compaction diagram for a freshly dug spoil with a maximum particle size of 30mm and a fines content of between about 10 and 20%; its mean grading curve is labelled "Spoil C" in Figure 1. The permeability data shown in Figure 7 were obtained in flexible-walled permeameters on laboratory compacted 100mm diameter specimens with an effective confining pressure of 250kPa and a hydraulic gradient of either 9 or 18. On the diagram a boundary has been drawn to the zone where the measured permeability is consistently less than $2x10^{-9}$m/s, which in some circumstances might be considered acceptable. However, this tentative design limit may not be reliable, in view of the likely differences between these permeabilities and large scale measurements discussed (and illustrated) in this paper. Therefore, if this type of approach is adopted, it is recommended that an allowance for the effects of scale is made. In view of the results shown in Figure 6, it is also important that the confining stress chosen for testing is carefully justified with respect to the field conditions. It should be noted that the data shown in Figure 7 were obtained for an unconventional engineering application and that the design limit shown was not actually adopted.

At construction stage, the compaction parameters (density and moisture content) must be kept within the limits of the acceptable zone as defined by the preliminary laboratory testing. In addition, it is desirable to monitor permeability directly.

Figure 8 shows an example of permeability data obtained during construction of a liner by testing 100mm diameter thin-walled tube samples in a flexible-walled permeameter. The effective confining pressure ranged between 150kPa and 300kPa and the hydraulic gradient was 10. The spoil used in the liner had weathered in tips on site over many years. It was passed through a 40mm screen prior to placement and initially had a fines content of between 24 and 44%; its mean grading curve is labelled "Spoil D" in Figure 1. Upon compaction, the coarser particles broke down readily, so that it was not difficult to take tube samples. It can be seen that all the data in Figure 8 comfortably comply with the usual requirement that the permeability is less than $10^{-9}$m/s and that the compaction states lie in or close to the desired region, indicating a good quality of construction. The limits adopted for control purposes are also shown in the figure. In this case one can have reasonable confidence that, under a corresponding confining or overburden stress, the real (large scale) permeability will also be less than $10^{-9}$ m/s. Nevertheless, ideally such measurements should be supported either by large scale field permeability testing on a test pad or large scale permeameter tests on blocks taken from the liner. For SDRI tests on colliery spoil, the present research has highlighted two potential problems. Firstly, the equipment is not necessarily easy to install and disturbance effects may be significant if the diameter of the inner ring is less than 1m. Secondly, the saturated (or near saturated) permeability may be underestimated if the infiltration takes place through preferential pathways and leaves most of the spoil in an unsaturated condition. Therefore, it is recommended that only inner rings of more than 1m diameter are employed and that the condition of the spoil beneath the SDRI is carefully established after the test. This may also be important for deciding on the appropriate depth of infiltration when interpreting the results in terms of permeability.

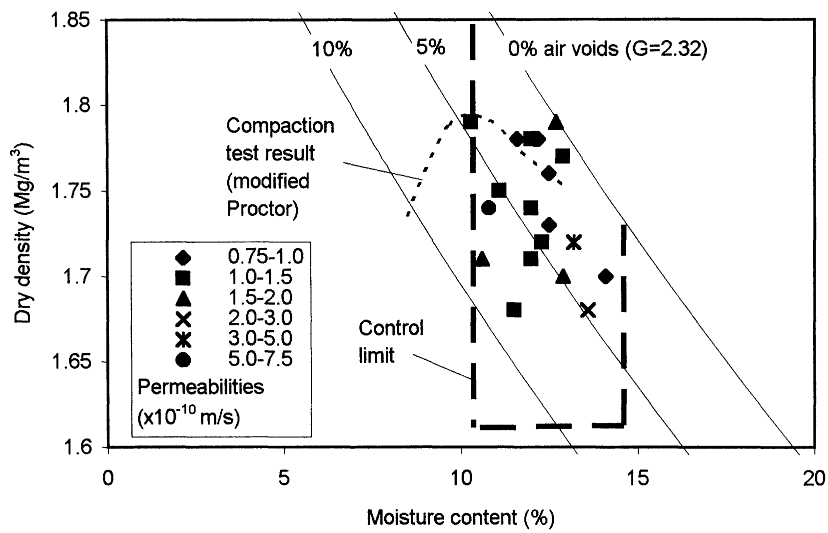


**Figure 8** Example of control of compaction and permeability

The above comments assume that a source of reasonably consistent spoil is available. In practice, natural variation in the spoil may introduce a variability of the optimum moisture content with a given compaction method, with potentially severe consequences if the spoil is unintentionally compacted dry of optimum. Also, variation in the mineral composition may lead to a wide variation in the average specific gravity (see Table 1) which is used to calculate dry density and the position of the zero air voids line on the compaction diagram. It is important to measure the specific gravity adequately and it is fundamentally better to use void ratio rather than dry density when comparing compaction states (although dry density has been used in this paper). The variation of the coal content could also affect the calibration of a nuclear density gauge, if such an instrument were being used to monitor compaction. These effects may make the determination and operation of design and control limits such as those in Figures 7 and 8 difficult and may justify the use of an alternative indicator of compaction such as the Moisture Condition Value test (Murray *et al.,* 1992). However, the Authors have no knowledge of other approaches having been tried for colliery spoil.

Because permeability is sensitive to the fines and especially the clay content of a spoil, even a limited amount of natural variation can obscure the expected trends for the change of permeability with dry density and compaction moisture content. Thus, in Figure 8 no particular pattern is discernible in this respect. In Figure 7, a specimen with one of the lowest densities and lowest compaction moisture contents had the lowest permeability; specimens with closely similar compaction states (dry densities 1.95-2.00Mg/m$^3$ and compaction moisture contents 5.5-6.5%) had very different permeabilities. This variation further complicates the problem of defining suitable design or control limits.

In the face of material variations and the difficulties that they cause, it is possible that a statistical approach to quality control might be devised. However, this would depend on generating sufficient test data to support the statistical model and in practice an engineering judgement of risk is likely to be made instead.

With regard to the initial selection of materials for liner construction, the wide variation in the nature and properties of colliery spoils makes generalisation difficult and guidelines based on index tests (Daniel, 1993) are not necessarily reliable. However, the research described above has established that the maximum particle size, up to 50mm, does not have a significant influence on the permeability, given good mixing and compaction and a well graded material with an adequate fines content. Where such large particles are not screened out, field trials are needed to ensure that adequate mixing occurs. Lower permeabilities are achieved as the fine fraction increases and the data given in this paper, for spoils with a variety of fines contents, provide limited guidance on this trend. On present evidence, it appears unlikely that consistently satisfactory results will be obtained with less than 20% fines, when the requirement is for a permeability of less than $10^{-9}$m/s.

## Conclusions

1) Colliery spoils vary in their suitability for use in landfill liners, depending upon their history of formation, alteration, extraction and storage, and individual spoils must be investigated in detail in order to determine their suitability. This must be set against the economic and environmental benefits that can be realised by using colliery spoil.

2) Colliery spoils which do not degrade easily or initially contain at least 20% of fines (silt and clay) are unlikely to prove suitable in normal circumstances.

3) The permeability of compacted colliery spoil is not significantly affected by the maximum particle size, providing the fines content is adequate and the material is well graded and well mixed. Unfavourable gradings can lead to particle migration, with uncertain consequences.

4) The permeability of compacted colliery spoil reduces significantly with increasing confining pressure in a similar manner as it does for clays. This should be recognised when specifying and interpreting permeability tests.

5) For colliery spoils, a compaction moisture content slightly wet of optimum secures the lowest permeability. Although the permeability may not be as sensitive to compaction moisture content as it is for clays, large increases in permeability are still possible if the spoil is compacted dry of optimum.

6) As for clays, the testing method has a profound influence on the measured permeability of colliery spoil and the results of conventional small scale laboratory tests may be grossly misleading. Therefore, large scale tests, either in situ or on undisturbed samples in the laboratory, are recommended as part of the validation of liner performance.

7) The same framework for designing colliery spoil liners and controlling their construction can be adopted as that established for clays. However, variability of grading and mineralogy may make this difficult in practice and careful engineering judgement may need to be exercised.

## Acknowledgements

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## References

ASTM D5093 (1990). *Standard test method for field measurement of infiltration rate using a double-ring infiltrometer with a sealed-inner ring,* American Society for Testing and Materials, Philadelphia.

Benson, C.H., Hardianto, F.S. & Motan, E.S. (1994). Representative specimen size for hydraulic conductivity assessment of compacted soil liners. *Hydraulic Conductivity and Waste Contaminant Transport in Soil, ASTM STP1142*, American Society for Testing and Materials, Philadelphia, 3-29.

BS 1377 (1990). *Methods of test for soils for civil engineering purposes, British Standards Institution*, London.

Cousens, T.W. & Studds, P.G. (1996). The attenuation and leaching behaviour of colliery spoils. *Waste Management* 16, 195-202.

Czerewko, M.A. (1997). *Diagenesis of mudrocks, illite 'crystallinity' and the effects on engineering properties*. PhD thesis, University of Sheffield.

Daniel, D.E. (1984). Predicting the hydraulic conductivity of compacted clay liners. *Journal of Geotechnical Engineering*, ASCE, 110 (2), 285-300.

Daniel, D.E. (1993). Clay liners. *Geotechnical Practice for Waste Disposal*, Chapman and Hall, London, 137-161.

Daniel, D.E. (1994). State-of-the-art: laboratory hydraulic conductivity tests for saturated soils. *Hydraulic Conductivity and Waste Contaminant Transport in Soil*, ASTM STP1142, American Society for Testing and Materials, Philadelphia, 30-78.

Elsbury, B.R., Daniel, D.E., Sraders, G.A. & Anderson, D.C. (1990). Lessons learned from compacted clay liner. *Journal of Geotechnical Engineering*, ASCE, 116 (11), 1641-1660.

Hird, C.C., Norton, E. & Joseph, J.B. (1997a). Permeability testing of a colliery spoil landfill liner. *Geoenvironmental Engineering - Contaminated ground: fate of pollutants and remediation*, Thomas Telford, London, 337-342.

Hird, C.C., Norton, E. & Joseph, J.B. (1997b). Particle migration effects on colliery spoil liner permeability. *Proceedings 6th International Landfill Symposium*, Sardinia 3, 131-140.

Kenney, T.C. & Lau, D. (1985). Internal stability of granular filters. *Canadian Geotechnical Journal* 22 (2), 215-225.

Mitchell, J.K., Hooper, D.R. & Campanella, R.G. (1965). Permeability of compacted clay. *Journal of Soil Mechanics and Foundations Division, ASCE*, 91 (SM4), 41-65.

Murray, E.J., Rix, D.W. & Humphrey, R.D. (1992). Clay linings to landfill sites. *Quarterly Journal of Engineering Geology* 25 (4), 371-376.

Norton, E. (1998). *The permeability of a compacted colliery spoil for use as a landfill liner*. PhD thesis, University of Sheffield.

Sai, O.J. & Anderson, D.C. (1990). Field hydraulic conductivity tests for compacted soil liners. *Geotechnical Testing Journal*, ASTM, 13 (3), 215-225.

Shackleford, C.D. (1994). Report of technical committee on environmental control (TC5). *Proceedings 1st International Congress on Environmental Geotechnics,* Edmonton, 981-1005.

Smith, C.C., Cripps, J.C, & Wymer, M.J. (1997). Specification of a colliery spoil landfill liner. *Geoenvironmental Engineering - Contaminated ground: fate of pollutants and remediation*, Thomas Telford, London, 383-388.

Taylor, R.K. (1984). *Composition and engineering properties of British colliery discards*. Mining Department, National Coal Board, London.

Taylor, R.K. (1988). Coal Measures mudrocks: composition, classification and weathering processes. *Quarterly Journal of Engineering Geology* 21 (1), 85-99.

Taylor, R.K. & Spears, D.A. (1970). The breakdown of British Coal Measure rocks. *International Journal Rock Mechanics and Mining Science* 7, 481-501.

Taylor, R.K. & Spears, D.A. (1972). The influence of weathering on the composition and engineering properties of in situ Coal Measures rocks. *International Journal Rock Mechanics and Mining Science* 9, 729-756.

Trast, J.M. & Benson, C.H. (1995). Estimating field hydraulic conductivity of compacted clay. *Journal of Geotechnical Engineering*, ASCE, 121 (10), 736-739.

Trautwein, S.J. & Boutwell, G. (1994). In-situ hydraulic conductivity tests for compacted soil liners and caps. *Hydraulic Conductivity and Waste Contaminant Transport in Soil, ASTM STP1142*, American Society for Testing and Materials, Philadelphia, 184-223.

Trautwein, S.J. & Williams, C.E. (1990). Performance evaluation of earthen liners. *Waste Containment Systems: Construction, Regulation and Performance, ASCE Geotechnical Special Publication No. 26,* American Society of Civil Engineers, New York, 30-51.

# The design and control of bentonite enriched soils

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## Introduction

Bentonite enriched soil systems are used as landfill liners because they are economic and they work. However, it is well known that bentonite is a swelling mineral and that its swelling is strongly influenced by the local chemical environment. This has lead to regular concerns about the long term performance of bentonite enriched soil systems. In this paper a simple two phase model is used to represent the bentonite enriched soil (BES), to highlight important parameters for control of the material on site and to identify situations which may lead to significant increase in its permeability on exposure to aggressive chemical environments.

## Permeability of bentonite enriched soils

Typically mineral barriers to contaminant migration are required to have a permittivity of $10^{-9}\,s^{-1}$ that is to have a barrier performance equivalent to one metre of material of permeability $10^{-9}$ m/s. Thus a 300 mm thick layer of a bentonite enriched soil (BES) could be required to have a permeability of $3.3\times10^{-10}$ m/s. In practice this permeability may be further reduced by regulatory requirements to perhaps $10^{-10}$ m/s or lower. BES materials are therefore likely to be subject to stringent permeability controls which can require considerable design effort prior to the start of construction work on site. Furthermore relatively large increases in the bentonite content with corresponding significant increases in cost may be required to compensate for modest deviations from a design permeability and thus the mix design needs to be robust.

*Geotechnical engineering of landfills.* Thomas Telford, London, 1998

## Fines in soils

Before considering the properties of BES it is instructive to consider briefly the properties of soils in general. Geotechnical engineers are familiar with the use of the $D_{10}$ (the particle size for which 10% of the soil is finer) as an indicator of soil behaviour. A material with a $D_{10}$ in the clay range might be expected to behave rather like a clay and have a clay type permeability. Though when applying such considerations the whole particle size distribution for the soil needs to be considered especially for gap graded materials.

As a more general approach than the adoption of $D_{10}$ as an indicator parameter consider a soil arbitrarily divided into fine and coarse fractions with p% fines. It follows that $D_p$ is the dividing particle size between the fractions and if it is assumed that all the moisture in the soil is associated with the fine fraction so that there is a fines paste filling the voids between the coarse grains, then:

$$w_f = w / p$$

where $w_f$ is the moisture content of the fines paste and w is the normal moisture content of the whole soil, that is the weight of water divided by the sum of the dry weights of the coarse and fine fractions. The dry density, $\rho_d$ of the whole soil is given by:

$$\rho_d = [\rho_w (1-A_r)] / [w + p / G_f + (1 - p) / G_c]$$

where $\rho_w$ is the density of water, $A_r$ the soil air void ratio and $G_f$ and $G_c$ the specific gravities of the fine and coarse fractions respectively.

Figure 1 shows the relationship between moisture content of the fine fraction (the paste) and dry density of the whole soil. It can be seen that the saturated moisture content of a paste formed from a soil containing 10% fines must be in the range from 289% to 77% for dry densities of the soil from 1.5 to 2.2 Mg/m$^3$ (assuming that the average grain specific gravities of the coarse and fine fractions, $G_c$ and $G_f$ are both 2.65). Thus for natural soils the paste moisture content actually may be substantially greater than the liquid limit if the soil is saturated and the paste fills all the voids between the coarse particles.

There is little risk that the paste could be extruded from a soil at such a water content (extruded as in the sense of squeezing toothpaste from a tube) by typical in-situ hydraulic gradients (e.g. Jefferis, 1972 reported in Xanthakos, 1979). Even a weak paste can resist extrusion by substantial gradients.

However, there is a severe risk that there could be erosion of the fines from a soil with a $D_{10}$ in the clay range and a low dry density if the soil were subjected to a sustained hydraulic gradient. Erosion is dangerous as it will cause a rapid and substantial increase in permeability. Even for 20% fines the 'paste' moisture content may be unacceptably high unless dry densities in excess of perhaps 1.8 Mg/m$^3$ can be achieved. These calculations show that although $D_{10}$ is often used as an indicator of soil properties, these properties may not always be as robust as anticipated and natural soil liners should be analysed with just as much caution/suspicion as BES liners.

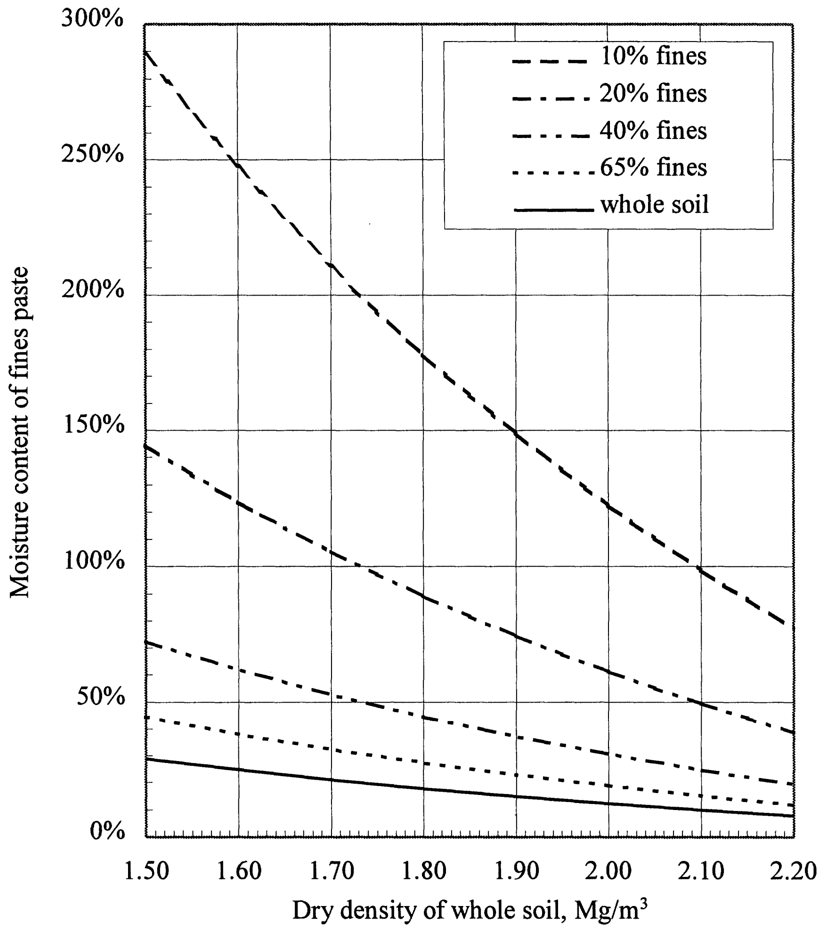


**Figure 1** Moisture content of fines - Dry density of whole soil

## The bentonite enriched soil system

A BES will contain bentonite, a selected soil (usually a sand though other materials such as pulverised fuel ash may be used), water and air. The bentonite enriched soil system is no different from any other soil system except that it is likely to be markedly gap graded unless the soil used to prepare the BES is very widely graded. Thus the concept of $D_p$ can be crucial for BES design.

