Investigating the stability of geosynthetic landfill capping systems

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INVESTIGATING THE STABILITY OF GEOSYNTHETIC LANDFILL CAPPING SYSTEMS

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A thesis submitted to the Department of Civil & Building Engineering in partial fulfilment of the requirements of Loughborough University for the degree of Doctor of Philosophy

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Abstract

The use of geosynthetics in landfill construction introduces potential planes of weakness. As a result, there is a requirement to assess the stability along the soil/geosynthetic and geosynthetic/geosynthetic interfaces. Stability is governed by the shear strength along the weakest interface in the system. Repeatability interface shear strength testing of a geomembrane/geotextile interface at low normal stresses suitable for capping systems showed considerable variability of measured geosynthetic interface shear strengths, suggesting that minor factors can have a significant influence on the measured shear strength. This study demonstrates that more than one test per normal stress is necessary if a more accurate and reliable interface shear strength value is to be obtained. Carefully controlled inter-laboratory geosynthetic interface shear strength comparison tests undertaken on large direct shear devices that differ in the kinematic degrees of freedom of the top box, showed the fixed top box design to consistently over estimate the available interface shear strength compared to the vertically movable top box design. Results obtained from measurement of the normal stress on the interface during shear with use of load cells in the lower box of the fixed top box design, raise key questions on the accuracy, reliability and proper interpretation of the interface shear strength data used in landfill design calculations. Tests on the geocomposite/sand interface have shown the interface friction angle to vary with the orientation of the geocomposite's main core, in relation to the direction of shearing. Close attention needs to be paid to the on-site geocomposite placement in confined spaces and capping slope corners, as grid orientation on the slope becomes particularly important when sliding is initiated. Attempts to measure the pore water pressure during staged consolidation and shear along a clay/geomembrane interface in the large direct shear device suggest that this interface is a partial drainage path.

Keywords: Geosynthetics, Capping system, Interface shear strength, Large direct shear device, Grid orientation, Slope stability, Pore water pressure.
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The geosynthetic materials used were provided free of charge by the manufacturers. The geomembranes were provided by GSE Lining Technology (UK), the geotextiles by Geofabrics (UK) and the geocomposites by both GSE & Terram (UK).

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1. Introduction

1.1 Landfill: A General Overview

Landfill is currently the largest waste disposal option in the United Kingdom (UK). Until recently, landfills have been the most economical and environmentally acceptable method of disposing of residual solid waste. Leachate (liquid that collects at the bottom of a landfill) was allowed to percolate through the landfill base with the expectation that it (the leachate) would be purified by the unsaturated soil beneath the landfill and by ground water. The concept of soil attenuation has since changed significantly as increasing concern regarding the environmental risks associated with landfilling of waste has led to extensive environmental regulations requiring fully engineered geotechnical structures for landfills. Landfill is still the main means of managing most of the UK’s waste.

The modern landfill is a carefully designed structure built into or on top of the ground. In it, the waste is isolated from groundwater, air and rain with a bottom liner and daily covering of soil. Modern landfill practice now includes monitoring of incoming waste, waste placement and compaction, installation of environmental monitoring (e.g. collecting/analysing gas and water samples) and construction of control facilities e.g. lining systems, leachate collection and extraction systems and gas collection and extraction systems.

1.1.1 Landfill Legislation in the UK

Before the introduction of the Control of Pollution Act 1974 (COPA), most landfill sites were operated on the basis of dilute and disperse. The introduction of COPA required local authorities to license and regulate landfill sites. Waste Management Paper 26 (DoE 1986) helped set initial standards for landfill engineering including bulk earthworks, under drainage, engineered clay, use of synthetic liners, leachate management, etc. Waste Management Paper 26A
(DoE 1993) included criteria developed to determine when monitoring of landfill gas emissions at a restored landfill could be discontinued and when the site could be used for unrestricted development.


The Integrated Pollution Prevention and Control (IPPC) Directive (1996/61/EC) applied a more integrated environmental approach to the permitting and regulation of a range of industrial activities including landfill.

The Landfill Directive (1999/31/EC) was introduced "to prevent or reduce as far as possible negative effects on the environment, in particular the pollution of surface water, groundwater, soil and air, and on the global environment, including the greenhouse effect, as well as any resulting risk to human health, from the landfilling of waste, during the whole life-cycle of the landfill". The Directive provides for the location of landfills, and the technical and engineering requirements for aspects such as water control and leachate management, protection of soil and water and methane emissions control. The Directive sets demanding targets to reduce the amount of biodegradable municipal waste sent to landfill, by encouraging waste minimisation and increased levels of recycling and waste recovery.

The Directive defines the categories of waste (municipal, hazardous and non-hazardous waste and inert waste) and applies to all landfills. It lays down a standard waste acceptance procedure to avoid any risks and sets up a system of operating permits for landfill sites.

Pollution Prevention and Control Regulations 2000 were introduced under the Pollution Prevention and Control Act 1999, to build on existing environmental
control systems. The PPC Regulations 2000 are gradually replacing the pollution control regime set up under Part 1 of the EPA 1990. This transitional process will be completed by 2007.

To implement the Landfill Directive (1999/31/EC), Section 5.2 of the PPC 2000 Regulations was replaced by The Landfill (England and Wales) Regulations 2002 and came into force on 15 June 2002. The Landfill Regulations 2002 set new standards for the design and operation of landfills, requiring landfill operators to apply for a Landfill Permit.

Important aspects applicable to capping and restoration include the consideration of the setting of the landfill and its visual impact in relation to residential, recreational and other amenity areas (Environment Agency, 2004). The classification of a given landfill (i.e. if hazardous or non hazardous), affects the design of its capping, restoration and after use.

1.1.2 Purpose of Design

The Landfill (England & Wales) Regulations 2002 require that infiltration of water into the landfill is controlled, that there is engineered containment and sealing of waste, that landfill gas is controlled and collected (and utilised) or flared. A stability risk assessment is carried to ensure that the waste mass is stable and that there is no uncontrolled slippage of any of the components of the lining or cover system. Stability of landfill lining or cover systems is controlled by the shear strengths mobilised at the interfaces between the various components. Any potential stresses that could cause deformation of the materials used and/or formation of shear zones in the containment system must be controlled.

To maintain the integrity of the containment system during operation and over the long-term post closure management period, it is necessary that the behaviour/interaction of the materials in the component system during construction and after closure of the landfill (when settlement of waste can cause deformation in the lining system) is assessed.
1.1.3 **Construction Quality Assurance**

On completion of the landfill liner or capping design process, construction quality assurance (CQA) plans are submitted to the EA for approval prior to any cell or cover construction on site. The CQA plan indicates the quality assurance measures available to ensure that the works comply with the standards and specifications of the license and best practicable environmental option.

Independent CQA supervision takes place on site and a CQA report incorporating materials testing and as-built information (with photographic evidence) is submitted to the EA for approval. CQA monitoring is enforced by the EA to ensure compliance not only with the specification and design but that suitable materials are used.

1.1.4 **Landfill Closure**

On cessation of disposal, the landfill is capped and restored as agreed with the EA in the restoration plan. There is usually continuous monitoring of landfill gas, leachate, groundwater and settlement levels over a long period of time. When post closure monitoring is completed and the EA is satisfied that the condition of the land is unlikely to cause pollution of the environment or harm to human health, a 'Certificate of Completion' is granted and the Waste Management Licence is surrendered.

1.2 **Components of a Typical Landfill Capping System**

The design of landfill caps is site specific and mostly depends on the intended functions of the capping system as a whole. Landfill caps range from a one layer system of vegetated soil to a complex multi-layered system of soils and geosynthetics, the latter used more in wet climates.