As bentonite has a very strong affinity for water, it is likely that most of the water will be associated with the clay and thus the paste model described above can be used to describe a BES with, p, the fraction of fines now representing the bentonite content (or if the soil contains clay sized fines, the bentonite plus clay fines content of the soil).

Typically the bentonite content of a BES may be in the range 5 to 15%. Below 5% the BES is unlikely to achieve the required permeability and may be sensitive to erosion (unless the 'soil' contains a significant proportion of fines). Above about 15% the strength properties of the material may be undesirably dominated by the soft bentonite paste and it may show little sand type behaviour. Furthermore other materials may be more economic.

## Applications of bentonite enriched soil liners

BES systems will find application as landfill liners where no suitable natural clay is available and/or the increased air space that can be achieved with a thin BES liner is significant.

Fundamentally the selection of BES as against other liner systems is likely to be dominated by cost rather than technical considerations. The bentonite component of a BES will make it substantially more expensive than a locally available natural clay liner material. Thus for a BES the design brief is likely to be to use the lowest bentonite content consonant with achieving the required technical performance. When designing a BES it will be necessary to consider not only the cost of the bentonite but also:

- the required in situ permeability
- the required chemical compatibilities
- the properties of the bentonite, liquid limit, permeability, swell etc.
- the grading of the soil(s) that could be used to prepare the BES
- the cost of the soil(s) that could be used to prepare the BES

There has been a significant amount of work published on the properties of BES systems and outline design rules exist (see for example Kenney *at al.,* 1992 and Mollins *et al.*, 1996). Rather less work has been published on the significance of the soil properties.

In some instances the type of soil used to prepare the BES may be dictated by what is available but it should be recognised that a poorly selected soil may double the bentonite demand for a BES and still give a material of poor permeability and chemical resistance. In the design process it will be necessary to balance the costs of obtaining an appropriate soil against the extra costs of bentonite if a locally available but poorly graded material is used.

## Materials for bentonite enriched soils: The bentonite

Factors that will need to be considered when designing a BES include the source of the bentonite and its properties including:

- Swelling
- Liquid limit

- Ion exchange capacity

For the present discussion swelling is perhaps the most important parameter.

## Swelling

Unfortunately there are many different procedures used to determine the swelling of soils and there is little consensus in the literature as to the preferred procedure for bentonites. The following sections give a brief outline of some test procedures:

### *Swelling volume test*

Supplier's literature for bentonites may include a swelling volume. The test procedure is to very slowly sprinkle 2 g of the bentonite powder on to the surface of 100 ml of water in a measuring cylinder, allowing each portion of bentonite to settle before adding the next. The volume of swollen material is recorded at 2 hours. Typically specifications may require a final volume of greater than 24 ml per 2 g of bentonite powder. If the initial moisture content of the bentonite is 11% by dry weight (10% by moist weight - NB in the bentonite literature moisture contents often are reported by moist weight) then the moisture content of a material swollen to 12 ml/g will be 1313% - more of a thick slurry than a soil. It is important to note that the test effectively measures the volume of water that can be retained by a bentonite. A dry bentonite powder will not swell to this volume if exposed to water.

### *Water absorption*

ASTM E946-83 (re-approved 1987) gives a procedure which more realistically represents the swelling of a bentonite powder when exposed to water. In this procedure a sintered aluminium oxide plate is flooded with water in a dish to within 0.25 inch of the surface of the plate. A 5 cm diameter ring is placed on a dry filter paper and 2 grams of oven dried bentonite (at 105°C) are sprinkled onto the paper preferably using a vibrating spatula. The paper is then placed on the sintered plate and the water temperature measured. The dish is then covered and after 18 hours the water temperature is re-measured and the bentonite and paper removed and weighed. A filter paper without bentonite is subjected to the same procedure to determine the water taken up by the paper alone. The sintered plate is designed to be large enough to allow up to four samples to be tested at once. The absorption is calculated as follows:

$$\text{Absorption, \%} = [(W_w - W_d) / W_d \times 100] - 3.3\,(T_a - 20)$$

where $W_w$ is weight of hydrated bentonite in grams, $W_d$ the weight of dry bentonite at the start of the test, and $T_a$ the average of the initial and final temperatures of the water.

### *Water intake test*

A similar type of test to the above absorption test is referenced in GLR, German Geotechnical Society (1993). This follows the Enslin procedure (see the German standard, DIN 18132) whereby water is drawn from a capillary tube into bentonite powder supported on a small sintered disc. No limit value is prescribed in GLR (1993) but a plot is presented showing sorption up to 700% moisture content.

### *Settling test*

Sridharan & Prakash (1998) propose a settling test and a free swell index test. Both tests involve settling behaviour and therefore are measures of water retention rather than free swell. Although in theory water retention and swelling tests may converge to a common value, in practice, because of the inevitable differences in the final degree of dispersion of the clays in the different procedures, the results will be different.

### *Consolidation tests*

Swelling behaviour under one-dimensional stress has been measured by a number of authors (see for example Studds *et al.*, 1998). Such tests give useful information on the behaviour of bentonite but have the disadvantage of requiring a significant amount of laboratory time and equipment.

### *Liquid limit*

Although swelling can be a useful parameter when attempting to identify limiting values for bentonite/soil ratios for BES systems, it should be remembered that a bentonite at its swelling limit will have effectively zero strength. In this state it will be very sensitive to erosion and chemical interaction effects which cause shrinkage (e.g. ion exchange of sodium for magnesium or calcium or increase in the ionic strength of the pore solution). It is appropriate to consider as a possible reference point a moisture content at which the bentonite still retains some soil type properties. The most widely known parameter of this type is the liquid limit. The liquid limit has the advantage that there is a standardised measurement procedure which is straightforward and indeed some suppliers include the information in their literature.

From a standpoint of engineering practice the liquid limit is probably the most useful parameter with which to reference bentonite-water interaction. Typically the liquid limit of a sodium bentonite may range from less than 200% to over 700% depending on the source of the raw material and the processing prior to supply.

It should be noted that some bentonites are strongly polymer enhanced and this may increase the liquid limit. These bentonites should be treated with some caution. They may be appropriate in short term applications such as slurries for excavation support but in long term applications such as liners it will be necessary to consider the consequences of any degradation of the polymer (this need not be damaging).

## Materials for bentonite enriched soils: The soil

The soil used to prepare a BES can have a substantial impact on the mix design as the permeability of a BES will be influenced by:

- The permeability of the bentonite 'paste' in the pores between the soil particles (this will be a function of the bentonite content and the porosity of the soil grains)
- The cross-sectional area available for flow - this will be a function of the porosity of the soil grains
- The soil packing which will increase the flow path length because the permeating fluid must follow a tortuous path between the grains.

Mollins *et al.* (1996) found a single straight line relationship between permeability and void ratio on a log-log plot (NB overall void ratio and not bentonite void ratio) for two BES mixes (10 and 20% bentonite) and a pure bentonite mix. Such a relationship is not unusual for a single soil when subject to increasing confining pressure, however, it seems somewhat surprising that they found a single relationship for what were in effect three distinct soils. The finding suggests that the three soils effectively behaved as a single soil.

Mollins *et al.* then went on to develop a procedure to account for the effects on BES permeability of both the reduction in flow area due to the soil grains and the tortuosity they introduce. The effect of the reduction in flow area is well known and is simply equal to the porosity of the soil. Thus a BES with a soil porosity 40% will, in theory, have a permeability of 40% of that of the bentonite paste in the pores (assuming there are no preferential flow paths at the bentonite-soil grain contacts). Mollins *et al.* suggest that the effects of tortuosity can further reduce the permeability of a BES by a factor of 10 to 20. This seems to be a relatively large effect - especially as an analysis of the problem of tortuosity based on the Carman-Kozeny equation suggests that the effect of tortuosity is limited to a permeability reduction of about 1.6 times.

Kenney *et al.* (1992) compared predicted and measured permeabilities of BES systems. Their procedure used the simple porosity correction noted above, that is:

$$k_m = k_b \times n$$

where $k_m$ is the permeability of the BES, $k_b$ that of the bentonite water paste in the pores of the BES and n the porosity of the soil (excluding the bentonite).

Thus Kenney *et al.* took no account of tortuosity. Their predicted and experimental results showed reasonable agreement (though it must be allowed that both the data of Kenney *et al.* and Mollins *et al.* show some experimental scatter) as is expected for permeability data and without more detailed analysis it is not to possible resolve the conflict between the two procedures and to assess whether tortuosity has the substantial impact suggested by Mollins *et al.*

It is important to recognise that the BES, as tested in the laboratory, is likely to be in a slightly different state from that in the field after compaction. It will be saturated at the effective confining stress of the test (it is important that this is related to the design effective stress in the field).

Figure 2 shows a typical set of results for a range of bentonite contents. It can be seen that there is a general trend of decreasing permeability with decreasing bentonite moisture content. However, there are outliers, particularly for the lower bentonite contents in the BES, where the moisture content is high and markedly above the liquid limit for the particular bentonite, which was about 280%. The average porosity of the soil in the BESs shown in Figure 2 was 0.41 with a standard deviation of only 0.024. Thus the porosity showed too little variation to enable valid investigation of its effect. This demonstrates a typical problem - field mixes are likely to show only very limited ranges of bentonite moisture content and soil porosity. Thus investigation of the effect of

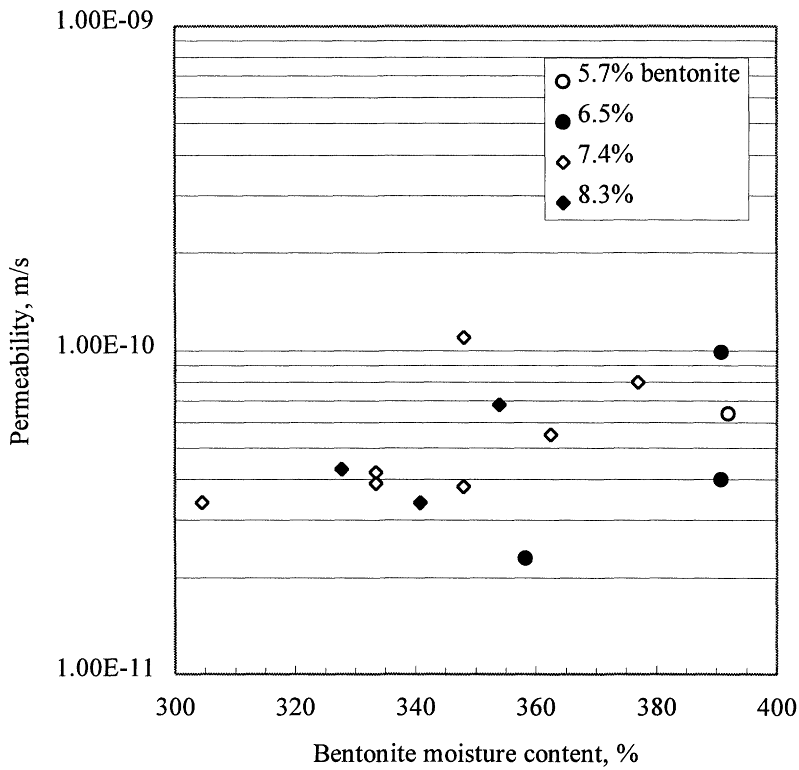


**Figure 2** Permeability of BES as a function of bentonite moisture content

these parameters in the field is difficult.

It should also be remembered that the bentonite content in the BES is fundamental to its cost effectiveness (e.g. a 0.5% increase from 8% to 8.5% bentonite will be significant). Designers must be aware of the need for tight but robust mix design which is difficult given that Figure 2 represents a typical scatter of results from BES testing.

## The behaviour of BES in-situ

Typically in the field a BES will be mixed and stockpiled in advance of its placement as a liner. The moisture content of the as-mixed BES will be designed to be appropriate to the compaction regime to be employed when laying it on site. If the BES stockpile is exposed to rain, a skin of wetter bentonite will form at the surface but the moisture content of the mass of the material will be little changed provided suitable storage times are respected. Once compacted and covered with a further lining system (e.g. an HDPE geomembrane), if moisture is available from below, the bentonite will slowly swell and expand the BES (provided there is sufficient bentonite in the voids) to achieve equilibrium with the overburden stress (NB the overburden stress will change with time as a landfill is filled and thus the moisture content of the bentonite in the BES will also change over time). As will be shown below it is very important that the bentonite in the pores of the soil used to prepare the BES should behave as a coiled spring (that is the overburden stress should be borne, at least in part, by the bentonite and not wholly by intergranular stress between the soil grains) at least at the low effective stresses, for example, when landfilling starts. This can be achieved only by proper design of the BES and matching of the properties of the soil and the bentonite.

A fundamental parameter for the soil will be the dry density that can be achieved under the field compactive effort. This dry density can be regarded as the baseline. It is likely to be the maximum that can be achieved in a BES system - the addition of bentonite is likely to significantly reduce the dry density that can be achieved. NB it is important to note that the reference state is the dry density of the soil without the added bentonite particles. The bentonite and associated water must fill all the voids in the soil at an acceptable bentonite moisture content.

Figure 3 shows the bentonite moisture contents necessary to fill the voids for a range of soil dry densities and bentonite contents (a grain specific gravity of 2.65 has been assumed for the dry soil and 2.78 for the dry bentonite).

It can be seen that if the acceptable moisture content is greater than about 300% then there is little constraint on the dry density of the soil (the acceptable moisture content may be determined by reference to the liquid limit of the bentonite). For example, 6% bentonite with a soil of dry density of 1.7 $Mg/m^3$ (a realistic field value) would give a bentonite moisture content of 300%.

However, for a poorer bentonite this moisture content might not be acceptable; if only 200% moisture were acceptable, then the bentonite content would have to be over 8% thus making the mix significantly more expensive.

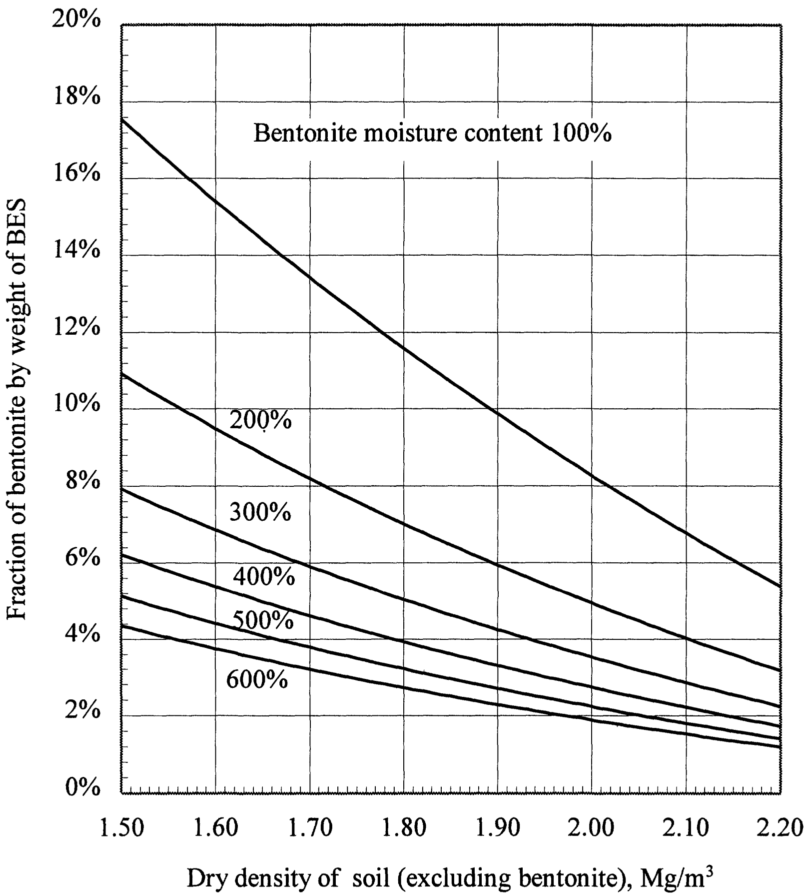


**Figure 3** Fraction of bentonite - Dry density relationship

The message is simple, the higher the acceptable moisture content of the bentonite the lower the acceptable bentonite content in the BES. Cost-benefit analyses may be carried out to assess bentonites from different sources (with different liquid limits) using the data from Figure 3. Such analyses may show that the importation of Wyoming bentonite is not cost effective but that the careful choice between sodium converted calcium bentonites from different sources is cost effective.

## Field Control

In the field it is necessary to control the placement of the BES. The bentonite content of the mix can be controlled by good weigh batching and check tests using the methylene blue procedure - though this is of limited resolution. In addition to bentonite content, research by Golder Associates has shown that the density of the compacted BES is fundamental to both the economy and performance of the BES.

Figure 4 shows a plot of moisture content of the bentonite in a BES as a function of the dry density for various bentonite contents (this figure presents similar information to Figure 1 but is framed differently).

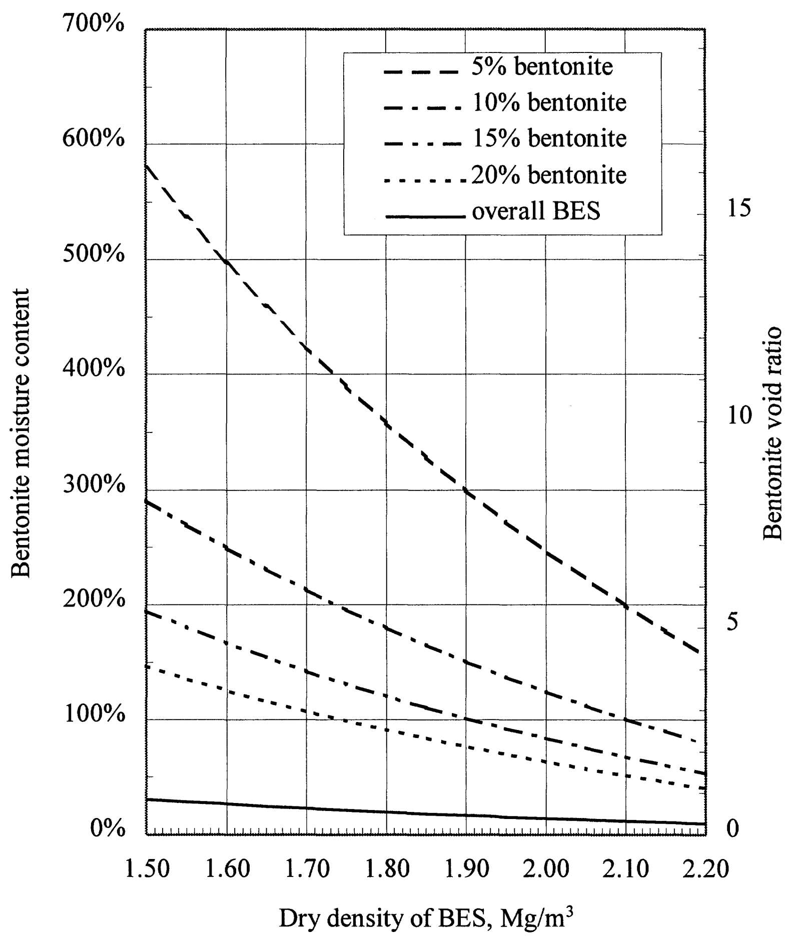


**Figure 4** Moisture content of bentonite - Dry density relationship

A number of points can be drawn from Figure 4:

1. The moisture content of the bentonite paste in a BES reduces significantly as the dry density increases.
2. It can be seen that for 5% bentonite content the moisture content is very high except at very high dry densities. In general Mediterranean bentonites will be used for liners in the UK as the substantial extra cost of Wyoming bentonite can seldom be justified (there may be no benefit - merely extra cost).
3. For the 5% bentonite at the lower dry densities the moisture content is likely to be above the liquid limit. Although a mix at such a water content could function as a liner there would be a serious possibility of erosion of the bentonite if any significant hydraulic gradient were maintained across the liner.
4. Low bentonite contents can be used if the soil has a significant fines content and thus the statements in (2) and (3) above should not be taken as barring the use of low bentonite contents.
5. It could be argued that because Wyoming bentonites can have higher liquid limits than Mediterranean bentonites they should be preferred for liners. However, all sodium bentonites are susceptible to chemical interaction effects and a higher water content material is likely to be worse affected. Thus for real liners the use of Wyoming bentonite at a lower concentration, but still at possibly higher overall cost than Mediterranean bentonite, may bring no benefits.
6. Dry density can be increased by increasing the compactive effort but the benefit becomes progressively smaller as the effort increases. Despite this it will be very important properly to compact the BES both to reduce the air voids content and to knead the bentonite into the soil (in service the bentonite moisture content will be a function of the thermal, moisture and stress conditions within the BES pores).
7. The most effective way to increase the dry density will be by using a well graded soil.
8. Increasing the dry density can markedly reduce the bentonite demand as there are fewer voids to be filled. A 10% bentonite BES at a dry density of 1.6 $Mg/m^3$ has approximately the same moisture content as a 5% bentonite at 2.0 $Mg/m^3$.
9. At constant bentonite moisture content, increasing the dry density of the soil will reduce the permeability of the BES as it will reduce the area available for flow and increase the tortuosity.
10. At constant bentonite content, increasing the dry density of the soil will improve the resistance of the BES to chemically induced increase of permeability as it will permit a lower bentonite moisture content.

## Chemical effects

Exchange of the sodium ions in the bentonite with ammonium, potassium, calcium and magnesium or other ions in a landfill leachate or an increase in the ionic strength of the porewater (the mix water may have been tap water or good quality ground/surface water of lower ionic strength than landfill leachate) generally will reduce the swelling capacity and liquid limit of bentonites. However, it is important to note that if the bentonite is mixed with fresh water before it comes into contact with contaminated water the effect of the contaminants is likely to be small (the quantity of water used to prepare a BES in the field - typically to slightly above Proctor optimum moisture content for the mix will be sufficient to cause some swelling/separation of the bentonite particles and thus reduce later chemical sensitivity). For example, Kenney *et al.* (1992) found a less than half an order of magnitude increase in permeability when bentonites with void ratios between 3 and 10 were permeated with 40 g/litre sodium chloride (i.e. a solution slightly stronger than sea water). Gleason *et al.* (1997) found substantial effects on a BES containing 5% sodium bentonite (liquid limit 603%) when permeated with 0.25 molar calcium chloride solution (a similar result was observed for a 15% calcium bentonite, liquid limit 124%). These results are as expected from the above discussion on bentonite moisture content in a BES and merely confirm that damage can occur if the fines in a BES are at an unacceptably high moisture content in relation to some water affinity indicator such as the liquid limit.

It follows that a fundamental requirement of BES design is that there should be sufficient bentonite paste (bentonite plus water) to ensure that the paste is under stress. The paste should be the equivalent of a coiled spring between the soil grains. The soil grains may be in contact if the external effective stress is high (e.g. from a substantial depth of waste). Chemical effects must not reduce the 'spring' in the paste to zero otherwise damage will ensue. Maintenance of the 'spring' requires a sufficient bentonite content.

## Drying and freezing

Although chemically induced damage should be limited in a well designed BES subject to permeation by most leachates, greater damage may occur if there is drying or freezing of the liner after a chemical change in the porewater has occurred i.e. an effectively dry bentonite is hydrated in a chemically contaminated water. Drying may occur as a result of heat from the decomposing waste. For the maximum damage the following sequence of events would be necessary:

1. Placement of BES.
2. Permeation/diffusion of a chemically aggressive leachate into the BES leading to a slight increase in permeability (slight as the bentonite was wetted in fresh water).

3. Substantial drying of the bentonite; this requires heat but without any leachate permeation.
4. Re-wetting of the dried bentonite with the aggressive leachate.

It is difficult to see how this sequence of events could occur but if a landfill is to be designed or operated such that it is possible, then the BES would have to be designed to withstand the regime (the simplest way to do this is to measure the liquid limit of the bentonite when mixed from the dry state with the aggressive leachate and then design accordingly).

Freezing can have the same effect as drying. However, it is unlikely to occur in liners except prior to placement of the protection layers and waste. Freezing at this time will have little effect on the liquid limit of the bentonite but could disrupt its compaction. Freezing could occur in capping layers but chemical conditions are unlikely to be as aggressive as for liners, though it should be noted that under low effective stress conditions the bentonite in a BES may swell substantially and if there is later ion exchange (e.g. with calcium from the cover soils) and the BES layer does not compress to achieve a new lower equilibrium moisture content then the damage can be severe. BES materials must be subject to some effective stress (as must geosynthetic clay liners).

## Bentonite enriched soil mixing

Good mixing of the bentonite and soil is essential to the performance of BES systems. Haug & Wong (1992) showed that preparation of BES dry of Proctor optimum leads to increased permeability. They attributed this to the difficulty of mixing soil with a rather dry bentonite. Intimate mixing requires that the bentonite is soft enough to be spread over the soil grains. A particular advantage of sodium bentonites is that they swell on contact with water and thus tend to absorb all available water (though of course up to a swelling limit). Other clays such as calcium bentonite will not so readily sorb water and thus may require more mixing energy to homogenise their water content and soften them to the state when they can be spread over the surface of the soil.

## Conclusions

Bentonite enriched soils are important mineral lining materials. For economic design it is necessary to consider not only the properties of the bentonite and the effects of any chemical interactions but also to consider the properties of the soil and in particular the dry density that it can achieve (without bentonite) under field compaction conditions. The use of well graded soils which can achieve high dry densities can reduce bentonite demand, improve the permeability of the mix and its chemical resistance. As dry density (or its equivalent, soil porosity) is a fundamental parameter for BES design it would be useful if publications on the subject provided data on this. Unfortunately it is often omitted.

The permeability of a BES will be a function of the permeability of the bentonite paste in the pores of the material and the porosity of the soil grains. The paste must be at an acceptable moisture content if a low permeability is to be achieved and also resistance to chemical effects, drying and freezing. It has been suggested that the paste moisture content can be designed by reference to some measure of the swelling of the bentonite. This may be acceptable provided a substantial factor of safety is applied to the resulting moisture content. It is suggested that a simpler procedure is to consider the liquid limit of the bentonite and apply a more modest factor of safety, which should be determined by further research.

## References

American Society for testing materials, E946 - 83 (Reapproved 1987), *Standard test method for water absorption of bentonite porous plate method.* DIN 18132, Wasseraufnahmeversuch.

German Geotechnical Society (1993). GLR, *Geotechnics of landfill design and remedial works, technical recommendations*, Ernst & Sohn, Berlin.

Gleason, M.H., Daniel, D.E. and Eykholt, G.R. (1997). Calcium and sodium bentonite for hydraulic containment applications, *ASCE Journal of geotechnical and geoenvironmental engineering*, 123, 5.

Haug M.D. and Wong L.C. (1992). Impact of molding water content on hydraulic conductivity of compacted sand-bentonite, *Canadian Geotechnical Journal*, 29, 2.

Kenney, T.C., van Veen, W.A., Swallow, M.A. and Sungalia, M.A. (1992). *Canadian Geotechnical Journal*, 29, 3.

Mollins, L.H., Stewart, D.I. and Cousens, T.W. (1996). Predicting the properties of sand-bentonite mixtures, *Clay minerals*, 31.

Sridharan, A. and Prakash, K. (1998). Characteristic water contents of a fine grained soil-water system, *Geotechnique*, XLVIII, 3.

Studds, P.G., Stewart, D.I. and Cousens, T.W. (1998). The effects of salt solutions on the properties of bentonite-sand mixtures, to be published *Clay Minerals*.

Xanthakos, P. (1979). *Slurry Walls*, McGraw Hill, 82-83.

# Geosynthetics in Landfill Design

Geosynthetics, as defined by the International Geosynthetics Society are "planar, polymeric (synthetic or natural) material used in contact with soil/rock and/or any other geotechnical material in civil engineering applications". The definition is broad, as is their range of uses; barriers, drainage, filtration, protection, reinforcement, separation and surface erosion control are all potential uses. Most of these uses have applications within landfill engineering. This section comprises three papers that cover the stability of geosynthetics, recent testing developments and an overview of geosynthetic clay liners.

The paper by Jones & Dixon is concerned with the stability of geosynthetic landfill lining systems, a subject close to the heart of many geotechnical engineers. The stability of such systems are dependent on the available shear strength between the various interfaces within the system, and the authors present a summary of shear strengths for various geosynthetic/ geosynthetic and geosynthetic/soil interfaces. The data is produced from an extensive literature search supplemented by testing carried out by the authors.

The authors also present a methodology for assessing the stability of a cover soil resting on an inclined geosynthetic lining system. The method is an extension of the existing wedge method, and worked examples are given. The authors do point out, that whilst values of interface shear strengths from the paper may be used for schematic designs, they recommend site specific testing for detail designs. Further, they suggest that simply limiting equilibrium analysis may not be able to assess the stability of a geosynthetic lining system that is subjected to waste compression, a topic considered further in the first section of the symposium.

As well as ensuring the stability of an inclined geosynthetic lining system, the design engineer must safeguard the integrity of the geomembrane barrier. Darbyshire *et al.* consider some the issues surrounding the performance testing of geomembrane protection materials. They track the UK experience of protection systems and highlight the need for measuring the protection performance. A laboratory test, originally developed in Germany, can be used to assess the performance of geomembrane protection systems by subjecting a section of the lining system to a constant loading. The authors describe the difficulties associated with inconsistencies between testing houses. These problems led to the development of the Environment Agency cylinder test which is described by the authors and which will standardise this testing in the

UK. Interpretation of the results of the cylinder test is a topic of current debate, and this paper should focus future discussions.

Geosynthetic clay liners (GCLs) are assembled planar structures of geosynthetic materials and bentonite, and are used as primary and/or secondary liners in landfill applications. Gartung & Zanzinger present a comprehensive overview of the engineering properties and use of GCLs. The function and properties of GCLs, together with the importance of quality control are highlighted in the paper; which includes two case histories from Germany on performance observations from GCLs used in capping applications. At the Hamburg-Georgswerder site, problems of root damage, desiccation and cation exchange arose due to lack of sufficient soil cover. However, the second case history, at Nuremberg, reports a successful use of a GCL on a landfill capping system. The authors conclude with advice on the use of GCLs in landfill covers given by the German Institute of Construction; if this guidance is followed in the UK, desiccation can be avoided.

# The stability of geosynthetic landfill lining systems

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## Introduction

Geosynthetic materials are now commonly used in landfills for many applications such as:

- Geomembranes used as primary liners as barriers to leachate and landfill gas escape.
- Geotextiles used as separation layers, filter layers and as geomembrane protectors.
- Geosynthetic Clay Liners (GCLs) used as primary or secondary liners.
- Geonets and geocomposites used as leachate, landfill gas and groundwater drainage layers.
- Geogrids used for reinforcing applications.

The stability of a geosynthetic landfill lining system is controlled by the shear strength between the various interfaces, i.e. geosynthetic/geosynthetic and geosynthetic/soil interface shear strengths. This paper considers the stability of geosynthetics on landfill side slopes and in sloping capping applications by presenting a summary of available interface shear strength values from the literature, supplemented by testing carried out at The Nottingham Trent University. Design methods promoted by various authors are discussed and modifications suggested.

## Background

The shear strength developed at a geosynthetic interface is dependent on both the normal stress applied to the interface and the displacement at the interface. Several authors (e.g. Seed *et al.*, 1988, Byrne 1994 etc.) have indicated that most geosynthetic interfaces are strain softening, i.e. they exhibit a reduction in shear

*Geotechnical engineering of landfills.* Thomas Telford, London, 1998

stress at displacements beyond peak strengths. Typically for each normal stress, the shear stress increases from the origin with increasing displacement until a peak value is achieved. Subsequent displacement results in a reduction in shear stress to a constant or residual value.

If the peak and residual strengths are plotted against the relevant normal stresses, the resulting failure envelope can be defined. A linear Coulomb-type failure envelope is usually obtained which defines the interface shear strength in terms of the friction angle ($\delta$) and cohesion intercept ($\alpha$). It should be noted that these parameters only define the failure envelope for the range of normal stresses tested and that extrapolation of both friction angle and cohesion intercept outside the range may not be representative. These interface shear strength parameters can be used to assess the stability of any slope containing a geosynthetic, using a conventional soil mechanics approach.

## Measurement of interface shear strength

The measurement of geosynthetic interface shear strength can be carried out by three main methods; direct shear testing, ring shear testing and testing with a tilting table. Direct shear testing can be carried out in standard soil shear boxes with dimensions of 60 mm x 60 mm and 100 mm x 100 mm which can be regarded as index testing, or can be more performance-related using larger 300 mm x 300 mm and 300 mm x 400 mm direct shear apparatus. All direct shear apparatus have limited displacements and it has been shown (Jones, 1998) that even displacements of 100 mm may not mobilise the true residual interface shear strengths.

Ring shear testing can be carried out to investigate the true residual strengths since the apparatus can produce unlimited displacements. It should be recognised, however, that the direction of shearing in a ring shear test is not comparable to the field and thus true residual shear strengths may only be of academic interest and the large strain strengths obtained from a direct shear test in a 300 mm x 400 mm apparatus may be sufficient for design applications. In addition, ring shear testing should not be used to measure peak interface shear strengths (Dixon & Jones, 1995).

The third main method of measurement is the use of a tilting table which has been used predominantly in Europe. There is currently no consensus on the size of apparatus required to provide performance results and its use is limited to low normal stresses. It may be, however, that the tilting table may be more accurate in determining the behaviour of geosynthetic interfaces at low confining stress.

## Interface shear strength values

The following paragraphs summarise a literature search carried out to investigate the range of shear strengths published for various geosynthetic interfaces. The results of the literature search have been supplemented by over 200 direct shear tests carried out at The Nottingham Trent University (Jones, 1998). Peak and residual shear strengths have been plotted against the appropriate normal stress (Figures 1, 2 and 3) and linear regression has been used to generate the failure envelope for each interface. The peak and residual shear strength envelopes are given, together with the correlation coefficient ($R^2$) which gives a statistical determination of whether the assumed linear regression is strong; a perfect straight line fit giving an $R^2$ value of 1.0.

## Smooth HDPE geomembrane

The results of testing on smooth HDPE geomembranes are presented in Figure 1 and a summary is given in Table 1 below.

| Interface | Interface shear strength parameters | | | | | |
|---|---|---|---|---|---|---|
| | Peak | | | Residual | | |
| | $\delta$ (°) | $\alpha$ (kPa) | $R^2$ | $\delta$ (°) | $\alpha$ (kPa) | $R^2$ |
| Geonet | 9.0 | 1.0 | 0.74 | 6.9 | 1.8 | 0.80 |
| Non-woven geotextile | 9.8 | -0.8 | 0.88 | 5.8 | 0.3 | 0.88 |
| Sand | 26.9 | -4.0 | 0.90 | 16.2 | 0.0 | 0.95 |
| Clay - undrained | 10.3 | 7.1 | 0.48 | 2.3 | 15.0 | 0.09 |
| Clay - drained | 21.5 | 2.1 | 0.86 | 17.1 | -6.1 | 0.97 |

**Table 1** Summary of results for smooth HDPE geomembrane

The summary plot of shear stress vs. normal stress for a smooth geomembrane/geonet interface (Figure 1a) shows a scatter in data points with a poor straight line fit for both peak and residual conditions with $R^2$ values of 0.74 and 0.80 respectively. This linear regression gives a peak friction angle of 9.0°, which reduces to 6.9° at large displacements. This interface has low cohesion intercepts for both peak (1.0kPa) and residual (1.8kPa) conditions. For the smooth geomembrane/non-woven geotextile interface, a peak interface friction angle of 9.8°, reducing to 5.8° for residual conditions (Figure 1b) is calculated; there is negligible cohesion intercept for this interface. Both peak and residual conditions give strong straight line fits both with correlation coefficient values of 0.88, however there is still a degree of scatter in the results (Figure 1b).

The smooth geomembrane/sand interface has much higher shear strength than the two interfaces discussed above. The peak interface shear strength using linear regression is $\delta$ = 26.9° and $\alpha$ = -4.0 kPa, and there is a good straight line fit with $R^2$ = 0.90 (Figure 1c). The residual values give slightly less scatter and thus a higher correlation coefficient of 0.95, and a residual friction angle of 16.2°.

Testing of the interface shear strength between geosynthetics and cohesive soil is more difficult than the testing of geosynthetic/geosynthetic or geosynthetic/granular interfaces since there is the possibility of pore water pressures at the interface during shearing. Such pressures may be positive or negative (suctions) and will lead to a decrease or increase in effective stress at the interface thus making the assessment of interface shear strength more difficult. The assessment of whether the results quoted in the literature are based on undrained or drained conditions is based on either the various authors' descriptions or on an interpretation of the shearing rates used by the current authors. It is considered that the results presented may not be true undrained or drained conditions and thus caution is required when assessing the results.

For undrained tests it may be that the interface shear strength will be dependent on the undrained shear strength of the clay. However, not all authors reported the clay strength and this makes any accurate assessment of the results difficult if not impossible. The scatter in results for smooth HDPE geomembrane/clay interface (Figure 1d) is not unexpected. Correlation coefficients of 0.48 and 0.09 for the peak and residual envelopes respectively demonstrate this scatter. There is a clear increase in shear strength with increasing normal stress with a peak interface shear strength parameters of $\delta$ = 10.3° and $\alpha$ = 7.1 kPa. However, the friction angle of the residual envelope is negligible ($\delta$ = 2.3°) and the cohesion intercept is 15.0 kPa.