The modern landfill cap, a schematic of which is shown in Figure 1.1, is an engineered multi-component system constructed directly on top of the waste after a given cell or void space has been filled to capacity.
1.2.1 Purposes of Landfill Cover

The primary purposes of the landfill cover are:

- To minimise/control the infiltration of meteoric water (rainfall and snow fall) into the underlying waste, thus limiting the production and migration of leachate,

- To limit the uncontrolled release of landfill gases e.g. methane into the atmosphere, and the possible spread of fires,

- To physically separate the waste from the environment to protect public health, and

- To provide a suitable surface for the re-vegetation and/or reclamation of the site.

The components of a typical landfill cover are discussed below.
1.2.2 Foundation Soil Layer
As the initial waste separator, the sub base soil layer also helps contour the existing surface of the landfill. Mechanical compaction on this layer by way of heavy proof rolling with large compactors will generally serve to minimise differential settlement of the final cover over the short term. Waste degradation over the long term will cause settlement of this layer and those above it.

1.2.3 Gas Collector Layer
An effective landfill gas collection/venting system is essential to municipal solid waste landfills with decomposing/putrescible waste or volatile organics, which produce gas. The gas collection layer may be constructed of coarse grained highly permeable soils e.g. sand, gravel, (up to 300 mm thickness). High in-plane transmissivity geosynthetic materials e.g. geonets, thick needle punched non-woven geotextiles and geocomposites, can also be used.

1.2.4 Hydraulic/Gas Barrier Layer
The hydraulic barrier layer is the most critical of all cover system components. It is designed to directly block the percolation of water through the cover system into the waste and to promote drainage or storage of the water in the overlying layers, which is later removed by runoff, evapo-transpiration or internal drainage (Quian, et al, 2002). Landfill gases too, from beneath the barrier layer, are prevented from escaping into the atmosphere but are forced to migrate laterally beneath it.

The simplest form of barrier consists of a single material type e.g. compacted clay, bentonite enriched soils (BES), geomembranes, geosynthetic clay liners, etc. Engineered clay and a geomembrane can be combined to form a better performing composite double barrier. Linear low density polyethylene (LLDPE), Low density polyethylene (LDPE) or Very Flexible Polyethylene (VFPE) are the most common geomembranes used in the UK for landfill capping. They are available in both smooth and textured surfaces for increased
shear strength interfaces. Geomembranes are impermeable to water but if not protected can be easily damaged during installation.

1.2.5 Drainage Layer
The drainage layer is connected with the various perimeter trenches and drainage ditches. It drains the overlying soil layer by transporting meteoric water through the cover system, away from the barrier layer. This reduces the head of water on the barrier layer and helps to reduce and control the pore water pressures in the cover soil; a very important aspect in the overall slope stability.

Selection of drainage material must allow for adequate filtration, reduce clogging by fines to a minimum and prevent puncture of the underlying hydraulic/gas barrier layer. Where drainage soils e.g. sand and gravel are used, a carefully designed filter must be constructed. Water must be allowed to freely discharge from the drainage layer into collector ditches from where it can be removed from the landfill.

1.2.6 Restoration Cover Soil
Cover soil/topsoil is used to contour the surface of the landfill and to support the plants that will be used in the long-term closure design of the landfill. Fertile topsoil is used in this layer to allow vegetation growth. Topsoil thickness is generally ~ 300 mm, deep enough for suitable plant root penetration, to reduce erosion. Geosynthetic erosion control mats can be used temporarily during and after seeding to allow enough time for the establishment of vegetation. Coarse to fine sized cobbles and gravels are used as a replacement to topsoil in arid areas.

1.3 The Use of Geosynthetics
Planar geosynthetics are increasingly becoming the standard in capping systems because of the economic benefits that come through increased void space, quicker construction times and satisfaction of environmental regulatory requirements. Geosynthetics are used as drainage and filtration materials (e.g. investigating the stability of geosynthetic landfill capping systems
Chapter 1: Introduction

geocomposites, geotextiles), hydraulic/gas barriers (e.g. geomembranes) and protection/reinforcement (e.g. geotextiles) layers in landfill capping systems.

Using geosynthetics in a cover system in combination with other geosynthetics or weak soil layers, however, introduces planes of weakness or interfaces of low shear strength. The material interface behaviour greatly contributes to the response of the whole system and may control its performance, therefore the individual soil and geosynthetic material aspects have to be considered

1.3.1 Geosynthetic interface shear strength

Interface shear strength can be defined as strength at failure on a particular interface. Interface shear strength is obtained by laboratory testing of the geosynthetic materials and soils specific to the project, at the normal stress, density and moisture content suitable for the proposed design, in a direct shear apparatus.

Three replicate tests are performed, with the only variable being the normal stress ($\sigma'_n$). The middle normal stress ($\sigma'_n$) value is targeted to the site specific condition while the lower and higher values cover the range of possible normal stresses available. The three tests are plotted on a shear stress vs. horizontal displacement graph and a peak and residual shear stress obtained for each normal stress. For each normal stress, the shear stress increases with increasing displacement until a peak shear stress is achieved. Further displacement shows a reduction in the shear stress to an eventual residual value. Geosynthetic interfaces generally show a reduction in shear stress at displacements beyond peak strengths.

The Coulomb criterion has been used to express interface shear strength as a function of the normal stress, as shown in Equation 1.

$$\tau = \sigma'_n \tan \delta' + \alpha'$$  \hspace{1cm} \text{Equation 1}

Where:
\[ \tau = \text{shear stress} \]
\[ \sigma_n = \text{normal stress} \]
\[ \delta' = \text{Peak angle of shearing resistance (friction angle)} \]
\[ \alpha = \text{y-intercept (cohesion)} \]

For movement to occur along the interface, the shear stress has to overcome a shearing resistance as well as the value of the y-intercept.

Landfill covers are associated with low overburden stresses i.e. normal stresses less than 30 kPa, which control the shear strength along the interfaces and the overall stability. Therefore, incorporating geosynthetic materials challenges the stability of the capping slope.

1.4 Landfill Capping Stability Concerns

The stability of geosynthetic cover systems is controlled by the shear strengths mobilised at the interfaces between the various soils and geosynthetics involved in the system. When geosynthetics are incorporated, failure is known to most likely occur at the geosynthetic interfaces because they typically provide the lowest shear strength in the system. The potential shear plane is generally linear and parallel to the angle of slope. The overlying cover soil usually slides along the surface with the lowest interface shear strength. Landfill cover design must carefully consider slope stability, long-term degradation and erosion.

1.4.1 Accurate/Reliable Interface Shear Strength Data

The current state of practice in slope stability analyses often uses interface shear strengths based on prior experience, published data and/or limited site specific testing, without taking into consideration the possible variability in results. For low normal stress design requirements, sometimes interface shear strength test data obtained at high normal stresses is extrapolated to get low normal stress values.
The quality and reliability of the shear strength data is highly dependent on the design of the test apparatus used, operator/test procedure and variability of both geosynthetic and soil materials.

1.4.2 Water On The Interface
In the landfill cover, surface water can collect above the hydraulic liner and leachate can collect below it. The presence of water on, above or below the interface can reduce the stability of the system by contributing to the disturbing forces, increasing seepage flow and reducing the effective stress on the upper and lower surfaces of the hydraulic barrier.