For the drained case the smooth geomembrane/clay interface has less scatter than the undrained conditions (Figure 1e). This may be associated with no pore pressures at the interface or may be due to the lower number of data points available. Both peak and residual envelopes have strong correlation coefficients of 0.86 and 0.97 respectively, and the peak interface friction angle of 21.5° reduces to a residual value of 17.1°. The cohesion intercept reduces from 2.1 kPa for the peak to -6.1 kPa for the residual shear strength. Since the residual envelope is only based on four data points it is not considered to be representative.

## Textured HDPE geomembrane

The results of testing on textured HDPE geomembranes are presented in Figure 2 and a summary is given in Table 2 below.

| Interface | Interface shear strength parameters | | | | | |
|---|---|---|---|---|---|---|
| | Peak | | | Residual | | |
| | $\delta$ (°) | $\alpha$ (kPa) | $R^2$ | $\delta$ (°) | $\alpha$ (kPa) | $R^2$ |
| Geonet | 11.0 | 3.0 | 0.98 | 9.1 | 9.2 | 0.96 |
| Non-woven geotextile | 25.8 | 6.9 | 0.88 | 13.1 | 3.6 | 0.88 |
| Sand | 27.4 | 6.9 | 0.96 | 25.5 | 15.5 | 0.90 |
| Clay - undrained | 4.4 | 36.0 | 0.13 | 3.1 | 34.0 | 0.21 |
| Clay - drained | 10.7 | 26.7 | 0.93 | - | - | - |

**Table 2** Summary of results for textured HDPE geomembrane

The information available on the interface shear strength between textured HDPE geomembranes and geonets is limited and this may be because the increase in interface shear strength over and above the smooth geomembrane is marginal. Figure 2a summarises the available information, although there are only five data points for the peak strength and three points for the residual strength. The peak interface shear strength based on this data is $\delta = 11.0°$ and $\alpha = 3.0$ kPa with a correlation coefficient of 0.98, which compares with a friction angle of 9.0° for the smooth geomembrane case (Figure 1a). The residual interface shear strength for the textured geomembrane ($\delta = 9.1°$ and $\alpha = 9.2$ kPa) needs to be treated with care since it is only based on three data points.

The majority of data presented for the shear strength of textured geomembrane/non-woven geotextile interfaces are from the results of the testing carried out by the authors (Jones & Dixon, 1998), although other information from the literature has been used to develop Figure 2b. A peak friction angle of 25.8° is obtained together with a cohesion intercept of 6.9 kPa, which reduces to residual values of $\delta = 13.1°$ and $\alpha = 3.6$ kPa, although there is a significant range of values with $R^2$ values of 0.88 for both the peak and residual case.

The interface shear strength results for the textured geomembrane/sand interface are shown on Figure 2c which give peak parameters of $\delta = 27.4°$ and $\alpha = 6.9$ kPa with a correlation coefficient of 0.96. This interface, although strain softening, does not seem to exhibit a large reduction in shear strength with increased displacement since the residual friction angle is 25.5° with a relatively high cohesion intercept of 15.5 kPa.

From the results of undrained tests on textured HDPE geomembrane against clays (Figure 2d), it can be seen that the dependency of shear strength on normal stress is limited with peak and residual friction angles of 4.4° and 3.1° respectively. Cohesion intercepts for both peak and large strain conditions are similar with a peak value of 36.0 kPa and a residual value of 34.0 kPa, however both envelopes give poor linear relationships with $R^2$ values of 0.13 and 0.21. The shape of the envelopes suggest that the shear strength between textured geomembrane and a clay tested without an allowance for the dissipation of pore pressures is almost independent of normal stress, and is likely to be related to the undrained shear strength of the clay. Since the data shown on Figure 2d has been obtained from eight separate references with different clay at different remoulding conditions, the extent of the data scatter is not surprising.

The results shown on Figure 2d compare well with the observations made by Orman (1994), who found that failure of a textured HDPE geomembrane/silt interface occurred within the silt along the line of the asperities on the geomembrane sheet. Thus it is to be expected that the undrained interface shear strength of a textured geomembrane/clay is independent of normal stress and probably equal to the undrained shear strength of the clay.

There is little information on geomembrane/clay interfaces tested at strain rates slow enough to dissipate pore waters pressures although the available data indicates that the shear strength of this interface is dependent on normal stress (Figure 2e). Again the small amount of data available means that caution is required when analysing the results, however, linear regression gives a peak interface shear strength corresponding to $\delta = 10.7°$ and $\alpha = 26.7$ kPa. Closer inspection of the plot reveals that a non-linear fit may be more representative for the peak shear strength envelope, possibly curving downwards at lower normal stresses and passing through the origin. There is insufficient data to determine the residual shear strength for this interface, however, it is likely that the residual interface shear strength will be the residual shear strength of the clay. The asperities of the textured geomembrane are very similar to the upper sintered brass platten on the standard Bromhead ring shear apparatus (Bromhead 1979).

## Non-woven geotextile

The results of testing on non-woven geotextiles are presented in Figure 3 and a summary is given in Table 3 below.

| Interface | Interface shear strength parameters | | | | | |
|---|---|---|---|---|---|---|
| | Peak | | | Residual | | |
| | $\delta$ (°) | $\alpha$ (kPa) | $R^2$ | $\delta$ (°) | $\alpha$ (kPa) | $R^2$ |
| Geonet | 13.1 | 17.9 | 0.76 | 15.4 | 4.1 | 0.92 |
| Gravel | 35.0 | -1.0 | 0.87 | 19.9 | 30.1 | 0.99 |
| Sand | 33.0 | -1.3 | 0.93 | 28.7 | 7.7 | 0.92 |
| Clay - undrained | 25.3 | 5.3 | 0.91 | 17.7 | 55.6 | 0.98 |
| Clay - drained | 32.5 | 4.4 | 0.98 | - | - | - |

**Table 3** Summary of results for non-woven geotextile

The results of shear strength testing on non-woven geotextile/geonet interfaces are plotted in Figure 3a and linear regression of all the data points give peak interface shear strengths of $\delta = 13.1°$ and $\alpha = 17.9$ kPa with an $R^2$ value of 0.76. For the range of normal stresses considered, the residual envelope is similar to the peak in terms of its mobilised shear strength, however the friction angles and cohesion intercept are different. The best fit line through the residual data points is given by $\delta = 15.4°$ and $\alpha = 4.1$ kPa, i.e. a higher friction angle but a lower cohesion intercept with a correlation coefficient of 0.92.

The non-woven geotextile/gravel interface has a high shear strength with some values in the literature reported as high as 48°. Most of the results available are for tests carried out at normal stresses less than 200 kPa (Figure 3b) and linear regression gives a friction angle of 35.0° with a cohesion intercept of -1.0 kPa. This reduces to a residual shear strength corresponding to $\delta = 19.9°$ and $\alpha = 30.1$ kPa. The peak shear strength envelope shows a reasonable strong straight line fit with a correlation coefficient of 0.94, while the residual envelope has a very strong fit with $R^2 = 0.99$, however the residual is based on a small number of data points.

There is much more information available in the literature on the interface shear strength between sand and non-woven geotextiles, and this is also a high strength interface with a peak friction angle of 33.0° and a cohesion intercept of -1.3 kPa (Figure 3c). The residual shear strength for this interface is

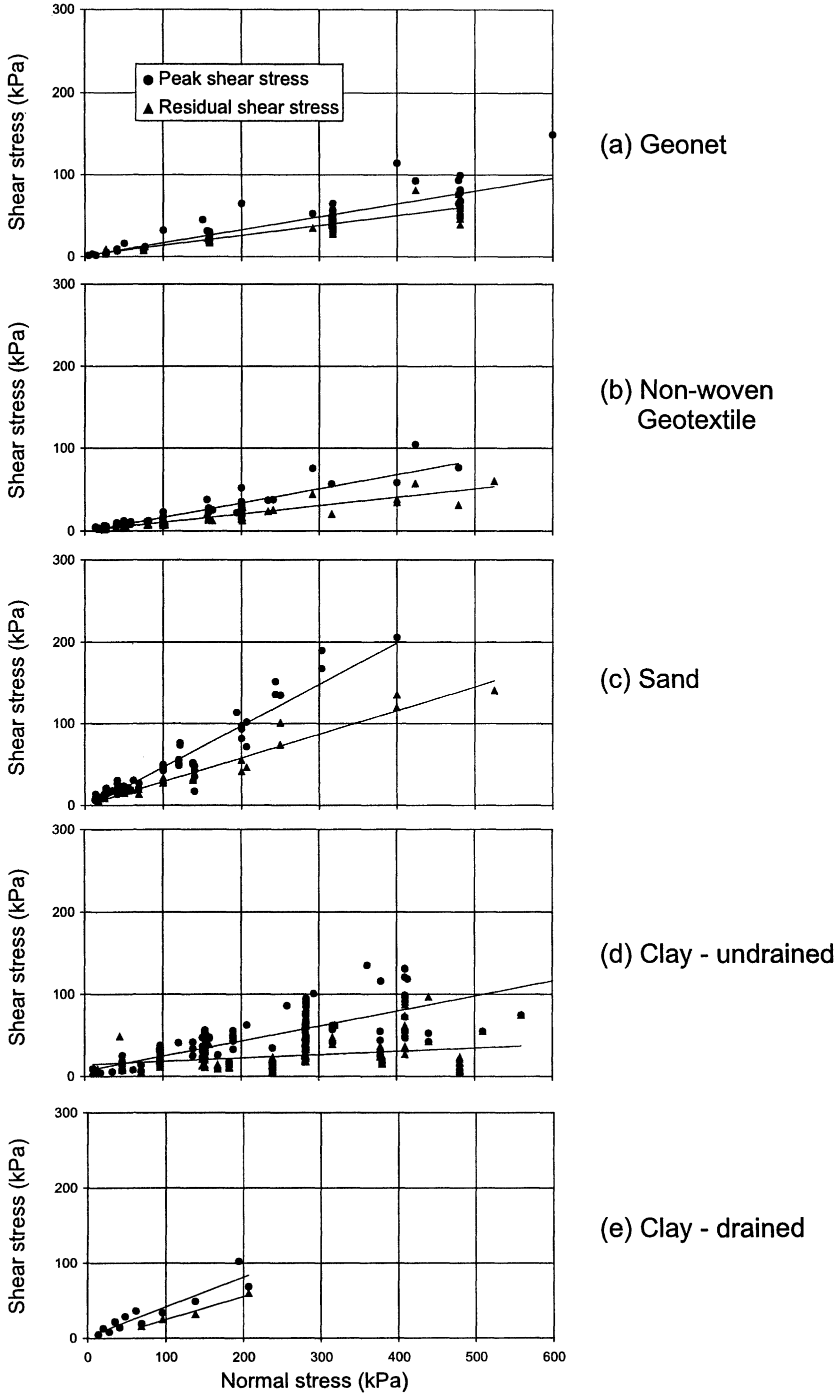


**Figure 1** Smooth HDPE Geomembrane

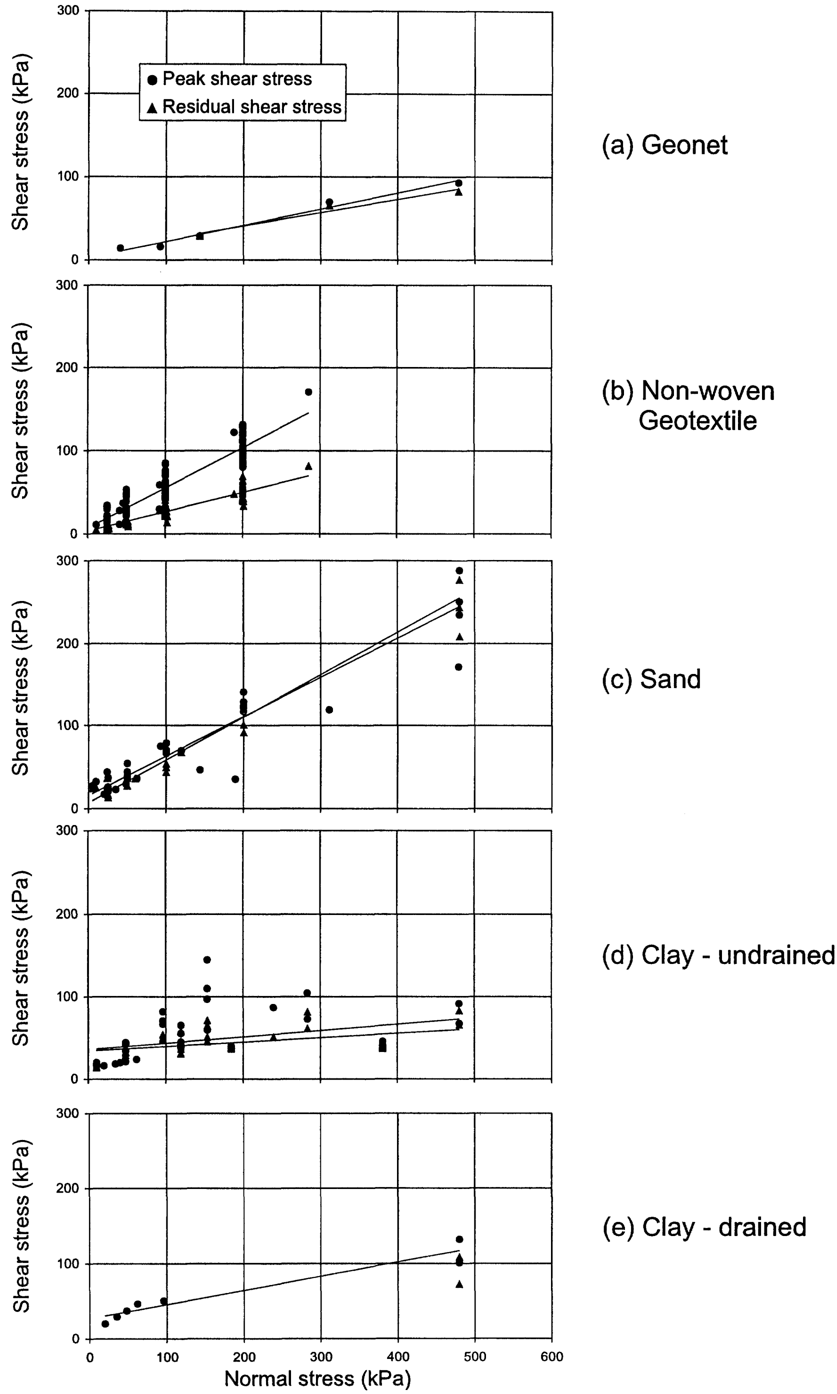


**Figure 2** Textured HDPE Geomembrane

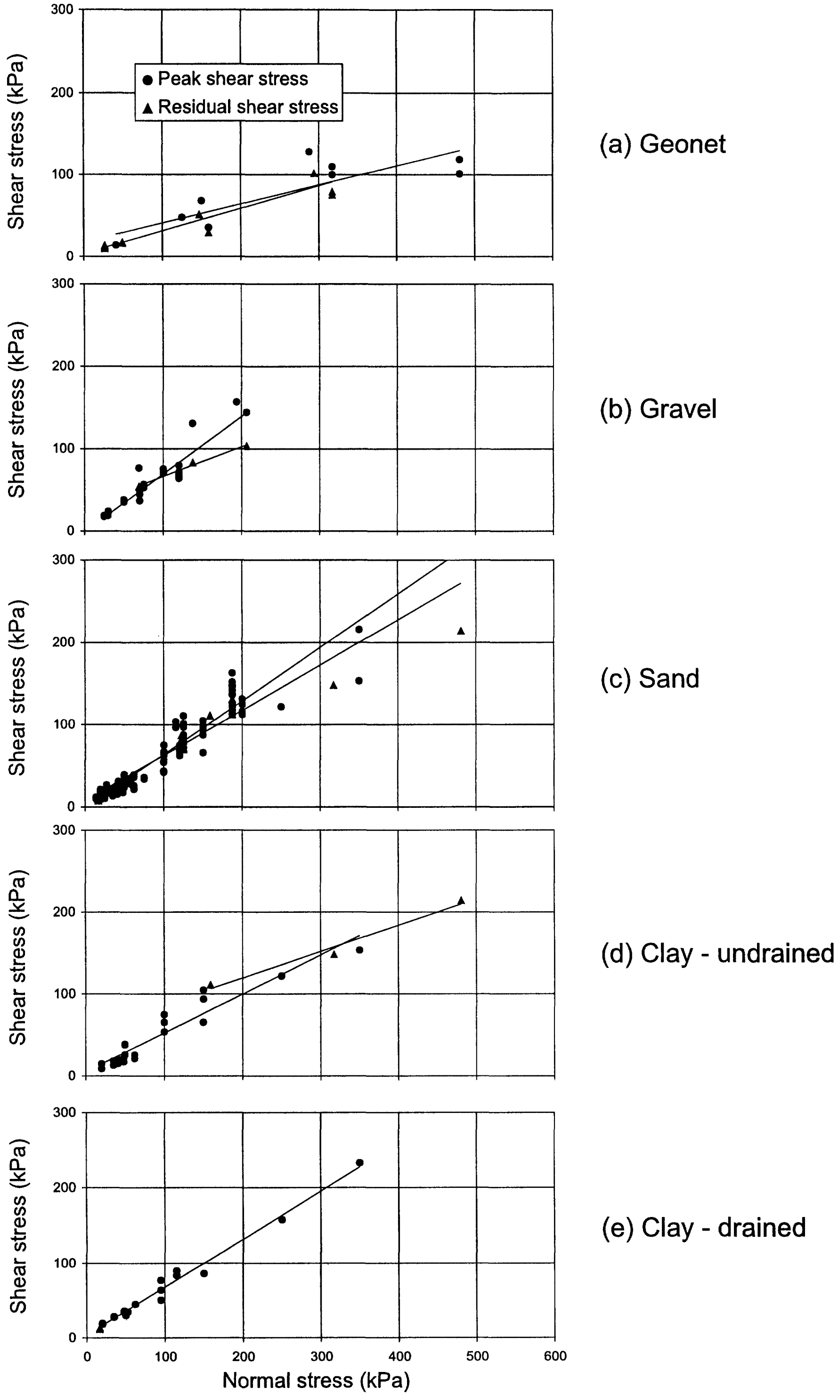


**Figure 3** Non-woven Geotextile

reduced to a value of $\delta$ = 28.7° and $\alpha$ = 7.7 kPa. The peak interface shear strength envelope has been generated from over a hundred data points and the scatter is minimal with an $R^2$ value of 0.91. Less data was available for the residual plot, however the amount of scatter is less with a correlation coefficient of 0.98.

The results of undrained tests on non-woven geotextile/clay interface shown on Figure 3d. Peak interface shear strengths of $\delta$ = 25.3° and $\alpha$ = 5.3 kPa are obtained with a correlation coefficient of 0.91, which reduce to $\delta$ = 17.7° and $\alpha$ = 55.6 kPa for large strains. The residual envelope is based on three data points, has an extremely high cohesion intercept and has an $R^2$ value of 0.98. The peak interface shear strength is predominantly frictional in nature however the high cohesion intercept of the residual envelope could be indicative of dependence on the undrained shear strength of the clay. In particular it may be that the failure plane exists in the outer layer of the geotextiles' fibres which are clay filled, and thus the shear strength is a combination of the fibres' frictional (and possibly tensile) strength together with the clay's strength.

A higher shear strength is obtained for drained tests on non-woven geotextile/clay interfaces, as shown on Figure 3e. The summary plot of all data points gives a good straight line fit ($R^2$ = 0.98) for the peak interface shear strength with a high friction angle of 32.5° and a cohesion intercept of 4.4 kPa. There is insufficient information to generate a residual interface shear strength envelope.

## Overview of stability analysis from the literature

In considering the stability of a slope lined with geosynthetics, several failure mechanisms need to be assessed. Conventional limit equilibrium methods such as Bishop (1955) and Janbu (1973) or approximate methods such as the charts proposed by Taylor (1937) can be used to assess the overall stability of the host slope. The use of geosynthetics often introduce potentially weak planes into the system and require special consideration.

The stability of a cover soil above the geosynthetics was discussed by Martin & Koerner (1985), and using an infinite slope approach presented the factor of safety against the failure of a uniform cover soil as:

$$F = \frac{\tan \delta}{\tan \beta} \qquad \text{Equation 1}$$

where $\delta$ is the friction angle between the geosynthetic and cover soil,
$\beta$ is the slope angle.

The above equation applies when the cover soil is dry or subjected to an external hydrostatic water pressure distribution. However, such conditions where there is external water pressures are normally restricted to ponds and reservoirs, and it is more useful to consider active seepage in the cover soil. For full depth seepage, Martin & Koerner (1985) suggest an approach based on a reduction in effective normal stress on the liner, i.e.

$$F = \frac{\gamma_b \tan\delta}{\gamma_s \tan\beta} \qquad \text{Equation 2}$$

where $\gamma_b$ is the buoyant unit weight of cover soil
$\gamma_s$ is the saturated unit weight of cover soil

Note that $\gamma_b = \gamma_s - \gamma_w$, where $\gamma_w$ is the unit weight of water. This is a conservative approximation and assumes that the water pressures are calculated using vertical depth below ground level.

Giroud & Beech (1989) give two reasons why a finite slope is more stable than an infinite slope assumed in the analysis method described above; the presence of a geosynthetic anchorage at the crest, and the buttressing effect of the soil at the base of the slope. As slippage along the critical geosynthetic interface occurs, tensile forces are generated in the geosynthetics above the critical interface, and these tensile forces contribute to the stability of the potential sliding block. The authors summarise the three factors contributing to the lining's stability as:

- Geosynthetic tension resulting from the crest anchorage.
- Shear resistance developed along the interface.
- Toe buttressing effect.

In their limiting equilibrium method, Giroud & Beech (1989) proposed dividing the system into two wedges and forces that are balanced in the vertical and horizontal directions. This method provides two equilibrium equations and three unknowns, and an iterative process is required to provide a solution. A major drawback with this method is that the distribution of tensile stresses within the geosynthetic layers cannot be determined. Koerner & Hwu (1991) proposed a limiting equilibrium method also based on the two part wedge method, and considered sliding of the active wedge to be resisted by only the shear strength along the geosynthetic/cover soil interface and the passive soil wedge buttress at the toe of the slope. The factor of safety (F) with respect to sliding of the system is a solution of the following quadratic equation:

$$aF^2 + bF + c = 0 \qquad \text{Equation 3}$$

where,

$$a = \frac{\gamma h L}{2}\sin^2(2\beta) \qquad \text{Equation 4}$$

$$b = -[\gamma h L\cos^2\beta\tan\delta_u\sin(2\beta) + \alpha_u L\cos\beta\sin(2\beta) + \gamma h L\sin^2\beta\tan\phi\sin(2\beta) + 2ch\cos\beta + \gamma h^2\tan\phi] \qquad \text{Equation 5}$$

$$c = (\gamma h L\cos\beta\tan\delta_u + \alpha_u L)(\tan\phi\sin\beta\sin(2\beta)) \qquad \text{Equation 6}$$

and,

| | |
|---|---|
| $\gamma$ | unit weight |
| h | thickness of cover soil (measured perpendicular to slope) |
| L | slope length |
| $\beta$ | slope angle |
| $\phi$ | angle of internal friction of cover soil |
| c | cohesion of cover soil |
| $\delta_u$ | interface friction angle at the upper interface |
| $\alpha_u$ | apparent cohesion at upper interface |

This approach assumes that the factor of safety is the same value at every point along the sliding surface defined by the two wedge mechanism. By default this means that the factor of safety is the same with respect to the shearing resistance at the active wedge/geosynthetic interface as that with respect to the shearing resistance of the cover soil beneath the passive wedge. Koerner & Hwu (1991) further proposed a model to assess the tension in a geosynthetic due to unbalance interface shear forces. By assuming uniform mobilisation of the interface shear strengths along the geomembrane, they developed an expression for the tensile force per unit width of slope as follows:

$$T = \left[(\alpha_u - \alpha_l) + \gamma h \cos\beta(\tan\delta_u - \tan\delta_l)\right].L \qquad \text{Equation 7}$$

where,

| | |
|---|---|
| $\delta_l$ | interface friction angle at the lower interface |
| $\alpha_l$ | apparent cohesion at lower interface |

This equation expresses the imbalance between the maximum shear force that can act at the geosynthetic upper interface and the maximum shear force at the lower interface. When the upper shear force is smaller than the force at the lower surface the geosynthetic is in equilibrium and is not stressed. However, when the upper shear force is greater than the lower, a tensile force T is required in the geomembrane to ensure equilibrium. A major shortcoming with this method is that the tensile force computed is independent of the level of shear stress effectively mobilised at the upper interface. The shear force at the upper interface in this equation should be the mobilised shear force. Bourdeau *et al.* (1993) proposed a coupling between Equations 3 and 7 by replacing the ultimate upper shear strength with a mobilised value calculated by dividing the ultimate value by the factor of safety calculated in Equation 3, i.e.

replacing $\alpha_u + \gamma h \cos\beta \tan\delta_u$

by $\dfrac{\alpha_u + \gamma h \cos\beta \tan\delta_u}{F}$

which gives a new expression for the tensile force in the geosynthetic:

$$T = \left[\left(\frac{\alpha_u}{F} - \alpha_l\right) + \gamma h \cos\beta\left(\frac{\tan\delta_u}{F} - \tan\delta_l\right)\right].L \qquad \text{Equation 8}$$

For a multi-layered system, the limit method proposed by Koerner (1990) can be used to determine the tensile forces in subsequent lower layers. This is a force equilibrium procedure which balances forces in the direction parallel to the slope. The shear force mobilised in the upper surface of a geosynthetic is transferred to its lower surface by shear until the maximum shear strength of that interface has been reached, and the remaining force will be taken in tension in the geosynthetic.

The above methods do not consider the effect of seepage forces on the stability of a cover soil. Soong & Koerner (1995) have developed a model that considers seepage flow parallel to the slope, i.e. a flow net within the cover soil mass consists of flow lines parallel to the slope and equipotential lines perpendicular to the slope. They produce two models for stability assessment; one for the case of a horizontal seepage build-up and one for a parallel-to-slope seepage build-up. The second model only will be considered below.

The expression developed by Soong & Koerner (1995) for the factor of safety against sliding of a cover soil on a geosynthetic can also be represented by a quadratic equation (Equation 3) with the following constants:

$$a = W_A(\sin\beta)(\cos\beta) - U_h(\cos^2\beta) + U_h \qquad \text{Equation 9}$$

$$b = -W_A(\sin^2\beta)(\tan\phi) + U_h(\sin\beta)(\cos\beta)(\tan\phi) - N_A(\cos\beta)(\tan\delta) - (W_P - U_v)(\tan\phi) \qquad \text{Equation 10}$$

$$c = N_A(\sin\beta)(\tan\delta)(\tan\phi) \qquad \text{Equation 11}$$

For the case of parallel-to-slope seepage build-up, the constants in the above equations are given by:

$$W_A = \frac{\left[\gamma_d(h - h_w)(2H\cos\beta - (h + h_w)) + \gamma_{sat} h_w(2H\cos\beta - h_w)\right]}{\sin(2\beta)}$$

$$W_P = \frac{\left[\gamma_d(h^2 - h_w^2) + \gamma_{sat} h_w^2\right]}{\sin(2\beta)}$$

$$U_n = \frac{\left[\gamma_w h_w \cos\beta(2H\cos\beta - h_w)\right]}{\sin(2\beta)}$$

$$U_h = \frac{\gamma_w h_w^2}{2}$$

$$N_A = W_A\cos\beta + U_h\sin\beta - U_n$$

$$U_v = \frac{U_h}{\tan\beta}$$

where $W_A$ = total weight of the active wedge
$W_P$ = total weight of the passive wedge
$U_n$ = resultant of the pore pressures acting perpendicular to the slope
$U_h$ = resultant of the pore pressures acting on the interwedge surfaces
$U_v$ = resultant of the vertical pore pressures acting on the passive wedge
$N_A$ = effective force normal to the failure plane of the active wedge
$\gamma_d$ = dry unit weight of the cover soil
$\gamma_{sat}$ = saturated unit weight of the cover soil
$h_w$ = thickness of saturated cover soil (measured perpendicular to slope)

It should be noted that for the case of parallel-to-slope seepage build-up, the ratio of $h_w/h$ can be defined by the parallel submergence ratio, PRS.

## Proposed stability analysis methodology

Soong & Koerner (1995) consider a granular cover soil with an internal friction angle of $\phi$, and in the consideration of seepage forces this is satisfactory. In addition, the interface shear strength between the upper geosynthetic and the cover soil is only represented by a friction angle ($\delta$). In an attempt to make this approach more generic, the effect of a cover soil with cohesion (c) and an interface with a cohesion intercept of $\alpha$, the equations have been re-written to include these terms. The inclusion of these parameters will change the b and c terms in the quadratic equation as follows:

$$b = -\left[W_A \sin^2\beta\tan\phi\right] + \left[U_h \sin\beta\cos\beta\tan\phi\right] - \left[\cos\beta((\alpha L) + N_A\tan\delta)\right] - \left[(W_P - U_v)\tan\phi\right] - \left[\frac{ch}{\sin\beta}\right]$$

Equation 12

$$c = \sin\beta\tan\phi[\alpha L + N_A\tan\delta]$$

Equation 13

Further, the stress normal to the interface used in the calculation of the geosynthetic tensile force (Equation 8) should take account of the piezometric surface. This equation now becomes:

$$T \quad = \quad \left[\left(\frac{\alpha_u}{F} - \alpha_l\right) + \left(\gamma_{sat} h_w + \gamma_d (h - h_w)\right) \cos\beta \left(\frac{\tan\delta_u}{F} - \tan\delta_l\right)\right].L$$

Equation 14

It is proposed that the stability of a cover soil over several layers of geosynthetics together with the tension developed in the geosynthetics can be established as follows:

1. Calculate the factor of safety against cover soil sliding using the approach of Soong & Koerner (1995), modified to allow for c and $\alpha$.
2. Calculate the mobilised tension in the upper geosynthetic using Bourdeau *et al.* (1993) with the modification for $\gamma_{sat}$ and $\gamma_d$.
3. Calculate the mobilised tension in the remaining geosynthetics.

## Example 1

This methodology is used in the following example. Consider the stability of a landfill capping system comprising 1 m of gravely cover soil resting on a non-woven geotextile protection over a 1mm thick smooth HDPE geomembrane. A blinding layer of sand has been placed beneath the geomembrane. The maximum slope height is 20 m and the slope gradient is 1:3 (18.4°). The following internal strengths and interface shear strengths (obtained from Tables 1, 2 and 3) apply:

| | |
|---|---|
| Cover soil: | $\phi = 35°$, $c = 0$ kPa |
| Cover soil/geotextile: | $\delta = 35°$, $\alpha = 0$ kPa |
| Geotextile/smooth geomembrane: | $\delta = 10°$, $\alpha = 0$ kPa |
| Smooth geomembrane/sand: | $\delta = 27°$, $\alpha = 0$ kPa |

The cover soil has a dry unit weight of 18 kN/m$^3$, and a saturated unit weight of 21 kN/m$^3$. Consider a case of a parallel submersion ratio of 0.25.

The length of the slope is given by:

$$L \quad = \quad \frac{H}{\sin\beta} \quad = \quad \frac{20}{\sin 18.4} \quad = \quad 63.36 \text{ m}$$

Also, the height of water in the cover soil (perpendicular to the slope) is:

$$h_w \quad = \quad PSR \times h = \quad 0.25 \times 1.0 \quad = \quad 0.25 \text{ m}$$

1. Calculate the factor of safety against sliding

First calculate the constants:

$$W_A = \left[\frac{18(1.0-0.25)(2x20\cos 18.4-(1.0+0.25))+21x0.25(2x20\cos 18.4-0.25)}{\sin(2x18.4)}\right]$$

$$W_A = \left[\frac{495.52+197.95}{0.599}\right] = 1157.71 \text{ kN}$$

$$W_P = \left[\frac{18(1^2-0.25^2)+21x0.25^2}{0.599}\right] = 30.36 \text{ kN}$$

$$U_n = \left[\frac{10x0.25\cos 18.4(2x20\cos 18.4-0.25)}{0.599}\right] = 149.32 \text{ kN}$$

$$U_h = \frac{10x0.25^2}{2} = 0.31 \text{ kN}$$

$$N_A = 1157.71\cos 18.4 + 0.31\sin 18.4 - 149.32 = 949.30 \text{ kN}$$

$$U_v = \frac{0.31}{\tan 18.4} = 0.93 \text{ kN}$$

From Equation 9:

$$a = 1157.71\sin 18.4\cos 18.4 - 0.31\cos^2 18.4 + 0.31$$

$$a = 346.78$$

From Equation 12:

$$b = -[1157.71\sin^2 18.4\tan 35] + [0.31\sin 18.4\cos 18.4\tan 35] - [\cos 18.4(0 + 949.30\tan 35)] - [(30.36 - 0.93)\tan 35] - [0]$$

$$b = -732.03$$

From Equation 13:

$$c = \sin 18.4\tan 35[0 + 949.30\tan 35]$$

$$c = 146.91$$

Now calculate factor of safety from:

$$F = \frac{-b+\sqrt{b^2-4ac}}{2a}$$

$$F = \frac{-(-732.03)+\sqrt{(-732.03)^2-4x346.78x146.91}}{2x346.78}$$

$$F = \frac{732.03+576.27}{693.56}$$

$$F = 1.89$$

2. Calculate mobilised tension in upper geosynthetic (geotextile)

From Equation 14:

$$T = \left[(0-0)+(21\text{x}0.25+18(1-0.25))\cos 18.4\left(\frac{\tan 35}{1.89}-\tan 10\right)\right]63.36$$

$$T = \left[0+17.79\left(\frac{\tan 35}{1.89}-\tan 10\right)\right]63.36$$

$$T = 218.84 \text{ kN}$$

It is unlikely that the tensile strength of a non-woven geotextile will withstand this tension and it will lead to failure of the geotextile in tension and sliding of the cover soil and geotextile on the geomembrane. There will therefore not be any tension in the geomembrane since failure will occur above it.

## Example 2

Now consider the same case as above but this time the smooth geomembrane is replaced by a textured geomembrane. The relevant interface shear strength parameters are:

Geotextile/textured geomembrane: $\delta = 26°$, $\alpha = 7$ kPa
Textured geomembrane/sand: $\delta = 27°$, $\alpha = 7$ kPa

Since the upper geosynthetic remains the same, the calculated factor of safety remains the same. The tension in the geotextile is obtained from Equation 14:

$$T = \left[(0-7)+17.79\left(\frac{\tan 35}{1.89}-\tan 26\right)\right]63.36$$

$$T = -575.68 \text{ kN}$$

Since T is negative, the shear strength of the lower interface is greater than the mobilised shear stress on the upper interface and there is no tension in the geotextile. The mobilised shear stress is thus transferred from the geotextile to the geomembrane with no tension induced in the geotextile. Now check if there is any tension in the geomembrane:

$$T = \left[\left(\frac{7}{1.89}-7\right)+17.79\left(\frac{\tan 26}{1.89}-\tan 27\right)\right]63.36$$

$$T = [-3.30 - 4.47]63.36$$

$$T = -492.30 \text{ kN}$$

Hence the geomembrane can also transfer the shear stress to the sand below without any tension.

# Discussion

## Interface shear strength

The interface shear strength parameters given in this paper have been taken from technical papers available in the literature, in-house testing carried out by Golder Associates in north America and testing carried out at The Nottingham Trent University. The testing was generally carried out in direct shear apparatus of varying size, together with ring shear testing to obtain some of the residual shear strength parameters. The geosynthetics and soils used in the testing vary widely and caution should be exercised when using the data presented in Tables 1 to 3. It is suggested that these values may be used in preliminary designs, however the authors stress the importance of site specific performance testing. In particular, the mean values of friction angle and cohesion intercept presented are taken from tests carried out at normal stresses over a range up to 600 kPa. The values presented in this paper may not be reliable for the design of landfill capping systems and other applications with low normal stresses.

The friction angle and cohesion intercept obtained from any interface shear strength testing are simply parameters that describe the failure envelope for the range of normal stresses used. In other words, they describe the position of the best fit line through the data. A reported cohesion intercept does not necessarily imply that there is a shear strength under zero normal load, although some interfaces, e.g. textured geomembrane/non-woven geotextile and internal strength of geocomposites, will have an actual strength at zero load due to either the mingling of geotextile fibres within the asperities of the geomembrane or from bonding between various layers of a geocomposite. It is up to the judgement of the engineer as to what allowance is made for the cohesion intercept in a design situation.

## Stability analysis

The method presented in this paper expands on the work of others as described above. It may be for the case of capping systems that this simple limiting equilibrium method will give satisfactory results. In the case of a landfill side slope, however, the settlement of the waste will induce displacements at the interfaces. In order to model these conditions, numerical techniques can be used (Jones, 1998) to quantify the mobilised shear stresses in the system. If such analyses cannot be justified then the authors would recommend that the design engineer uses peak interface shear strength values on the base on the landfill only, and that consideration should be given to using residual shear strengths along the side slopes.

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## References

Bishop, A.W. (1955). The use of the slip circle in the stability analysis of earth slopes, *Geotechnique*, 5, 7-17.

Bourdeau, P.L., Ludlow, S.J. & Simpson, B.E. (1993). Stability of Soil-Covered Geosynthetic-Lined Slopes: A Parametric Study, *Proc. Geosynthetics '93*, Vancouver, Canada, 1511-1521.

Bromhead, E.N. (1979). A simple ring shear apparatus, *Ground Engineering*, 12, 5, 40-44.

Byrne, R.J. (1994). Design issues with strain softening interfaces in landfill liners, *Proc. Waste Tech '94*, Charleston, South Carolina, USA, Session 4, Paper 4.

Dixon, N. & Jones, D.R.V. (1995). Discussion on "Landfill liner interface strengths from torsional-ring shear tests", *Journal of Geotechnical Engineering*, ASCE, 121, 6, 509-510.