At present, there are no known methods available for measuring the magnitude, distribution and time dependent variation of pore water pressures generated at interfaces during shearing and seepage. New test methods are required to assess the pore pressures generated at interfaces when sheared in saturated and partially saturated conditions. This will enable the controlling effective stresses to be quantified and the relevant design parameters to be defined.

1.4.3 Selecting Appropriate Material Factors
Selecting appropriate interface shear strength, internal shear strength (e.g. for geocomposites) and geotechnical parameters (for cover soils) is crucial to the design and long term stability and integrity of the capping system.

For geosynthetic interfaces, the designer must choose whether to use peak or residual shear strength parameters for a particular design problem. Post peak (residual) shear strengths are generally generated by mechanisms such as:

- Excessive differential settlement which could cause strains in the various capping system components, as each layer responds differently to the strain.
- Failure along an interface on the slope due to mass movement of materials. Failure is known to most likely occur at the
geosynthetic interfaces as they typically provide the lowest shear strength in the system.

- Failure of the waste mass due to settlement, pore water pressure build up, etc.
- Movement of construction plant, seismic activity, gas uplift, etc.

Current design practice is such that the use of the cohesion value in design is left to the discretion of the engineer or generally ignored in design calculations.

Generally, the use of Eurocode 7:1997 (in which material properties have to be factored for conventional design situations) and the use of peak or residual shear strength values in design analyses, are still not very well understood by many slope design engineers.

1.4.4 Choosing a Factor of Safety

A proposed capping system design must achieve adequate factors of safety in respect of stability and integrity. Global safety factors are easy to use but are often misused and misapplied. A factor of safety is intended to account for uncertainty in design. The sources of uncertainty in the use of geosynthetics in design could come from the variability of the materials, accuracy of test method, effects of moisture on the barrier layer (i.e. from above due to percolation or from below due to gas uplift) and the large displacement shear strength.

Generally, as in the design of embankments, landfill slope designers tend to apply a factor of safety of 1.5 for cover system slopes. In such circumstances, it is possible to overestimate or underestimate the probability of slope failure. The choice of an acceptable factor of safety requires sound engineering judgement due to the multitude of factors that must be considered. Liu et al., (1997) have discussed these factors which generally fall under two categories namely;

- the consequences of a failure occurring and
- confidence in the information available.
Confidence in choosing a factor of safety depends on the complexity of the ground conditions, adequacy of the information obtained from the site investigation, laboratory testing and the certainty/accuracy of the design parameters e.g. shear strength and pore pressures.

1.4.5 Influence of Site Practices
The influence of site practices on the frequency and type of interface testing required provides feedback on the relevance of the laboratory test for obtaining shear strength parameters and their use in design.

Site practices may include variation in the properties of the materials delivered to site, degree of care taken during on site storage and transportation to required location, placement techniques, etc. It is, therefore, important that the testing conditions in the laboratory, including the moisture condition of the soil sample, side of placement and direction of placement of geosynthetic etc are as close as possible to field conditions.

1.5 Purpose of Research
The purposes of this investigation are threefold, namely:-

1. To develop conclusive methods for obtaining interface shear strength parameters at low normal stresses relevant to capping system design.

2. To investigate the structural performance/shear behaviour of recently developed geocomposite drainage materials.

3. To assess the role played by time dependent pore water pressures generated at the interfaces during shear, on the structural stability of cover systems.

1.6 Research Methodology
This research project was jointly sponsored by Golder Associates (UK) and Loughborough University and is a continuation of research work undertaken by Jones (1999) on the stability of landfill lining systems. Although this thesis
concentrates on capping systems, higher stresses associated with landfill slope and basal systems are discussed where appropriate.

The literature review gives a general overview of landfills and capping in particular. Other relevant topics that are discussed include slope stability, interface shear strength, factors that influence interface shear strength, direct shear device designs and operation, testing standards and interface testing of various material combinations.

A study was undertaken on the properties and benefits to cover systems of materials like geocomposites which at the commencement of this research project, were fairly new to the market.

The large direct shear device was used initially to quantify the internal forces within the standard 300 mm x 300 mm fixed shear box equipment when operated at low normal stresses. As a result, a rigorous but reproducible test procedure was developed. Shear strength testing involved various combinations of geomembranes, geotextiles, geocomposites and soils. Repeated tests on a particular geomembrane/geotextile interface at normal stresses suitable for capping systems were done. Other tests involving geomembrane/geotextile and geotextile/sand interfaces were undertaken on three different shear devices at varying normal stresses to compare the reproducibility of results. Operator error was minimised as much as possible. Load cells were successfully used in the bottom box of the fixed top shear device to measure the normal load on the interface during shear.

It was attempted to measure pore water pressures on the interface during shear in the large direct shear device. Pore pressure transducers were specially attached to the geomembrane in a geomembrane/clay interface to measure the pore water pressure at the interface before and during staged consolidation and through a very slow rate of shear, to find out if the interface was drained. For this experiment, the normal stresses used were much higher than would be required for capping systems. The challenge in setting up this experiment was in keeping
the system thoroughly de-aired, to enable rapid transducer response to pressure changes.

1.7 Main Findings

1. The pneumatic bag, a standard fitting for most large direct shear devices, is not suitable for use at low normal stresses because of uneven load distribution on the interface. A standard test method for interfaces of cover sealing systems has been defined.

2. For all combinations of the materials tested, the normal stress acting on the interface in the fixed top shear device has been found to be higher than the applied normal stress on the top of the sample. As a consequence, shear strength parameters obtained using the fixed top box device have been found to be consistently higher than those obtained using the vertically movable top box device.

3. Repeatability test results obtained using the same fixed top shear device, procedures, materials and normal stress have shown variability of calculated shear strength and hence shear strength parameters.

4. Interface shear strength between a geocomposite drain and adjacent soil has been found to be dependent on the orientation of the primary grid forming the core, in relation to the direction of shear.

5. Results obtained from measurement of pore water pressures on a clay/geomembrane interface during staged loading and over a period of a very slow rate of shear suggest that the interface is possibly a drainage boundary.

1.8 Guidance to Report

The literature review presented in Chapter 2 includes a description of landfill cover systems, current legislation governing landfill and landfill covers in particular, measurement of interface shear strength, a description of the large
direct shear device, design and stability of capping systems and a brief overview on measuring pore water pressure changes in shear.

Chapter 3 explores the experimental apparatus, material specification and test procedures used to determine geosynthetic interface shear strength and the design of the pore water pressure measurement laboratory experiments. There is a brief discussion on the research techniques, the limitations encountered and suggestions of alternative test methods and equipment.

Laboratory test results are presented in Chapter 4. They include repeatability, reproducibility and geocomposite grid direction interface shear strength test results, as well as pore water pressure measurement test results. The interface shear strength test results are presented in the form of shear stress vs. displacement, normal stress vs. displacement and shear stress vs. normal stress plots. The pore water pressure results include plots showing the variation of pore water pressure with increasing normal stress and the behaviour of the pore water pressures during the shearing phase.

Chapter 5 incorporates analyses and interpretation of the laboratory tests. The Monte Carlo simulation is used to help quantify shear strength parameters obtained from test results. The approaches for obtaining characteristic shear strength parameters and the importance of compliance testing during construction are discussed. The possibility of the clay/geomembrane interface being a drainage path is also raised.

Chapter 6 considers the original purposes of this project in light of the main findings. The implications of these findings on theory, testing and industry are discussed. Also included are the challenges encountered together with recommendations for further research.

Documents presented in the Appendices include detailed test procedures for the large direct shear devices, geosynthetic material specification sheets from manufactures and a list of papers published.
2. Literature Review

This chapter focuses on the measurement of interface shear strength and includes a discussion on the devices available, focusing on the large direct shear device. The standards used in measurement of interface shear strength in the large direct shear device are compared. Some of the factors responsible for the interface shear strength data are discussed. There is a brief overview on pore water pressures generated on the interface.

2.1 Devices for Measuring Interface Shear Strength

The shear strength characteristics of landfill interfaces are investigated in direct shear devices. The devices used for measuring geosynthetic/geosynthetic and geosynthetic/soil interface shear strengths include the tilting table, the ring shear apparatus, and the direct shear apparatus.

The direct shear apparatus is the industry standard device for measuring geosynthetic interface shear strengths, and is discussed in more detail.

2.1.1 The Tilting Table Apparatus

In the tilting table, a geosynthetic is attached to a hydraulically activated tilt table, while the other geosynthetic or soil is attached to or located within a solid block that rests on a horizontal table. A normal stress is applied using dead weights or by means of a block or the self weight of the soil in the box. The table is kept horizontal while a desired normal stress is applied. The table is then inclined at a constant rate about a fixed point until the block just begins to move. The angle of inclination is increased until the box fully slides.

The angle of inclination of the table at the point where sliding initiates is equal to the angle of static friction for the interface (i.e. peak shear strength).

A schematic of the tilting table is presented in Figure 2.1.

Investigating the stability of geosynthetic landfill capping systems
The tilting table apparatus allows for testing under very low normal stresses. It is, however, not possible to measure post peak behaviour with this device. Table 2.1 highlights the advantages and limitations of using this device.
Table 2.1  Devices for Measuring Interface Shear Strength (after Marr 2001)

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Direct Shear</td>
<td>Equipment readily available and not expensive.</td>
<td>Horizontal displacement to 7.5 mm maximum due to small size.</td>
</tr>
<tr>
<td>(ASTM D-3080)</td>
<td>Specimen preparation and set up is relatively easy</td>
<td>Cannot measure strength of materials with high cohesion at low normal stresses</td>
</tr>
<tr>
<td></td>
<td>Can test undisturbed soil samples</td>
<td>Limited to materials with geometrical features (particle sizes, reinforcement spacing, texturing) less than 1/10 of box size (6-10 mm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Failure plane is fixed so measured strength may be higher than for field conditions.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cannot control drainage so must run slowly.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cannot test undrained conditions.</td>
</tr>
<tr>
<td>Large Direct shear (0.3m)</td>
<td>Can test materials with geometric features up to 25 mm.</td>
<td>Shear displacement may not reach residual.</td>
</tr>
<tr>
<td>ASTM D-5321 and D6423 BS6906</td>
<td>Can induce shear displacements of 75-100 mm</td>
<td>Friction may be relatively large at low normal stresses.</td>
</tr>
<tr>
<td></td>
<td>Boundary effects are reduced.</td>
<td>Applied normal load may not get to the failure plane.</td>
</tr>
<tr>
<td></td>
<td>Equipment is readily available</td>
<td>Failure plane is fixed so measured strength may be higher than for field conditions.</td>
</tr>
<tr>
<td></td>
<td>Can test undisturbed soil samples</td>
<td>Significant area change at large displacements changes the vertical and shear stresses.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cannot measure strength of materials with high cohesion at low normal stresses.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cannot control drainage so must run slowly.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cannot test undrained conditions.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>More expensive than standard direct shear test.</td>
</tr>
<tr>
<td>Tilt Table</td>
<td>Large displacements possible</td>
<td>Normal stresses less than about 25 kN/m² only.</td>
</tr>
<tr>
<td></td>
<td>Failure plane is not forced</td>
<td>Measures only peak strength</td>
</tr>
<tr>
<td></td>
<td>Can test multiple interfaces at the same time.</td>
<td>Limited availability</td>
</tr>
<tr>
<td></td>
<td>Can test a constant shear stress over long time easily.</td>
<td>Considerable effort to use equipment</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cannot test hydrated or undrained condition.</td>
</tr>
<tr>
<td>Ring Shear Device</td>
<td>Unlimited continuous shear displacement.</td>
<td>Not widely available.</td>
</tr>
<tr>
<td></td>
<td>Area of shear plane remains constant.</td>
<td>Complex specimen preparation procedures.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Small specimen size</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Forced failure plane location</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Difficult to prepare specimens to field conditions.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Non uniform shear displacement within sample</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Direction of shear constantly changes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cannot test undrained condition.</td>
</tr>
</tbody>
</table>
2.1.2 The Ring Shear Apparatus

The ring shear apparatus (RSA), shown in Figure 2.2, has been used for assessing friction mobilised at the interface between soils and geosynthetics by many researchers including Stark & Poeppel (1994) and Jones (1999).

![Bromhead Ring Shear Apparatus (Bromhead, 1979)](image)

The RSA consists of a 5 mm thick annular sample of 70 mm and 100 mm inner and outer diameters, respectively, constrained between two concentric rings. The sample is vertically confined between two porous platens by a level loading system. Load is applied through a 10:1 lever arm. The base plate is rotated by a variable speed motor and gear driving through a worm drive, causing the sample to shear close to the top platen. Torque transmitted through the sample is reacted by a pair of matched proving rings bearing on a cross arm. Transducers are placed parallel with the proving ring dial gauges. Settlement of the upper platen during consolidation is monitored with a dial gauge to which a transducer can be attached, and connected to a data logger for automatic data acquisition.

The RSA has a maximum normal stress of 1000 kPa and a maximum shear stress of 500 kPa, with unlimited displacement. Significant relative displacement can be developed without reversing the shear direction. The test...
will eventually be fully drained regardless of the rate of strain used so the problem of measuring pore water pressures on the sliding surface does not exist. The advantages and disadvantages of using this device are briefly presented in Table 2.1 above.

2.1.3 Direct Shear Apparatus (DSA)

Over the years, various modifications have been made to the original Casagrande designed direct shear box. Design modifications are necessary for accurate and reliable determination of shear strength parameters. Some of the modifications have included changes in the top and bottom box sizes, variations in the top box designs (e.g. fixed or floating), increased distance of shear and adaptations for different normal loading systems.

Richards and Scott (1985) identified five types of shear box designs. The fixed, partially fixed, free, large base and central base designs are presented in Figure 2.3 below.

![Figure 2.3 Types of direct shear apparatus and geosynthetic fixation (Richards & Scott, 1985)]
The modern direct shear box consists of a square rigid metal box, with two halves in which the lower half is made to slide relative to the top half. Direct shear tests can be carried out in the standard 60 mm x 60 mm and 100 mm x 100 mm shear boxes, which are mainly used with soils. The disturbed samples are easy to prepare and the laboratory shearing technique is relatively simple.

The DSA is the most appropriate test device for evaluating geosynthetic interface shear strengths. The industry standard suitable for use with geosynthetics is the large direct shear device. It has test box sizes of 300 mm x 300 mm (top box) and 300 mm x 400 mm (bottom box), the latter longer in the direction of shear to provide a constant shear area. The normal load is applied vertically at the top of the sample via weights, hydraulic or pneumatic bellows. The lower box moves relative to the top box up to a displacement of just over 100 mm.

Figure 2.4 shows a schematic cross section through the one of the large direct shear boxes at Loughborough University.

Figure 2.4  Schematic cross section through a large direct shear device (Jones 1999)
It is a Brainard Kilman Model LG112 comprising of a 305mm x 305mm top box and a 305mm x 406mm lower box. It is able to achieve a maximum travel of 102mm with no loss in shear plane. The top box is fixed to the frame of the shear box while the lower box is allowed to displace horizontally.