Giroud, J.P. & Beech, J.F. (1989). Stability of Soil Layers on Geosynthetic Lining Systems, *Proc. Geosynthetics '89*, San Diego, USA.

Janbu, N (1973). Slope stability computations in embankment dam engineering, *Casagrande memorial volume*, eds Hirschfield & Poulos, John Wiley, New York, 47-86.

Jones, D.R.V. (1998). The stability of geosynthetic landfill lining systems, PhD thesis in prep., The Nottingham Trent University.

Jones, D.R.V. & Dixon, N. (1998). Shear strength properties of geomembrane/ geotextile interfaces, *Geotextiles and Geomembranes* (in press.)

Seed, R.B., Mitchell, J.K. & Seed, H.B. (1988). Slope stability failure investigation: Landfill Unit B-19, Phase 1-A, Kettleman Hills, California, *Report No. UCB/GT/88-01*, Dept. of Civil Engineering, University of California, Berkeley.

Koerner, R.M. (1990). *Designing with Geosynthetics*, Second Edition, Prentice Hall, Englewood Cliffs, NJ.

Koerner, R.M. & Hwu, B-L. (1991). Stability and tension considerations regarding cover soils in geomembrane lined slopes, *Geotextiles and Geomembranes*, 10, 4, 335-355.

Martin, J.P. & Koerner, R.M. (1985). Geotechnical Design Considerations for Geomembrane Lined Slopes: Slope Stability, *Geotextiles and Geomembranes*, 2, 229-321.

Orman, M.E. (1994). Interface Shear Strength Properties of Roughened HDPE, *Journal of Geotechnical Engineering*, ASCE, 120, 4, 758-761.

Soong, T-Y & Koerner, R.M. (1995). Seepage Induced Slope Instability, *Proc. 9th GRI Conference*, Philadelphia, USA.

Taylor, D.W. (1937). Stability of earth slopes, *J. Boston Soc. Civil Engrs.*, 24, 197-246.

# Performance testing of protection materials for geomembranes

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## Introduction

The wide acceptance of geomembranes as an element of landfill liner or cover systems both in the UK and abroad has driven the need to assess how these relatively thin membranes should be protected in an aggressive environment. A number of protection media are available ranging from natural materials such as sands and silts to synthetic materials such as geotextiles, and indeed recycled waste materials such as mats of shredded tyres. Understanding of how these materials perform and the actual degree of protection required is the subject of much debate although there is little national guidance available on how to design and specify these materials.

This paper considers some of these issues and looks to put the debate in context and provide background on the development of a cylinder test to assess some of the performance characteristics of these materials

## UK experience

### Mineral protectors

The first polymeric geomembranes were installed in the UK in the 1980s mainly on the back of experience in Germany and the United States. The importance of protecting the membranes was recognised but was perhaps overshadowed to some extent by the technical difficulties associated with installing the geomembrane itself. As membranes become a familiar sight on many landfills more attention is now being paid to protecting what is after all a costly investment. Regulators, designers and operators have also become more aware of how geomembranes can be damaged during construction of the liner or capping system, during waste or restoration emplacement, and in the long-term by point loads causing conditions leading to potential environmental stress cracking to develop.

The first protection layers placed over the early geomembranes were generally locally won minerals such as sands, silts and clays. The specification for these materials was generally derived from the geomembrane manufacturers' installation guidelines which tended to specify grain size distribution, grain angularity and the recommended minimum thickness of the layer. Grain size specifications ranged from less than 2 mm to less than 8 mm although it was recognised that the larger grain sizes could potentially cause damage through scratching the upper surface of the liner. It has been recognised that smaller grain sizes are prone to instability when saturated, so clays may be prone to excessive settlements while silts and fine sands may be prone to filter instability, particularly during the construction phase when significant volumes of water may require removal from the site (see Kirschner & Kreit 1993).

Grain shape was generally specified as "non sharp" although further definition of this parameter was rare. This specification often led to difficulties with borderline materials such as crusher run fines which maintained a fine grain size but tended to be angular.

The thickness of the protection layer ranged from 100 mm to 150 mm although greater thickness have been recorded (up to 300mm in some cases). It is apparent that these thicknesses were derived more from construction practice in other engineering fields rather than scientific justification and technical calculation.

The above specifications, so long as the materials were properly installed, probably erred on the side of caution and properly designed mineral protection layers are as acceptable today as they have always been. There is little doubt however that the inherent variability of natural materials requires careful attention by the designer and within the Construction Quality Assurance Plan (CQAP).

## Geosynthetic protectors

Since the late 1980s non woven geotextiles have won a significant part of the liner protection market used either in combination with a mineral or as a stand alone protector. The drivers behind this change include lower costs due to void space savings, ease and speed of installation (with a lower risk of construction damage to the liner) and the relative homogeneity of the material leading to simpler CQA. As with all manufactured materials there are significant variations between products and unfortunately there is no current industry standard (apart from the cylinder test) that allows the characterisation of these materials with regard to their likely protection performance over their predicted lifespan.

Other geosynthetics used as protectors for geomembranes are geocomposites, geosynthetic clay liners, protection mattresses and mats manufactured from shredded tyres.

There are a number of challenges which the landfill designer has to accept when proposing a geosynthetic as a protector for a polymeric liner, these are:-

i) establishing the required design life for the protector;

ii) establishing the geotechnical design parameters such as vertical and lateral forces, the nature of the point loads, likely shear stresses due to waste settlements (particularly on slopes), and predicting temperature, chemical and biological stresses at the base of a landfill;

iii) the appropriate specification of geotextiles in contract documents (i.e. whether to specify the materials properties such as mass per unit area, CBR value, or any combination of a range of tests; or to give a performance specification which the contractor or supplier has to meet with the proposed materials);

iv) making decisions with regard to the type and frequency of quality assurance testing; and

v) making decisions with regard to the constituents of the geotextile (virgin or recycled fibres not previously used in another product).

## Function of the protective layer

In order to make decisions with regard to testing protectors it is essential to establish the primary and secondary functions of this layer within the overall landfill design. North West Waste Regulation Officers (NWWRO) (1995) provide an overall objective for liners "to protect the liner system from stresses, puncture and penetration from overlying drainage media and waste". Kirschner & Witte (1994) recognises short term dynamic and long term static loadings which could damage geomembranes. German Geotechnical Society (1993) advises that the main role of the protective layer is the "permanent distribution of concentrated stresses on the geomembrane due to the angularity of the drainage blanket, the protective effect of the geotextile, if any, chemical resistance to leachate and resistance to slippage if appropriate".

These various definitions indicate the differing perspectives of designers and regulators and it is essential that the protector is seen as an element in the liner system and it will therefore interact with the materials around it. This suggests that the designer needs not only to have regard for the objectives outlined above but also for site specific geotechnical parameters of the lining system including the interaction of the protector with other materials in the system.

## Testing of protective layers

There are at least three different stages of testing which should be routinely undertaken on any material which is proposed to be included in a landfill engineering project. These tests which can include index and performance tests are:-

i) Design related testing - it is recommended that site specific performance testing is undertaken for the purposes of design. The data collect at this stage will inform the design process and should resolve any site specific questions such as side slope stability and the interaction between different materials in the lining system.

ii) Source quality control and source quality assurance - these tests are undertaken on the proposed materials at source (for example at the factory or quarry) and are designed to provide assurance that the proposed material is uniform and will meet the general performance criteria for the specified purpose.

iii) Construction Quality Assurance - is undertaken at the landfill site and includes conformance testing (checking that the material delivered is the material approved at the design stage) and installation checking and testing (providing assurance that the materials have been installed correctly in accordance with the agreed method).

Testing for quality should normally include all three of the above steps. It is essential to recognise however that the steps are not separate and that information collected at earlier stages, if relevant to the actual material to be used, may fulfil some of the requirement of the later stages. The frequency and nature of testing should be related to the risks associated with the potential failure of the material; if the consequences of failure could lead to significant pollution or significant costs related to remediation then it might reasonably be expected that the testing programme would be more rigorous.

The following list of tests does not include all possible material tests which could be undertaken. Site specific issues may require particular tests; the two basic principles to apply are, will the material perform as desired and has that material been installed to the approved design specification. No attempt has been made in the following list to define test types, the reason for this is that many tests (particularly those for geosynthetics) are likely to change due to developments in Europe where new test protocols have been and are being developed.

## Mineral protective layers

Cohesive protective layers would normally be subject to the following:-

- Compaction tests to determine a compaction curve
- moisture content tests
- consolidation tests
- shear strength tests

Granular protective layers would normally be subject to the following:-

- particle size distribution tests
- chemical compatibility tests
- slake durability tests
- mineralogy tests

- crush strength tests
- tests to assess the angle of friction (saturated if the material is not free draining)

Mineral protectors may need a separator layer between the protection layer and the drainage medium if there is any likelihood of the materials mixing during either the construction or the operation phases of the site.

### Geosynthetic protective layers

Geosynthetic protectors should be subject to the following:-

- penetration tests such as the CBR puncture test
- mass per unit area tests
- carbon black content tests
- static plate tests such as the cylinder test
- tensile tests
- chemical tests
- base polymer type tests
- thickness tests
- cone drop tests
- installation damage tests

## Performance testing of protectors

The primary objective of a protector is to protect the liner during construction, operation and post closure phases of landfill development from stresses, puncture and penetration, ensuring that the pollution protection function of the liner is not compromised. Given this, the designer is faced with the challenge of predicting the nature of, and calculating the magnitude of such stresses, and matching these with a suitable material which will provide an appropriate level of protection with a factor of safety linked, through risk assessment, to the consequences of failure.

Given the current level of understanding of the geomechanics of waste bodies this is no easy task. Many natural and manufactured construction materials have been extensively tested for other applications such as highways and this work is often misapplied to landfill engineering. There remain however a significant number of unknowns which make accurate quantitative design difficult. These include macro scale problems such as predicting the frictional stresses imposed during emplacement, waste degradation and settlement and predicting constructional stresses imposed when installing drainage materials; and micro scale problems such as the magnitude of point stresses developed through protector layers due to static and shear loadings. It is attempting to address these gaps in our understanding of the physical, chemical and biological environment within landfills which has led to the development of performance tests such as those detailed below.

Any performance test is developed from the principle of modelling reality as closely as possible. It is inherent in such tests therefore that they are a simplification of reality and assumptions are necessarily made. The designer should be aware of the assumptions underlying such tests. Basic assumptions include uniform conditions such as temperature, loading rates, waste densities and simplifications of the subgrade. More profound assumptions relate to the rheological behaviour of HDPE geomembranes (e.g. upon which the failure criterion of 0.25% local strain in the cylinder test is based (Sehrbrock, 1993)).

## Field trials

One valuable source of performance data would be the exhumation of protection layers that have been installed in landfills. Geophysical testing of liners using electrical methods (in use since the early 1990s) has demonstrated the damage that can be caused to geomembranes during the installation of mineral protection layers. These tests have shown that installation damage can be caused due to the selection of an unsuitable material (for example by oversized or sharp gravels); and the selection of incorrect plant and machinery either over stressing the geomembrane due to machinery tracking and slewing over thin layers of protection material or causing direct puncture during spreading or placing of minerals. Some of these problems are related to inadequate initial design, poor supervision during the installation of the protector and plant operators not having a full appreciation of how easily geomembranes can be punctured.

There are no records of protectors being systematically tested and assessed after long term burial under significant depth of waste. This is an area that could be explored if wastes were being removed for reasons other than simply to assess the protector as the costs associated with such removal would be significant.

## Trial pads

Trial pads have been constructed on a number of landfills with different protectors used to allow a comparison between different materials. The concept here is to develop a small area of liner using the plant and equipment which will be used during the full scale construction (see Figure 1). The advantage of such trials is that the actual construction materials and plant can be used, theoretically making the test more true to life. It is however difficult to load the pad to reflect the pressures likely to be caused by the overburden of the waste. Undertaking such trials can however be complex, especially if it is intended to compare a number of different materials. One of the main difficulties involved is assessing the stresses on the liner, particularly when relatively small stresses could lead to long-term failure.

**Figure 1** A trial pad under construction

## Laboratory performance tests for geotextiles

To date there are no laboratory scale tests developed to directly assess the efficacy of mineral protectors although there is no reason why some of the principles underlying the cylinder test could not be applied to mineral protectors.

Laboratory performance tests have been developed to test the protection efficiency of geotextiles. The two most common tests used in Europe are derivations of the "Quo Vadis Schutzlagen" (translated as "whither next protection layers") cylinder test and the Austrian pyramidal puncture test. These tests have been developed to provide the opportunity to provide comparable, repeatable tests for assessing geotextile protection efficiency, and are generally based on the principle of applying design loads onto a geotextile/geomembrane combination and assessing either the stress at rupture or using telltales to assess the degree of strain. The principles and methodologies of these tests are well documented elsewhere (see Environment Agency, 1998 and August & Luders, 1997) and therefore it is not intended to cover specific test details in this paper. However, a significant proportion of the literature on the cylinder test has been produced by German researchers and is not readily available in main steam journals. In order to aid a reader who wishes to obtain information on the development of the test, and issues of interpretation of test results, a bibliography containing a number of the key references is included at the end of the paper.

## Development of the Environment Agency cylinder test

The work undertaken by the Quo Vadis Schutzlagen working group in Germany in the late eighties and early nineties led to the development of a cylinder test in which gravels from the proposed drainage layer are placed over a geotextile/geomembrane/metal plate/rubber pad assembly with design loads applied. The telltale plate is then measured to assess the degree of strain which the geomembrane has been exposed to, so that the effectiveness of the geotextile in protecting the geomembrane from stresses can be determined. Figures 2 and 3 show a schematic of the general test arrangement and a view during operation, respectively.

Experience of the test in the UK has generally been positive providing a method of comparing different geotextiles for their suitability as protectors. Problems arose in 1997 as the number of test houses providing the test grew and inconsistencies in the test apparatus, test methodology and reporting of results started to become apparent. In June 1997 one of the test houses decided that action was needed to ensure a greater degree of consistency and it was at this stage that the Environment Agency was invited to provide the regulator's point of view and to hold responsibility for the publication and maintenance of the test protocol until it could be passed to either to British Standard Institution or CEN (European Committee for Standardisation) or it was superseded by another test.

**Legend**

1 applied load
2 upper steel plate
3 sand
4 separator geotextile
5 drainage aggregate
6 geotextile specimen
7 geomembrane
8 metal sheet
9 dense rubber pad
10 lower steel plate
11 load cells
12 cylinder
13 settlement gauge

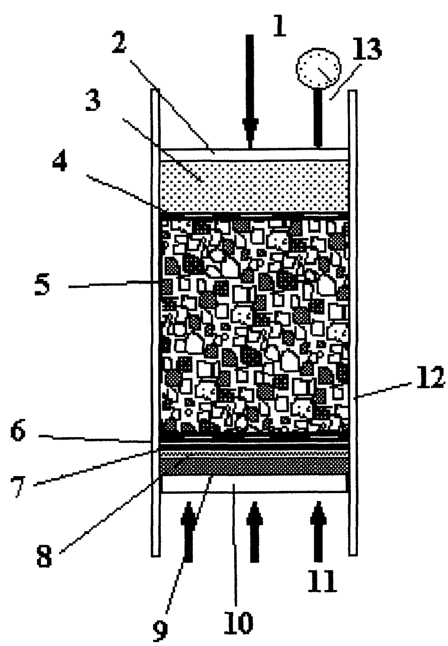


**Figure 2** Schematic of cylinder test apparatus

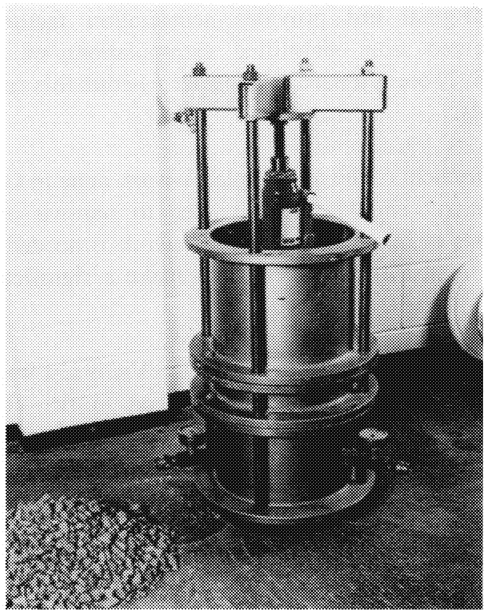

**Figure 3** Cylinder test in progress

A small team was formed of interested parties including UK and German geotextile manufacturers, test houses, academia, industry and representatives from an environmental body (interested in undertaking research on the issue) to develop a detailed test methodology with the intention of bring a degree of consistency to the various aspects of the test. The output of the group is the Environment Agency's "A Methodology for Cylinder Testing of Protectors for Geomembranes" dated March 1998. This eleven page test method is heavily based on the original German work and the in-house methodologies developed by Geofabrics in Huddersfield and Exploration Associates in Sunderland.

From the start of the work it was decided that the proposed methodology should not include any guidance on interpretation of the results because the underlying principles of the test still require further research before they can be fully validated. The reason for allowing the publication of the test before full validation was in recognition of the widespread use of the test and its recognised conservative nature. The uniform test methodology also gives the opportunity for the further planned research to be undertaken on a single methodology. The main issues that remain outstanding with regard to the test are:-.

i) Assessment of the test results was developed from work on HDPE pipes as used in highway works. Further research is required to validate the assessment for HDPE in membrane form and for the landfill environment.

ii) The original test provides acceleration factors which where experimentally derived from simplified extrapolations of the deformation behaviour of HDPE. There is a need to consider whether this can be extended to cover other materials.

iii) Measurement of the telltale plate - work done in Germany suggests that plate measurement should be automated to remove operator error in selecting indentations to be measured. Further work is needed on this issue to assess whether automated measurement does make a significant difference to the overall result.

iv) The telltale plate in the original Quo Vadis test was 0.55mm organ pipe metal, in the UK the plate is 1.3mm lead to BS 1178. During the development of the cylinder test methodology a supporting test "a methodology for determining the deformation characteristics of lead sheet" (Annex A to Environment Agency 1998) was introduced to ensure consistency in the telltale plate. It is considered that with further development of the Annex A test it may be possible to widen the specification of the metal to allow other metals to be used so long as the result of the cylinder test remains consistent.

v) The dense rubber pad performs the function of a standard subgrade. Work is needed here to better characterise the types of subgrade that the pad represents.

## Conclusions

Performance testing of protectors for landfill liners and covers provides valuable design information and assurance that the proposed materials should perform in accordance with the pollution prevention objectives set by the Environment Agency. It is essential however to ensure that where performance tests are proposed they are a part of the overall design testing regime and that the results of such testing are seen in the context of the limitations of the test methodology. The Environment Agency cylinder test has been developed, in partnership with others, to bring consistency to the test methodology and to act as a starting point from which further research can be done.

## Acknowledgements

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## Bibliography

ASTM D5514 *Standard test method for large scale hydrostatic puncture testing of geosynthetics*. American Society for Testing and Materials, Philadelphia, PA, USA.