Normal stress is applied using compressed air in flexible rubber diaphragm with an air regulator attached to the airline which leads to the compressor. The lower box or travelling container is subjected to the same normal stress as the interface. The normal stress gauge is most accurate within the range of 50 – 500 kPa.

Horizontal displacement of the lower box is controlled by a DC motor control system and measured by a Linear Variable Differential Transformer (LDVT) which displays the displacement on a digital transducer readout. The shear force is measured by a load cell and displayed on a different digital transducer readout. The rate of shearing can be varied from 0.1mm/min to 7mm/min.

2.2 Fixed Top Large Direct Shear Apparatus

Most commercial large direct shear boxes for geosynthetic testing have fixed top box designs. A schematic of this device is shown in Figure 2.5.

![Figure 2.5 Schematic of a fixed top large direct shear device (Stoewahse, 2001)](image-url)
The top box is clamped onto the main frame of the shear device at all four corners to act as a reaction to the applied normal stress during shearing. The top box cannot move vertically, horizontally or sideways.

In the original design of the fixed device, it was assumed that because of a pre-existing or forced shear plane between the top and bottom boxes, no shear plane forms in the top box. Stoewahse (2001) has shown that shear planes do form in the soil in the top box above the geotextile/soil shear plane during shear.

Friction between the test material and internal walls of the top box both during application of the normal stress and shearing will alter the actual vertical stress acting in the shear plane by an unknown amount. At high normal stresses it is assumed that the friction between the side wall of the top box and soil is overcome due to the normal load transmitted on the top of the sample which forces a rearrangement of particles, leading to low side wall friction.

Machine friction develops between the travelling container and the remainder of the apparatus because the top box is subjected to the same normal load as the interface. Corrections must be applied to correct for this.

There are concerns regarding the magnitude of normal stress on the interface and its variation with time during shear. Floss & Fillibeck (1998) report a reduction of normal stress of about 20% using the fixed top device.

### 2.3 Floating Top Direct Shear Apparatus

This 300mm x 300mm large direct shear device manufactured by Wykeham Farrance Ltd, (England) has two loading systems, one for consolidation and one for shear.

A photograph of this device is presented as Figure 2.6.
The sample is consolidated via a hydraulic driver and sheared by a stepper motor drive unit. The device has a 100kN shear and consolidation force, a dial gauge of 25mm travel x 0.01mm for vertical displacement and another dial gauge 50mm travel x 0.01mm for horizontal displacement. The rate of shear can be varied between 0 – 10mm/min.

The Wykeham Farrance floating top DSA is ideal for testing shale, sand, brick rubble, industrial slag, colliery spoil, etc. It can also be adapted for testing geosynthetic materials e.g. geogrids, geomembranes and geotextiles.

2.4 Vertically Movable Direct Shear Apparatus

Research reported by Bluemel & Stoewahse (1998), Bluemel et al. (2000) and Stoewahse (2001) using different designs of top box of the large direct shear device, confirmed an effect on results of the design of the shear device, with fixed top devices yielding higher shear strengths for some interfaces.

The fixed top large direct shear device was modified to overcome the problem of unknown vertical stress along the shear plane. The modification included separation of the loading system from the top box. This allows the top box to
move vertically but not rotate. The average vertical stress acting on the interface can be determined by measuring the vertical support forces to the top box. Pressure applied at the top of the sample is electronically regulated to keep the resulting vertical force on the sample constant.

A schematic of the vertically movable top box design of the DSA is shown in Figure 2.7.

![Figure 2.7 Schematic of a vertically movable shear device (Stoewahse, 2001)](image)

**Figure 2.7  Schematic of a vertically movable shear device (Stoewahse, 2001)**

Stoewahse (2001)'s study on the sand/geotextile interface in both fixed and vertically movable devices showed variation of the normal force in the fixed top box. The observed variation of the normal force N in the fixed top box is due to friction forces acting at its internal wall. During shear, these forces are increasing and this affects the related shear force, T.

Results of direct shear tests in this device on sand are in good agreement with triaxial test results, while similar tests using a fixed top box give results significantly in error. Stoewahse *et al.*, (2002) conclude that the fixed top box design should not be used for tests wholly on soil. In the geotextile/sand interface, stretching of the geotextile seemed to affect the stress-displacement behaviour in some of the tests. This research led to the direct shear apparatus type with vertically movable top box being recommended for shear testing on soil and geosynthetic interfaces in Germany. The apparatus design has been adopted in the German DIN 18 137-3.
2.5 Interface Shear Strength Testing Standards

There are currently four internationally accepted standards available to use in the testing of interfaces for landfill lining and capping systems. The standards give guidance on test procedures as well as evaluation of measured data, beneficial to both designer and operator. They include BS 6901: 1991, ASTMD5321.92, GDA E 3-8: 1998 and the PrEN ISO 12957-1:1997.

Table 2.2 summarises the key elements of the test standards.

2.5.1 Status of current testing standards

BS6906:1991 is out of date and not suitable for performance tests. PrEN ISO 12957-1:1997 is the most current but only suitable for index tests. ASTM D5321:1992 and GDA E.3-8:1998 are more comprehensive and provide guidance for performance tests of various interfaces both in liner and cover systems, but need to be revised and updated.
Table 2.2: A Summary of the key elements of Interface test standards (Stoewahse et al, 2002).

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Scope</strong></td>
<td>Index tests + some guidance on performance testing</td>
<td>Index tests only</td>
<td>Performance tests</td>
<td>Performance tests</td>
</tr>
<tr>
<td><strong>Test Apparatus</strong></td>
<td>DSA 'about 300mm square'.</td>
<td>DSA minimum shear area 300mm square.</td>
<td>DSA minimum shear area 300mm square.</td>
<td>DSA minimum shear area 300mm square, for geosynthetics without surface structure and fine grained soil 100 mm square.</td>
</tr>
<tr>
<td><strong>Specific requirements of DSA</strong></td>
<td>$\alpha_s$ applied through rigid load plate. Measure vertical deformations. Design of box not specified.</td>
<td>Design should allow for sand dilation, $\alpha_s \pm 2%$. Fluid filled membrane systems allowed for application of $\alpha_s$. Measure vertical movement of loading plate at end of test.</td>
<td>$\alpha_s$ applied by device that maintains a constant uniform $\alpha_s$ for duration of test $\pm 2%$. Design should allow for soil deformations during shearing.</td>
<td>Design of DSA not specified.</td>
</tr>
<tr>
<td><strong>Number of Tests conducted</strong></td>
<td>9 tests in total, $\alpha_s = 50, 100$ and 200, kPa (3 tests at each $\alpha_s$). Highlights need to conduct tests in different directions and on different sides of geosyn.</td>
<td>4 tests in total, $\alpha_s = 50, 2 \times 100$ and 150 kPa.</td>
<td>Minimum of 3 $\alpha_s$, user defined. Test different directions and sides.</td>
<td>3 tests with 3 different normal stresses and 2 repeating tests with the mean value, which should match the expected normal stress in situ.</td>
</tr>
<tr>
<td><strong>Material Conditioning</strong></td>
<td>Sand and geosyn. 20° ± 5°C</td>
<td>Sand and geosyn. 20° ± 2°C</td>
<td>Soil and geosyn. 21°C ± 2°C</td>
<td>Soil mechanical laboratory conditions</td>
</tr>
<tr>
<td><strong>Method of fixing geosynthetics</strong></td>
<td>Clamp or glued to rigid sub-stratum</td>
<td>For geosyn. to rigid support to prevent any relative displacement between specimen and support (e.g. glue, friction support in shear area or clamped outside area).</td>
<td>Chamfered outside shear area or glued to rigid sub-stratum.</td>
<td>Recommendations about support and fixation of geosynthetics depending on the individual test case.</td>
</tr>
<tr>
<td><strong>Soil Properties</strong></td>
<td>Complying with fraction B (1.18mm to 600 $\mu$m) BS 4550</td>
<td>Standard sand in accordance with EN 196-1 (1.6mm to 0.08mm). Compacted w of 2% to $\rho_m=1.75\text{Mg/m}^3$. Performance tests, compact soil at $\alpha_{max}$ to 92 ± 2% Pmax.</td>
<td>User defined. Take care not to damage geosyn. during placement. Measure $p$ and $w$ after test.</td>
<td>Cohesive soils with not more than 33% $\rho_{sat} 'on the wet side' or as proposed by the landfill designer. Not less than 24 h preconsolidation time under normal stress equal to the test. Noncohesive soils compacted to medium density or as proposed by the landfill designer.</td>
</tr>
<tr>
<td><strong>Maximum particle size and Gap size (top/bottom base)</strong></td>
<td>Sand vs. geosyn. (index) not specified. Soil vs. geosyn. (performance) gap is $p_d/2$ or 1mm for fine grained soils. Maximum particle size &lt; 1/16th box depth. Geosyn. vs. geosyn. gap not specified.</td>
<td>Maximum particle size &lt; 1/16th box depth.</td>
<td>Maximum particle size &lt; 1/16th box depth.</td>
<td>Maximum particle size $d_{50}$ &lt; 1/16th of box length.</td>
</tr>
<tr>
<td><strong>Locations of materials in DSA</strong></td>
<td>Geosyn. vs. geosyn. rigid sub-stratum (i.e. not soil) Soil vs. geosyn. either rigid sub-stratum, geosyn. or soil in top box. Depth of soil layer not specified.</td>
<td>Geosyn. vs. geosyn. rigid sub-stratum in bottom box and sand in top box. Depth of sand layer = 50mm.</td>
<td>Geosyn. vs. geosyn. rigid sub-stratum (i.e. not soil). Soil vs. geosyn. supported by rigid sub-stratum. Soil either in top or bottom box. Depth of sand layer not specified.</td>
<td>Geosyn. vs. geosyn. rigid sub-stratum (i.e. normally no soil). Soil vs. geosyn. supported by rigid sub-stratum. Soil either in top or bottom box. Depth of soil layer not specified.</td>
</tr>
<tr>
<td><strong>Shearing rate</strong></td>
<td>Geosyn. vs. geosyn. and sand vs. geosyn. (index) 2mm/min. Soil vs. geosyn., variable rate depending on drainage.</td>
<td>Geosyn. vs. geosyn. 1mm/min.</td>
<td>Geosyn. vs. geosyn. 5mm/min if no material specification. Soil vs. geosyn., slow enough to dissipate excess pore pressures. If no excess pore water pressures expected use 1mm/min.</td>
<td>Geosyn. vs. geosyn. and non cohesive soil vs. geosyn., 0.167 to 1mm/min. Geotextile vs. cohesive soil 0.167 mm/min. Geosyn. liner vs cohesive soil 0.005 mm/min.</td>
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<td>Soil vs. geosyn., variable rate depending on drainage.</td>
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<td>Soil vs. geosyn., slow enough to disperse excess pore pressures.</td>
<td>Geotextile vs. cohesive soil 0.167 mm/min.</td>
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<td>If no excess pore water pressures expected use 1 mm/min.</td>
<td>Geosyn. liner vs cohesive soil 0.005 mm/min.</td>
</tr>
<tr>
<td>Derivation of shear strength parameters</td>
<td>Obtain $\phi$, $C_{u}$ from best fit straight line through all 9 points. Disregard any apparent adhesion ($a_{c}$) values.</td>
<td>Best fit straight lines through all points (peak and residual) to obtain $\phi$, $C_{u}$, $a_{c}$, and $a$. Failure envelopes defined by best fit straight lines to obtain shear strength parameters $\phi$, $C_{u}$, and $C$ intercepts.</td>
<td>Tests should be performed independently by a second institution. Best fit straight lines through all points (peak and residual) to obtain test values of $\phi$, $C_{u}$, $a_{c}$, and $a_{c}$. Derivation characteristic values. Disregard any apparent adhesion ($a_{c}$) values for noncohesive soils and for cohesive soils in special construction cases.</td>
<td></td>
</tr>
<tr>
<td>Specific reporting requirements</td>
<td>All plots and calculations. Describe failure mode. Report $q'$ of sand.</td>
<td>All plots and calculations. 'For comparison of index test results, all graphs and data have to be submitted to judgement of an engineer.' Description of 'post peak behaviour observed in each test'.</td>
<td>Detailed report about the test equipment, procedures and observations during testing, about the measured data and the further evaluation.</td>
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2.6 Factors Influencing Interface Shear Strength

Given the shear devices and testing standards for geosynthetic interfaces described above, the shear strength data obtained when tests are performed by different institutions often do not correspond to each other, generating complex discussions and raising various problems in defining reliable shear strength parameters for design. Various factors influence interface shear strength, but twelve are discussed below in detail.

a) Fixation of the geosynthetics

Blumel & Stoewahse (1998) emphasise the importance of the method of fixation of the geotextile to the rigid support, as insufficient fixation gave shear stress vs. displacement curves that did not have a peak value due to stretching of the geotextiles. Large direct shear devices are designed such that the geosynthetic can be fixed by clamping or gluing. Stretching of the geosynthetic is prevented by using a roughened block on top of the spacers in the lower box and a spacer.
bar along the clamp to further secure the geosynthetic. In the top box, the geosynthetic is clamped onto the top box and secured by a spacer bar. The geosynthetic should be cut a little wider than the outer dimensions of the top box that it is further secured in place by the weight of the top box and loading platen.

Due to the manufacturing process, geosynthetics can show different shear behaviour on either side and in the cross machine and machine directions. For the specific geosynthetic interfaces tested, Rankilor & Hieremans (1996) found the cross machine direction of the roll to have higher shear strength than the machine direction.

Most testing is carried out with the geosynthetic cut parallel to the direction of the roll. This is thought to be the direction of weakest strength and is the direction of slippage if the geosynthetic materials are rolled down the slope during installation.

b) Gap size between the top and bottom boxes

In the fixed top box, the gap between the top and bottom boxes must be set prior to shearing. As shown in Table 2.2, various testing standards recommend different gap sizes. For example, the ASTM D5321 (1992) allows the gap to be as small as $D_{85}$ of the sand, the PrEN ISO 12957-1:1997 allows $D_{85} + 0.5$ mm, while the BS 6906 (1991) stipulates $D_{85}/2$ or 1 mm for fine grained soils and does not specify a gap size for geosynthetic/geosynthetic interfaces.

Selection of a suitable gap size is crucial especially when soils are part of the interface. A big gap size might encourage significant extrusion of the soil during shear and may also increase the effects of side friction losses in the top box. The gap size must be sufficiently small so that no soil particles can migrate out of the box. Bemben & Schulze (1998) compared peak and residual stress ratios for two geomembranes and two sands with sand/geomembrane set-up. Shear device sizes ranged from 76 mm x 76 mm to 305 mm x 305 mm. The smallest gap size (3.7 mm) between the top and bottom boxes gave the highest peak stress ratio and the largest gap (7.6 mm) produced the lowest. They suggest 5.3 mm as a large enough gap size; this is about 3 mm larger than the
ASTM D5321 (1992) proposed size of $D_{45}$ of the sand. The small gap size recommended in the ASTM (1992) standard gave peak and residual stress ratio measurements that were too large or cyclic in nature. Bemben & Schulze (1998) clearly demonstrate that the gap size has a significant effect on measured interface shear strength.