August, H. and Luders, G. (1997). Investigation into the long-term integrity of protective materials for geomembranes in landfill basal liners. *Advanced Landfill Liner Systems* (Ed August, Holzlohner & Meggyes) Thomas Telford, London, 261-269 (ISBN 0 7277 2590 4).

British Standards Institution, *BS 1178, Specification for milled lead sheet for building purposes*, BSI, London.

Brummermann, K., Blumel, W. & Stoewahasw,C. (1994). Protection Layers for geomembranes. Effectiveness and testing procedures. *Proceedings of the fifth International Conference on Geotextiles, Geomembranes and Related Products*, Singapore, 3, 1003 - 1006.

Dixon, N., Jones, D.R.V. & Hayden, J. (1997). Performance of geoprotectors for landfill liners. *Proceedings of Geoenvironmental Engineering Conference* Cardiff, UK, (ed Yong and Thomas), Thomas Telford, 351-356.

Environment Agency (1998). *A methodology for cylinder testing of protectors for geomembranes*. Environment Agency, Regional Waste Team, Warrington, UK.

German Geotechnical Society (1993). *Geotechnics of landfill design and remedial works, technical recommendations GLR* (2nd edition), Ernst and Sohn, Berlin (ISBN 3 433 01273 3).

Keorner, R., Wilson-Famy, R.F. & Narejo, D. (1996). Puncture protection of geomembrane Part II, examples, *Geosynthetics International*, 3, 5, 655 -675.

Kirschner, R. & Kreit, V. (1993). Geotextiles, geogrids and related products - in the construction of landfill sites. *Waste Management*, March, 33-36.

Kirschner, R. & Witte, R. (1994). Use of geotextiles for the protection of geo-membranes in Landfilling of waste, barriers. (ISBN 0-419-15990-8).

Kreit, V. & Kirschner, R. (1993). Geotextiles for the protection of geomembranes in landfills. *Engineering geology of waste disposal*, Geological Society Engineering Geology Special Publication No. 11 (Ed. Bentley).

Luders, G. & Seeger, S. (1997). Geotextile protective systems for geo-membranes. *Advanced Landfill Liner Systems* (Ed. August, Holzlohner & Meggyes), Thomas Telford, London, 77-83 (ISBN 0 7277 2590 4).

Narejo, D., Wilson-Famy, R.F. & Koerner, R.M. (1996). Puncture protection of geomembranes, Part II: Experimental. *Geosynthetics International*, 3, 5, 655-675.

# Engineering properties and use of geosynthetic clay liners

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## Introduction

The construction of barriers against the flow of water is a common assignment in civil engineering. It applies particularly to environmental issues, such as the containment of hazardous substances and solid wastes in engineered landfills. The most conventional method to execute seals in landfill construction is by placement of layers of low permeability soil material. The technique of designing and placing compacted clay liners is well understood and based on a long history of application in hydraulic structures.

Mineral seals such as compacted clay layers are executed by heavy earth moving, soil processing and compacting equipment. The work depends on the weather conditions and obviously requires resources of suitable fine grained soil material. The design thickness of the mineral seal can be determined on the basis of computations of the hydraulic flux. Allowances have to be made in the design for the scatter of properties of natural soils as well as for inhomogeneities and imperfections in soil compaction. The thickness of mineral seals in landfill structures lies typically in the range of 0.5 to 1.5 m, depending on the functional requirements of the particular case. For example, in German regulations 0.5 m thick compacted clay layers are required for landfill caps. Basal liners for landfills to contain combustion residues from domestic waste should consist of geomembranes in combination with 0.75 m thick compacted clay liners and those for hazardous waste of composites of geomembranes and 1.5 m thick compacted clay layers.

The great thickness of the conventional mineral seals causes substantial earth moving activities with associated costs, delays and environmental impacts. The use of much thinner clay layers which are more efficient, is clearly of great benefit. So it is not surprising, that the application of pure bentonite, a clay with very high sealing effect, has been considered for mineral liners in landfill construction. Since the hydraulic conductivity of bentonite is smaller than that of

*Geotechnical engineering of landfills.* Thomas Telford, London, 1998

most of the compacted natural clays by about two orders of magnitude, a layer of one centimetre thickness of pure bentonite exhibits the same sealing effect as a compacted clay liner of 0.5 to 1.0 m thickness (Gartung, 1995).

In practice it is not possible to place a bentonite layer of a few centimetres thickness by the technology used in common earthworks. The bentonite must be installed by means of some kind of containment which is filled with dry bentonite industrially, transported to the site and then put into position in the structure. Thin panels of bentonite sandwiched between card boards have been used in building construction for a long time. Geosynthetic clay liners, GCLs, employ the same principle. A thin layer of dry bentonite is sandwiched between an upper and a lower flexible geotextile. The triple-layer-composite is held together by stitch bonding, needle punching or other textile techniques. So the construction product GCL appears as a thin mat that can be rolled up. GCL rolls are transported to the construction site by trucks. The GCL is easily placed on prepared grade. Once covered by a layer of protecting soil it serves the same purpose as the thick compacted clay liner, and rules have been established for the assessment of the equivalency of GCLs to compacted clay liners (Koerner *et al.,* 1995a).

The first GCLs were introduced more than 10 years ago. In the meantime about 10 different manufacturers of geosynthetics have been developing, producing and marketing their special types of GCLs. All of them follow the principle of sandwiching bentonite as the sealing mineral component between pervious geotextiles (Koerner *et al.,* 1995b). There are some differences in the bentonites used and also in the applied geotextiles. So a certain spectrum of GCLs with varying properties is available to the design engineer. One product which combines bentonite with a plastic geomembrane is also classified as a GCL. But due to the overriding sealing properties of the geomembrane these GCLs behave more like geomembranes and they are not considered in the present paper.

GCLs have been used in landfill capping systems to a great extent worldwide. In some countries like the USA where double liner basal seals are constructed, GCLs are also employed successfully as one of the two mineral liners at the bottom of landfills. In Germany where basal liners consist of composites of one geomembrane in intimate contact with one mineral seal, GCLs are not used in basal lining systems of landfills. Besides the application to landfills GCLs are used in hydraulic construction, e.g. of retaining ponds or in transportation facilities such as railways, roads and highways crossing areas where the ground water deserves special measures of pollution prevention.

## Function and properties

### Bentonite

Bentonite essentially consists of clay minerals of the smectite group, montmorillonite being the dominant species. Due to their particular physicochemical properties, montmorillonite crystals contain exchangeable cations, mostly sodium or calcium. For practical purposes, a distinction is made between bentonites which predominantly consist of montmorillonite with sodium cations and bentonites which predominantly consist of montmorillonite with calcium cations. For convenience they are called sodium-bentonite and calcium-bentonite. Both of these forms are encountered in nature and obtained by open pit mining. Bentonites are processed for many different industrial applications.

Most of the GCLs use natural sodium bentonite. Some use a bentonite which in nature occurs as calcium bentonite but by processing is converted to a sodium bentonite. To our knowledge up to now there is only one GCL that uses natural calcium bentonite. Some manufacturers add chemicals to improve the performance of their bentonite. The bentonite is delivered to the GCL manufacturer as a dry powder or in granulated form. Although it is derived from a mined natural product, there is very little scatter in the properties of processed bentonites delivered by competent suppliers.

Since 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%, calcium bentonite up to 300%. They are swelling 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, and their shear strengths. The hydraulic conductivity is lowest for sodium bentonite, that is why this mineral is preferably used for GCLs. It reaches an order of magnitude of $10^{-11}$ m/s. But at the same time, the shear strength is also lowest for sodium bentonite which causes concerns with respect to the stability of slopes (Madsen & Nüesch, 1995).

Since the properties which 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). More sophisticated mineralogical analyses may be advisable for product development and in special cases.

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 of its properties as long as the chemistry of the water-clay mineral system is not changed. Desiccation cracks heal and close again due to swelling provided 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 or even all of 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 GCL in place may undergo some changes with time due to cation exchange.

## Geotextiles

The most important property of geotextiles used for the production of GCLs is their durability. They have to resist the highly alkaline chemical environment of the bentonite and the chemistry of the water, in some cases leachate. Polypropylene (PP) and polyethylene of high density (HDPE) meet this requirement. So plastics of these types are commonly used for the upper and the lower geotextile layer of GCLs which may be woven fabrics or mechanically bonded nonwovens. The transmissivity of the geotextiles should be low to minimise liquid flow along the GCL surface. Generally, the geotextiles have to be selected according to their function, which is to retain the bentonite powder or granules and facilitate handling, transporting and placing of the GCLs. So the geotextiles should have apparent opening sizes in a suitable range and be of adequate mechanical robustness. Experience in quality control testing reveals that in practice only high quality geotextiles are employed for the production of GCLs. There are practically no complaints with respect to the geotextiles and no failures caused by their inadequacy.

## The geosynthetic composite GCL

GCLs are composites of bentonite and geotextiles. As mentioned above, their sealing function is based upon the low hydraulic conductivity and the swelling capacity of the bentonite. The geotextile components provide strength. They determine the mechanical properties of GCLs. The shear strength of hydrated bentonite is extremely low. In fact, a thin wet bentonite layer acts as a lubricant. GCLs without 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 is 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 GCL during hydration due to the restriction of the bentonite volume increase by the bonds, improves the sealing effect. The fibres that tie the GCL together are permanently stressed as long as the GCL is wet. Their tensile forces increase when the GCL is placed on a slope and the textile ties prevent the soil layers above the GCL from sliding on a slip plane within the bentonite layer.

Consequently, the internal strength of GCLs 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 GCLs meets design requirements for slopes in landfill engineering. In most practical cases, interface friction at the upper and lower surfaces of the GCL turns out to be the controlling parameter in stability analyses of landfill slopes rather than the internal shear strength.

# Testing

## General remarks

All of the properties of the components of the geosynthetic composite, GCL, discussed in the previous chapters, have to be determined by testing. Tests are carried out during the phase of product development for the selection of the best or most suitable bentonite and geotextiles. Tests are carried out during the production phase to verify the quality of the components. The relevant testing methods for bentonite and for geotextiles are compiled in Tables 2 and 5. Tests to be performed routinely on the geocomposite GCL are presented in Table 1.

In addition to the routine tests on bentonite, on geotextiles and on GCLs which determine their technical properties, numerous performance tests were carried out on GCLs or on GCL-soil systems. In such tests the behaviour of the GCL in special situations was studied. For example, the tightness of overlap joints was evaluated on the basis of large scale constant head permeation tests, the influence of strains on the sealing effect by tests on samples first submitted to tensile forces and subsequently to hydraulic gradients. Some large scale tests have been reported, simulating the tensile deformation of the GCL at the bottom of a pond observing the permeation during and after tensile straining of the GCL under constant head of water. The self healing properties of GCLs were studied on samples with holes of different sizes. GCL samples were submitted to freeze/thaw cycles and to dry/wet cycles etc.. Extensive testing was also performed on GCL samples exhumed in the field from landfill covers. They aimed at describing the condition of the GCL after a certain time in the field and comprised all of the tests mentioned already plus X-ray analyses for the detection of desiccation cracks. Research is under way at the present time to clarify the drying wetting, cracking and healing phenomena by suitable experiments.

Among all of the experimental methods mentioned, the tests for the determination of the sealing properties, permittivity tests, and the tests for the determination of the shear strength are of immediate importance to practical application of GCLs. Short comments are given on these two tests in the following paragraphs.

## Permittivity

The performance of the seal GCL is quantified by its permittivity. The flux, that is the quantity of water that permeates through the GCL over a certain area under a certain hydraulic gradient in a certain time is defined as the permittivity $\psi$, unit $m^3/(m^2 \bullet m \bullet s)$. The permittivity can be determined more precisely than the coefficient of hydraulic conductivity $k = \psi \bullet d$, in which the thickness d of the GCL at the time of flux measurement is introduced. Since it is not easily possible to measure d, most of the permeability coefficients k of GCLs presented in product documents are based on an estimate of the value d.

The experimental determination of the permittivity of GCLs is somewhat involved, because the flux through an "impervious" material is very small and very slow. The permittitivity of GCLs is measured in flexible wall permeameters or in rigid wall permeameters.

Triaxial testing apparatus is common in soil mechanics laboratories for the measurement of soil shear strength and soil permeability are used as flexible wall permeameters. They facilitate the determination of the flux through the GCL under fully saturated conditions, while they are carried out with back pressure. However, the stepwise application of the back pressure and the cell pressure in the triaxial apparatus have to be applied with great care to avoid either preconsolidation which leads to unconservative values of permittivity or sample disturbance which leads to unrealistically high values of permittivity. Triaxial tests require the application of a certain minimum surcharge load which is higher than the overburden pressure of landfill cover systems. The permittivity cannot be determined correctly for conditions of small vertical stresses. Since the chamber pressure acts as confining stress on the sample, triaxial tests cannot be used for the measurement of the permittivity of GCL samples which contain cracks or fissures, because evidently the cracks would be closed by the confining pressure.

The testing method with rigid wall permeameters overcomes some of the shortcomings of the triaxial tests. However, saturation of the GCL sample cannot be achieved under controlled conditions and the execution of permittivity tests in rigid wall permeameters is more expensive. That is perhaps why triaxial testing is more popular. Triaxial testing is also standardised in the USA as ASTM D 5887-95.

Only the rigid wall testing method should be employed for the determination of the permittivity of GCLs under low vertical pressures, like for the application in landfill covers, and for the measurement of the permittivity of GCLs which may contain cracks, fissures or macro-pores.

## Strength

When GCLs are employed in landfill capping systems, their internal shear strength and the shear resistance at their contact interfaces with adjacent soils or geosynthetics must be known for slope stability analyses. The relevant shear parameters are determined by shear tests in shear boxes 30 by 30 cm. A particular problem with such shear tests on GCLs is the transmission of the shear forces to the GCLs. In practice platens provided with short nails have worked successfully. There are a number of details to be considered when shear tests are executed for the determination of internal or contact shear strengths of GCLs.

Before the GCL is put into the shear box, it has to be conditioned to the desired water content. This process involves swelling of the bentonite. The type and amount of water added, the normal pressure acting on the GCL during swelling and the swelling time have to be selected and recorded. It makes a difference, whether the shear phase of the test is carried out under "dry" or under "wet" conditions, that means with or without external water having access to the bentonite in the GCL. The magnitude of the normal force acting on the shear plane during the test has to be decided, and the shear velocity. The soils involved as well as the geotextile components exhibit time dependent stress strain behaviour and strength. So this point is very important and has to be considered carefully. In shear tests for the determination of interface friction the properties of the material in contact with the GCL have to be taken into account. If the adjacent material is a soil, conditioning to the relevant density and water content must be achieved, and the appropriate drainage conditions during shear have to be observed according to soil mechanics practice. If the adjacent material is a geosynthetic product, its properties have to be considered likewise.

Long-term inclined plane tests for the determination of the internal shear strength of GCLs are reported by Heerten *et al.* (1995). There is some research under way for the development of special test devices for long-term shear tests (Trauger *et al.,* 1996). Large scale in situ tests on slopes were performed in the USA until some of the GCL contact planes failed (Carson *et al.,* 1998).

This short summary of shear testing should indicate that each application of GCLs involves site specific details that have to be studied carefully before shear parameters are selected for slope stability analyses from the literature or from cases executed in the past. Often it is mandatory to perform shear tests with site specific material and boundary conditions.

| Property | Test method | Components |
|---|---|---|
| Water adsorption or fluid loss | DIN 18132 or ASTM D 5891 | Bentonite |
| Swelling volume | ASTM D 5890 | Bentonite |
| Montmorillonite content | VDG P69 or by X-Ray Diffraction | Bentonite |
| Water content | DIN 18121-1 or ASTM D 4643 | Bentonite |
| Mass per unit area (GTX) | EN 965 | Geotextiles |
| Mass per unit area (GCL) | EN 965 or ASTM D 5993 | GCL |
| Bentonite content | ASTM D 5993 | GCL |
| Tensile strength Tensile elongation | EN ISO 10319 | GCL |
| Internal shear strength | LGA GK-3 or ASTM D5321 | GCL |
| Permittivity or index flux | DIN 18130-1 or ASTM D 5887 | GCL |
| Permittivity in overlapped areas | At present no referable standard (NRS) | GCL |
| Gas permeability | NRS | GCL |
| Installation damage | prENV 10722-1 | GCL |

**Table 1** Required product data for GCLs

## Product description

There are a number of different GCLs on the market at the present time. In order to evaluate their properties and to test their quality, the products must be properly described. Table 1 summarises all data needed for an adequate product description of GCLs.

The terms of delivery for geotextiles to be used in the manufacture of GCLs include detailed production descriptions. Moreover, all products to be used must be marked (prEN ISO 10320). The markings must continue to be easy to read, waterproof, and must be repeated at least every five metres. No product may be installed which is not clearly identifiable and marked.

## Quality aspects

### General

The sealing bentonite layer of GCLs has an effective thickness of 5 to 10 mm. GCLs are very efficient but at the same time very sensitive construction elements. Variations of 2 to 3 mm in thickness, which are not considered significant in conventional earthwork engineering, would result in changes in permittivity of 30 to 50 per cent. GCLs are also sensitive to the quality of the sealing clay. Greater variations in the mineralogical composition of the bentonite may easily result in a factor of ten in the coefficient of water permeability. Damage to the GCL during construction may well result the loss of the sealing function.

Because of this sensitivity to manufacturing and installation errors, quality assurance measures play a very important part in the manufacture and installation of GCLs. Experience has shown that GCLs perform reliably in actual construction cases when adequate quality assurance plans are set up and followed properly.

It is necessary to set up in quality assurance plans (QAPs) in accordance with different categories of applications, so that the scope of quality assurance may be standardised. Recommendations are made in this paper as to the minimum level of quality assurance that GCLs should have as liners in water protection zones and for landfill applications. This consists of certificates of suitability for the particular application, certificate of adherence to manufacturing standards, guarantee of competent delivery and storage, and careful installation.

The QAP is to include the manufacturing of the GCL, the local construction supervisors, the outside supervisors, the supervising authority and the company carrying out the installation (Gartung & Zanzinger, 1998).