The advice on gap size value from the test standards ASTM D5321 and BS6906 if used could lead to significant errors if additional considerations on the materials to be tested are not made. In tests with vertically movable and tilting top boxes, the top box may heave up to about 1 mm during shear but there is an immediate relief of constraints if the gap chosen at the start of the test is too small.

c) The nature of the geosynthetic including polymer hardness, manufacturing process, thickness, texturing, tensile strength and modulus

The effect of a polymer ageing or chemical degradation is difficult to quantify. Polymer softening improves the interface shear strength while an increase in hardness e.g. due to volatization of some of the plasticizers found in the geomembrane, would cause a decrease in the shear strength. Nataraj et al. (1995) investigated the interface friction parameters considering nine geosynthetics, six geomembranes, two cohesionless and one cohesive soil, using a direct shear device of 6.2 cm x 10 cm. Nataraj et al. (1995) found the interface friction to vary with the type, surface roughness and flexibility of the geosynthetics as well as the normal stress.

Snow et al. (1998) identified variability in shear strength resulting from geosynthetic material manufacturing processes e.g. geomembrane texturing, errors inherent in the testing procedure and from natural variations in the soil materials used. Lee et al. (1998) observed that the geomembrane surface topography affects shear resistance and the shear mechanisms operating at the interface. Jones & Dixon (1998a) recorded a particularly high apparent cohesion with the impinged textured geomembranes compared to blown film textured material.
d) **Thickness of soil layer in the top box**
Stoewahse (2000) tested a sand/geotextile interface in a 300 mm x 300 mm fixed top box and varied the thickness of the sand layer and the roughness of the loading plate he placed on top of the sand layer. A rough plate on a sand layer thicker than 50 mm showed no significant difference in the results. At thickness less than 50 mm, no peak values were observed. Rotation of the loading plate was observed at thickness less than 20 mm. In comparison, use of a smooth plate showed a noticeable difference in the results for a similar layer thickness. Stoewahse (2000) suggests a non cohesive sample thickness of at least 5 mm, a cohesive sample thickness of at least 30 mm and the use of a roughened loading plate.

e) **Dry/wet or submerged conditions of shear/cleanliness of shear surfaces**
The presence of water (even in the form of ice) at the interface is a major contributing factor in many recorded landfill cap failures. Saxena & Wong (1984) carried out direct shear tests on a smooth geomembrane versus dry and wet sand. Tests were conducted with vertical normal loads of 69, 138 and 207 kPa. They found that the shear strength of the saturated sand/geomembrane interface was lower than that of the dry sand for higher normal stresses.

Yegian & Lahlaf (1992) found that the interface shear strength of a HDPE geomembrane/geotextile interface could be reduced by perspiration from the touch of a dry hand. Residual friction values were reduced from 11° to 6°.

f) **Moisture Content/Consolidation/Drainage**
Mitchell *et al.*, (1990) found that the strength of the clay/geomembrane interface was affected by the presence of water at the interface. They tested a clay soil which was initially compacted at optimum-moisture density level then placed in water, allowing it to swell for 24 hours prior to testing. Testing conditions were similar to unconsolidated undrained conditions with no time allowed for pore pressure dissipation during loading and shearing. The same clay was sheared at optimum moisture-density condition with no water. They report a significant decrease in interface strength for the samples that were soaked prior to direct shear testing.
Pasqualini et al. (1993) investigated the effects of dry, wet and submerged conditions on a smooth LDPE geomembrane/compacted clay interface. Normal stresses of 66, 120 & 214 kPa were applied at a strain rate of 0.12 mm/min. Results indicate equal dry and submerged interface resistances for clay compacted at high water content. Pasqualini et al. (1993) also tested many wet and dry geomembrane/geotextile interfaces and found the wet interface strength values lower than the dry tests.

Floss & Fillibeck (1998) report that at normal stresses > 200 kPa, pore water and trapped pore air pressures develop within the clay directly after applying the load. Pore water and pore air pressures also influence the shear strength within the interface because they can decrease during the drained test whereas the shear strength increases compared to the undrained unconsolidated test. Clays are generally compacted on the wet side of the proctor-optimum for forming landfill liners, as this aids achieving the required permeability of \( \leq 1 \times 10^{-9} \) m/s.

Clayey soils if left submerged and unloaded will inevitably absorb water. This swelling leads to a reduction in interface shear strength. Consideration should be given as to whether the softened state is relevant for stability calculations. Floss & Fillibeck (1998) recommended the moisture content for the shear test slightly higher than the installation moisture content for landfill caps as field investigations have shown that the water content of the compacted clay liner will increase after installation. Stoewahse et al., (2002) recommend that interface shear strength tests be performed with submerged materials, unless special conditions have to be taken into consideration by the design engineers.

g) Type of soil (cohesive or non cohesive), particle size and angularity

Williams & Houlihan (1987) used the modified large direct shear box for tests on 35 interfaces which included 7 soils and 5 different geosynthetics, at the low normal stresses of 20 – 30 kPa, using a large (300 mm x 300 mm) shear box. The above authors found the interface friction angle to be influenced by the type of soil and its void ratio.
Jones & Dixon (1998) showed that grading, particle size and particle shape of overlying soils have a direct influence on the shear strength of a geomembrane/non-woven geotextile interface. The shear mechanisms and resulting peak interface behaviour of soils in contact with smooth geomembranes have been investigated by Dove & Frost (1999). Rolling and ploughing were the main mechanisms identified at peak.

The results of Dove & Frost (1999) show that the contact area of the soil/smooth geomembrane interface increases when the normal load is increased. Particle shape, roughness, normal stress and particle hardness, determine which mechanism controls the shear behaviour of a particular material combination and directly influence the shape of the conventional strength envelope.

**h) Temperature**

Research into the effect of temperature variations on clays has been performed by several investigators especially in connection with containment of high level nuclear wastes (Radhakrishna et al., 1984). Hueckel & Borsetto (1990) have showed that increased temperature leads to reduced shear strength.

Akber et al, (1985) subjected geomembranes to temperatures of -25°C to -5°C. The geomembranes became brittle and yielded low interface shear strengths when tested with soil. Pasqualini et al. (1995) obtained an increase in the residual interface shear strength for three different geomembrane/geotextile interfaces carried out at a higher temperature (29°C to 39°C) compared to those at 26°C. The temperature conditions of the test must therefore be reported. Ideally, tests should be carried out in a temperature controlled environment (20°C ± 2°C) and using materials conditioned to this temperature.

**i) Rate of shear**

A rate of shear is stipulated by each of the available testing standards. In practice, the rate of shearing a soil interface is usually lower than that of a geosynthetic/geosynthetic interface. A low shear rate allows for the dissipation of excess pore water pressures.
Williams & Houlihan (1987) found that in the range of 0.03 to 3.0 mm/min, the geosynthetic/geosynthetic interface friction parameters were relatively insensitive to the strain rate. Saxena & Wong (1984) used a shearing rate of 0.76 mm/min for a saturated geomembrane/sand interface. This rate was considered low enough to suggest drained behaviour. The shearing rates specified in the standards for geosynthetic/geosynthetic and sand/geosynthetic tests are regarded as appropriate. Stark et al (1996) performed ring shear tests on a geotextile vs. geomembrane interface and showed that peak shear strength values are independent of shearing rate for the range 0.03 mm to 40 mm per minute. Bluemel and Stoewahse (1998) found that for the geotextile/sand interface, the rate of displacement did not affect the interface shear strength to a great extent.