## Manufacturing quality assurance

### *MQC of raw materials and product*

*Bentonite*

All raw materials used in the manufacture of GCLs are to undergo a quality assurance check. Every delivery of bentonite to the manufacturer of GCLs must be accompanied by a factory test certificate from the bentonite manufacturer. This must contain the following current details:

- montmorillonite content
- water content
- water adsorption
- swelling volume

Of considerable importance, however, is the supplier's or freight agent's guarantee that only bentonite shall be transported in the container vehicles. If other materials have been transported in the vehicle, e.g. sand or artificial fertiliser, proof must be produced that the vehicle has been completely cleaned. After checking the water content of the bentonite and the methylene blue value, the container vehicle can be unloaded.

| Characteristic tested | Test method | MQA |
|---|---|---|
| Water adsorption or fluid loss | DIN 18132 or ASTM D 5891 | IT, RT |
| Swelling volume | ASTM D 5890 | IT, RT |
| Montmorillonite content | VDG P69 or by X-Ray Diffraction | IT, RT |
| Water content | DIN 18121-1 or ASTM D 4643 | IT, RT |

IT Initial test in a manufacturing quality assurance
RT Regular test in a manufacturing quality assurance (spot check)

**Table 2** Factory production control (FPC) of the bentonite

*Resin and fibres*

The resin to be used for geotextile production processed into pellets, must contain binding factory test certificates for every delivery. These details contain the continuously established values for the resins:

- viscosity number (DIN 53728)
- melt flow rate (ISO 1133)
- density (DIN 53479)
- percentage of stabilisers and carbon black

The raw material receipt of goods check on resin ensures trouble-free quality and production of geotextiles. Before unloading, every container vehicle is checked for the most important details, such as melt flow rate and density, and possibly for tensile strength, tensile elongation, melting limits and crystallinity. This shows whether the raw material delivered is within the limits required. After the raw material has been approved, the container vehicle may be unloaded.

| Characteristic tested | Test method | MQA |
|---|---|---|
| Melt flow rate (MFR) | ISO 1133 | IT, RT |
| Diff. scanning calorimetry (DSC) | ASTM D 3895 | IT, RT |
| Density | DIN 53479 | IT, RT |

**Table 3** Factory production control (FPC) of the resins

It should be ensured that all manufacturing parameters are constantly monitored and that regular checks are made on the fibres:

- fibre fineness or denier (DIN 53812-1)
- fibre strength and elongation (draft DIN 53816)

Random checks on the fibres are carried out to monitor changes in the melt flow rate (MFR) caused by extrusion, the density and the carbon black content (ASTM 1603/76), using thermogravimetric or oven weight analysis. These details are allocated to the material manufactured, using batch and bale numbers, before being made available to the manufacturer of non-woven geotextiles, in summary, in a binding factory test certificate.

In the same way, the initial products for the manufacture of woven geotextile materials are monitored. The following details for the slit film fibres are constantly checked:

- thickness and width (DIN 53812-1)
- fibre strength and elongation (draft DIN 53816)

| Characteristic tested | Test method | MQA |
|---|---|---|
| Oxidation induction time (OIT-value) | ASTM D 5885 | IT, RT |
| Diff. scanning calorimetry (DSC) | ASTM D 3895 | IT, RT |

**Table 4** Factory production control (FPC) of the fibres

*Fabrics*

If the geotextile manufacturer does not manufacture the filaments or slit film fibres himself, he shall carry out random checks to monitor the above-mentioned characteristics.

All details which affect production, such as equipment settings, mixtures, etc., are constantly documented internally. Together with the details from quality checks, which are also to be carried out constantly, this enables a traceable quality control manifest system.

| Characteristic tested | Test method | MQA |
|---|---|---|
| Mass per unit area | EN 965 | IT, RT |
| Thickness | EN 964-1 | IT, RT |
| Tensile strength<br>Tensile elongation | EN ISO 10319 | IT, RT |
| Static puncture test | EN 12236 | IT, RT |

**Table 5** Factory production control (FPC) of the geotextiles

If nonwoven geotextiles are to be used, an on-line metal detector is needed in the manufacturing process.

*GCL*

If the geotextile components have not been produced in the same factory as the GCL, they must undergo a receipt of goods check, just like the bentonite. During manufacture, permanent non-destructive testing processes are of great assistance. For example, the bentonite is to be sieved before installation. After it has been applied to the bearer layer, a non-contact check of the layer thickness of the bentonite is recommended. Each roll is to be checked for length, width and weight.

| Characteristic tested | Test method | MQA |
|---|---|---|
| Mass per unit area | EN 965 | IT, RT |
| Mass of bentonite | ASTM D 5993 | IT, RT |
| Water content | DIN 18121-1 or ASTM D 4643 | IT, RT |
| Tensile strength<br>Tensile elongation | EN ISO 10319 | IT, RT |
| Internal shear strength | LGA GK-3 | IT, RT |
| Water permeability index test* | No referable standard | IT, RT |
| Permittivity<br>or index flux | DIN 18130-1<br>or ASTM D 5887 | IT, RT |

* can only be carried out with pulverised bentonite

**Table 6** Factory production control (FPC) of the GCL

The composite strength created by e.g. the needling or stitching of bearer and covering layer should also be checked.

As a general rule, for in-house monitoring, all test equipment used in the laboratory is subject to test equipment monitoring. This means that the equipment and measuring recorders are to be checked against appropriate standard measures to adjust them to primary standards. These calibrations are to be recorded.

This ensures that the measured values are correct, and that any divergence from the required figures is recognised in good time. The test equipment is marked with badges, to show the period of validity of the measuring equipment or its shutdown.

### *Manufacturing quality assutance of GCLs*

It is required (e.g. in DIN 18200) to use an independent test institute, preferably an accredited test institute to act as MQA in the context of an outside monitoring contract, to ensure that the contractually guaranteed quality of goods is observed. This must include at least the following steps:

- Initial test (IT) to establish whether the technical prerequisites for proper production and MQC (laboratory) are available.
- Regular tests (RT), twice a year, to check that MQC and current production checks are being carried out properly.
- Special test for special reasons e.g., interim production stop, repeat of regular test which was failed, if called for by the independent monitor, the manufacturer or the customer.

The independent monitor's reports are to be made available, on request, to the customer of products which are undergoing outside monitoring.

The functioning of the QAS can be checked for effectiveness. That means that all data produced within quality control during the manufacture of the GCL must be traceable from the roll number. This includes the allocated roll numbers from the initial products (bearer, supporting and covering layer), the bale numbers of the fibres, the batch numbers of the granulate and, above all, the batch numbers of the sealing material. Retained samples are to be taken from the roll. These are to be stored in a dark, dry place, and marked with roll numbers. The MQA checks the coefficients shown in the Tables 2 to 6.

## Certificates of suitability

### *General requirements*

The following basic requirements may be made of GCLs:

- Sealing function
- Mechanical resistance
- Durability
- Constructability
- Other requirements
- Quality assurance

The sealing function of GCLs must be demonstrated. The permittivity shall be very small with respect to water and other liquids, that can occur in the particular application. These are especially rainwater and leachate in landfill constructions.

GCLs must be reliable seals under the milieu conditions of the particular application. This means that freezing/thawing cycles and drying/wetting cycles do not alter the properties to such an extent, that the sealing performance is considerably impeded. Further more, the durability of GCLs under the expected temperatures, in the expected chemical milieu shall be warranted. GCLs must resist micro-organisms and fungi. The design shall account for possible menaces like destruction by digging animals and desiccation or penetration caused by plant roots.

The influence of deformations on the permittivity of GCLs shall be small. This has to be demonstrated by permittivity tests on GCL samples under a biaxial state of strain covering the expected order of magnitude.

It has to be demonstrated that the sealing function is adequate in the field. Seams, connections to structures, penetration of pipes etc. must be reliably tight.

The mechanical strength of GCLs must be sufficient to resist all expected loading conditions during transportation, placement, in place short and long term to achieve adequate structural stability. Resistance must also be sufficient against erosion under the pertinent hydraulic gradients.

### *Laboratory examinations*

Certificates of suitability are to be produced by independent, accredited geosynthetic institutes. The following proofs must be submitted:

- water permeability
- water permeability in overlapping area
- water permeability in elongated condition
- capacity to withstand mechanical stresses
- water permeability after freeze/thaw cycles
- water permeability after dry/wet cycles
- gas permeability
- interface shear resistance
- internal shear resistance
- long-term shear resistance
- installation damage
- durability

### Field test sites

In the construction of barrier layers, field test sites are to be seen as large-scale suitability tests, in which the outside supervisor is to be involved, as well as the company's own supervisor. The following information is to be shown:

- suitability of the products under construction site conditions
- suitability of the installation method and equipment envisaged
- compliance with the required values for permeability in the scale of the construction
- compliance with stability
- determination of reference values for quality assurance.

The field test sites are not a part of the subsequent sealing work. They are to be prepared adequately before the construction of the liner system. The values resulting from the field test sites are to be compared and evaluated against those of the laboratory suitability examinations.

## Quality assurance on the construction site

### *Construction supervision*

It is the contractor's responsibility to ensure the following:

- the identity of the supplier of the rolls delivered to the construction site can be checked, using the delivery note and the roll label
- dry, even storage surface is available
- the rolls are unloaded and shifted carefully
- damage to the rolls caused by lifting with heavy equipment is avoided
- transport and storage regulations are observed
- the protective foils are not damaged.

The local construction quality control (CQC) must check:

- that installation is carried out on the basis of the prescribed installation plans
- that the subgrade has been properly prepared, i.e. free of stones and sufficiently load-bearing
- that the slope lengths and gradients are correct
- that the measurement of the anchoring ditches is correct and that these have no sharp edges
- that the roll weight is sufficient
- the external condition, the level area and that the liners are checked for mechanical damage
- that the prescribed overlapping width and the relevant instructions are observed
- that there are no stones or dirt in the overlaps
- that the bentonite layer in the joints is applied continuously
- that a stoneless protective layer is applied immediately the GCL has been laid
- that any swollen areas not covered with a protective layer are removed
- that driving over the GCL directly is avoided.

### *Control checks*

The following control checks may be carried out by the construction quality assurance (CQA) of the constructional measures, in the context of the project:

| Property | Test method |
|---|---|
| Swelling volume | ASTM D 5890 |
| Water content | DIN 18121-1 or ASTM D 4643 |
| Mass per unit area | EN 965 or ASTM D 5993 |
| Tensile strength<br>Tensile elongation | EN ISO 10319 |
| Splitting (peel) test | LGA GK-6 |
| Permittivity<br>or index flux | DIN 18130-1<br>or ASTM D 5887 |

**Table** 7 Control checks of the GCLs

## Performance observations

### General remarks

GCLs have been employed in many constructions world-wide during the last few years. Their performance has generally met expectations. In some cases of GCL-lined ponds leakage problems occurred during impoundment. Careful examination of the construction histories and partial or even total recovery of exhumed GCLs revealed in most of these cases, that the leaks had been caused by mistakes in GCL placement. It is of utmost importance to follow the instructions provided by the GCL manufacturer to obtain the designed sealing function. In this regard special attention has to be paid to the overlap joints.

GCLs in landfill capping systems are easily accessible. In a number of cases GCL samples were taken from working landfill covers and submitted to testing. The results of such studies indicate, that the bentonite may undergo some alterations with time, such as cation exchange. But as long as the in situ water content of the GCL does not fall below a certain limit, the hydraulic conductivity remains low. Two examples of GCL performance observations are briefly discussed below. In Germany, large scale lysimeters were installed for GCL-performance observations in various parts of the country during the past two years. So we expect more information about the long-term behaviour of GCLs under different climatic conditions in the near future.

### Hamburg-Georgswerder

In 1994 two large test fields and three smaller test pads were installed at Hamburg-Georgswerder, north Germany, at an old closed landfill (Steinert & Melchior, 1997). The test installations were added to existing field testing facilities where observations had been carried out over the years before and where all relevant measuring devices for weather data were available. The lysimeters for the observation of the performance of GCLs were placed at the sloping surface of the landfill cover well above the composite sealing system of a geomembrane and a compacted clay liner. They did not form part of the acting landfill cover. The interior of the landfill itself is known to have elevated temperatures. No moisture could dissipate from the landfill body to the GCL-test field because it was intercepted by the intact geomembrane of the capping system.

In order to obtain a quick response of the GCLs to precipitation and drying, the thickness of the cover soil above the GCLs was kept considerably smaller than in actual landfill constructions. It consisted of 30 cm vegetative soil with grass above 15 cm of coarse grained drainage gravel which was placed directly on top of the GCLs.

The simultaneous readings of precipitation and leakage through the GCLs showed, that during the first winter season the GCLs performed excellently. No substantial leakage was recorded. So it could be concluded that the test installation operated correctly. Observations were carried on over the next two years with extremely dry and hot summers. Then, at the end of summer 1995, the first heavy autumn rains lead to excessive leakage rates. Apparently the GCLs had developed some desiccation cracks and, against expectations based on laboratory testing, these cracks did not heal. Leakage rates remained high at about 25% of precipitation and the lysimeters responded immediately to all rain events.

The GCLs were exhumed and examined very carefully by several independent research institutes. Visual observations showed a certain amount of root penetrations, X-ray testing revealed crack patterns in the GCLs, the water content of the bentonite which in situ once had been at well above 150% had dropped to somewhere between 40 and 100%, the swelling potential, cation exchange capacity and water adsorption were considerably lower than they should be for sodium bentonite. Measurements of the hydraulic conductivity initially yielded high permittivities due to open cracks which closed gradually during testing when flexible wall triaxial permeameters were used.

The very extensive testing programme and theoretical studies lead to the conclusions that due to the insufficient thickness of cover soil the GCLs of the test fields had undergone severe desiccation with cracking. At the same time cation exchange had taken place, most of the sodium ions had been replaced with calcium ions, and this alteration of the bentonite had prevented quick self healing. Furthermore, root penetration had also played a role in increasing the permeability and in decreasing the water content of the hydrated bentonite. It was observed that generally desiccation was less severe in the overlaps. Evidently the upper GCL at overlaps had protected the lower one from drying.

This short summary of results cannot transmit all the important details of the research programme and all the findings. But it should indicate, that GCLs may not perform successfully in extreme situations. At Hamburg-Georgswerder the cover soil was too thin and had a too small water retention capacity, vegetation caused root penetration, there was a coarse grained drainage layer immediately above the GCLs, summers were unusually dry and hot, and elevated temperatures inside the landfill body contributed to desiccation. Many of these adverse conditions do not occur in normal landfill covers or can be avoided by proper design. So Hamburg-Georgswerder has to be regarded as a crash test rather than a case history.

### Nuremberg

A more representative case is reported from Nuremberg, where in summer 1997 a GCL was exhumed. The municipal solid waste landfill of Nuremberg in south Germany contains domestic waste. The section of the landfill discussed here, was closed and provided with a capping system in late 1994. The cover consists of a layer of slightly silty sand below the GCL, a GCL with natural sodium bentonite, a layer of 10 cm slightly silty sand immediately above the GCL, then 15 cm of coarse grained drainage gravel and 75 cm of slightly silty sand with grass vegetation at the top.

Like in Hamburg, in Nuremberg the exhumed GCL had undergone two very dry summers. But unlike in Hamburg, the GCL in Nuremberg showed no desiccation cracks. The moisture content was in the range of 100%, and no crack patterns could be detected by X-ray testing. There were no root penetrations either. Even though mineralogical analyses revealed a partial cation exchange from sodium to calcium bentonite, the hydraulic conductivity had not increased in comparison with the initial values determined before construction.

The performance observations at Nuremberg indicate satisfactory functioning of the GCL. Evidently, the conditions here are more favourable than at Hamburg-Georgswerder. The most important features appear to be the greater thickness and water retention capacity of the cover soil, and the fact that there is a silty soil layer immediately above and below the GCL thus reducing the tendency of the water saturated bentonite to dry out. Drainage gravel immediately above the GCL as carried out at Hamburg-Georgswerder and in many other applications enhances the tendency for desiccation of the moist bentonite of the GCL.

## Conclusions and recommendations

GCLs essentially function as mineral seals of extremely small thickness. Their main advantages are high efficiency, ease of handling, transporting and placement and uniform quality due to industrial production. Their limitations have to be observed. These are mainly related to the properties of the bentonite. In Germany they are presently accounted for by observing the following advice with respect to landfill covers, as issued by the approval authority DIBt, (Deutsches Institut für Bautechnik, German Institute of Construction):

- GCLs are installed in double layers, the upper layer protecting the lower layer against desiccation. Other types of desiccation protection are permitted, provided their efficiency is demonstrated.
- The soil immediately above and below the GCL should have characteristics which prevent the GCL from desiccating (silty sands meet this requirement).

- The cover soil must have a thickness of at least 1 m and has to provide sufficient water storage capacity.

These rules are aiming at preventing desiccation cracking which jeopardises the long-term sealing function, especially when it occurs along with cation exchange which can hardly be avoided.

This paper has also focused on the quality control and quality assurance aspects of GCLs. Since GCLs are used in critical and permanent applications it is important that a complete quality assurance system is in place.

Every aspect of the GCL must be subjected to the same rigor. This includes the bentonite and associated geotextiles. In this latter regard concern is focused on the resins, fibres and fabrics used in the manufacturing process. Obviously, the completed GCL must be tested as a composite material as well.

## References

Carson, D.A., Bonaparte, R., Daniel, D.E. & Koerner, R.M. (1998). Geosynthetic Clay Liners at Field-Scale: Internal Shear Test Progress. *Sixth International Conference on Geosynthetics*, 25-29 March 1998, Atlanta, Georgia, USA, 427 - 432.

Egloffstein, T. (1995). Properties and test methods to assess bentonite used in geosynthetic clay liners. *Geosynthetic Clay Liners*, Koerner, R.M., Gartung, E., Zanzinger, H. (eds.), Balkema, Rotterdam, 51 - 72.

Gartung, E. (1995). GCL summary and conclusions. *Geosynthetic Clay Liners*, Koerner, R.M., Gartung, E., Zanzinger, H. (eds.), Balkema, Rotterdam, 229 - 233.

Gartung, E. & Zanzinger, H. (1998). Need for Quality Approach: Installation and Quality Control. *GEO-BENTO 98,* 2-3 February, Paris, France, 345 - 353.

Heerten, G., Saathoff, F., Scheu, C. & von Maubeuge, K.P. (1995). On the long-term shear behaviour of geosynthetic clay liners (GCLs). *Geosynthetic Clay Liners*, Koerner, R.M., Gartung, E., Zanzinger, H. (eds.), Balkema, Rotterdam, 141 - 150.

Koerner, R.M. & Daniel, D.E. (1995a). A suggested methodology for assessing the technical equivalency of GCLs to CCLs. *Geosynthetic Clay Liners*, Koerner, R.M., Gartung, E., Zanzinger, H. (eds.), Balkema, Rotterdam, 73 - 98.

Koerner, R.M., Gartung, E. & Zanzinger, H. (eds.) (1995b). *Geosynthetic Clay Liners.* Balkema, Rotterdam, 245 p.

Madsen, F. & Nüesch, R. (1995). Characteristics and sealing effect of bentonites. *Geosynthetic Clay Liners*, Koerner, R.M., Gartung, E., Zanzinger, H. (eds.), Balkema, Rotterdam, 31 - 49.

Steinert, B. & Melchior, S. (1997). Untersuchungen zur Wirksamkeit von Bentonitmatten in austrocknungsgefährdeten Oberflächenabdichtungen. Landesumweltamt Nordrhein-Westfalen (ed.), *Sanierungsuntersuchung und Sanierung von Altlasten, Altlasten-*

*seminar des LUA NRW und des Abfallentsorgungs- und Altlastensanierungsverbandes*, Selbstverlag, Duisburg, 20 p.

Trauger, R.J., Swan, R.H. & Zehong, Y. (1996). Long-Term Shear Strength Behavior of a needlepunched Geosynthetic Clay Liner. *Testing and Acceptance Criteria for Geosynthetic Clay Liners*, ASTM STP 1308, Larry W. Well, Ed., ASTM, Philadelphia, USA, 103 - 120.