Performance testing requires that absolute shearing rates are specified according to the anticipated critical conditions on site (i.e. drained or undrained). Effective strength parameters are often required in design but drained tests involving cohesive soils do take a very long time and so are rarely carried out. The German guideline, GDA E 3-8 however stipulates drained tests for cohesive soil/geomembrane interfaces.

**J) Strain along Interface**

Post peak reductions in strength with displacement are characteristic of geosynthetic interfaces. It is usually difficult to determine residual interface shear strengths because large displacements may sometimes be required to reach the final residual strength. Residual shear strengths are often significantly less than the peak shear strengths (Mitchell, 1993). During and after construction of a landfill cover, both peak and residual shear strengths will be mobilised. Peak shear strengths are generally construction-induced. Design of capping slopes may, therefore, require the use of residual or large displacement shear strengths.

The actual strengths in the field are generally a combination of peak and residual strengths on the interface. Jones (1999) suggests that even the 100 mm obtainable from the large direct shear device may not be sufficient to mobilise the true residual interface shear strengths. Careful consideration of the
consequences of having only residual strengths available in the field then poses the challenge of obtaining reliable residual strengths from laboratory tests.

**k) Normal stress at the interface**
The shear strength at an interface is directly proportional to the normal stress. The applied normal stress must be given time to stabilise before shearing begins to ensure an even distribution in the given shear area. A 'seating' time of at least 10 minutes is deemed sufficient. Depending on the test device, the normal stress is applied to the top of the sample with dead weights, or pressure bags (filled with air or air and water). A stiff loading plate is used for load distribution over the test area. The common assumption is that the constant vertical stress derived from the load applied to the top of the sample is acting on the interface.

In tests on geosynthetics/soil interfaces using the fixed top device, Walter (1998) used load cells close to the interface to measure the normal stress at the interface. Walter (1998) noted an increasing difference between the normal stress applied at the top of the sample and the normal stress at the interface of up to about 50 kPa. This difference was shown to decrease with increasing normal stress at the top of the sample.

In tests on the geomembrane/clay interface, Floss & Fillibeck (1998) demonstrated that under very low normal stresses the high internal shear strength of the compacted clay could not be transmitted to the geomembrane because there was no intimate contact between the geomembrane and the clay. Intimate contact improved with increasing normal stresses, progressively developing high shear strength properties.

Generally, as normal loads increase, the shear strength envelope for a material or interface between two materials will tend to become more linear, yielding a curvilinear envelope, which will give lower available shear strength at higher normal loads than would be estimated by extrapolation from testing at lower normal load. Interface shear strength tests for higher normal stress designs carried out at lower normal stresses may over estimate the available shear
strength. The reverse is true for high normal stress test data extrapolated to obtain strength at low normal stress.

A constant and measurable normal stress at the interface enhances the reliability of interface shear strength data. This is intrinsic in the design of the vertically movable shear device (Stoewahse, 2001).

L) Equipment style and dimensions
Blumel & Stoewahse (1998) report a testing programme involving 20 laboratories, in which they investigate the shear strength between a geotextile/textured geomembrane and standard sand/geotextile interfaces. Up to four different designs of the top box were used for the same interface and test conditions. The devices used had differing modifications and were from different manufacturers. All the devices used had interfaces of at least 300 mm². The laboratories were experienced in measuring geosynthetic interface shear strengths. The tests were conducted dry at a constant rate of 10mm/hr for normal stresses of 20, 50, 100 & 200 kPa. There was an unsatisfactorily high scattering of the data presented from the different institutions for the same interfaces.

The results for the sand/geotextile interface presented in Figure 2.8 are for trials undertaken first in 1995 and repeated in 1996 with revised testing procedures. Each colour and line style represents one laboratory. The results show widely varying shear strength-displacement curves with different peak and residual shear strengths for the same material at one normal stress.

The shear stress vs. normal stress plots for these results are presented in Figure 2.9. The results show a reduced but nonetheless still significant scattering of test data in 1996.
Figure 2.8 Shear stress displacement curves sand/geotextile interface a)1995 and b)1996 (Stoewahse, et al 2002) 
(Each colour and line style represents one laboratory.)

Figure 2.9 Peak shear strength v. normal stress plots (Bluemel & Stoewahse, 1998)
2.7 Typical Geosynthetic Landfill Capping Materials

Geosynthetics are defined by the International Geosynthetics Society (IGS) as 'planar, polymeric (synthetic or natural) materials used in contact with soil/rock and/or any other geotechnical material in civil engineering applications. There are various types of geosynthetic materials including geomembranes, geotextiles, geocomposites, geogrids, geonets, geosynthetic clay liners, geopipes, etc. Their uses are varied and include separation, drainage, gas and hydraulic barriers, filtration, protection, reinforcement, soil erosion, etc.

For the purpose of this study, this section will concentrate on the materials typically used in landfill capping systems namely, Linear Low Density Polyethylene (LLDPE geomembranes, non woven geotextiles and geocomposites.

2.7.1 LLDPE Geomembranes

Geomembranes are relatively impermeable polymeric sheets generally used as barrier layers in the place of or in combination with a relatively impermeable mineral layer (clay). Geomembranes can be manufactured smooth or textured (with a roughened surface). Textured geomembranes have a higher friction surface compared to smooth geomembranes.

Geomembranes manufactured from virgin Linear Low Density Polyethylene (LLDPE) resins are extremely flexible, with high elongation, tremendous tear resistance and bursting strength. LLDPE geomembranes are preferred to HDPE (High Density Polyethylene) geomembranes for use in landfill covers mainly because they will conform to the new profile (before, during and after settlement) without a significant influence. In addition, they provide an alternative to clay liners that is more economical and quicker to install.

LLDPE geomembranes are usually installed in combination with geotextile protection layers. A typical manufacturer’s Data Sheet is presented in Appendix A.
2.7.2 Non woven Geotextiles

According to the IGS, non woven geotextiles are planar, permeable textile materials in the form of a manufactured sheet, web or batt of directionally or randomly orientated fibres, filaments or other elements, mechanically and/or thermally and/or chemically bonded. Geotextiles are mainly used as protection layers for geomembranes and as filtration layers.

In capping systems, geotextiles are mainly placed in combination with geomembranes, sand and sometimes clay. A typical manufacturers’ data sheet for non woven geotextiles is presented in Appendix A.

2.7.3 Geocomposites

Geocomposites have been defined by the IGS as manufactured or assembled material using at least one geosynthetic product among the components, and used in contact with soil/rock and/or any other geotechnical material in civil engineering applications.

In capping systems, geocomposites are used as synthetic drains designed for collection and removal of surface water and for landfill gas venting. They are mainly placed in combination with textured geomembranes, sands or restoration cover soil layers.

The critical engineering properties of a geocomposite include:

- The interface friction with adjacent soil and/or geosynthetic material layers
- Internal shear strength
- In-plane flow capacity of the geocomposite under design loads and boundary conditions, and
- Filtration characteristics of the upper geotextile relative to the soil retained by it or the liquid being allowed through it.

Geocomposite placement on caps is typically down slope in the direction of the roll. Placement at slope corners and in confined spaces does not normally necessarily follow the same pattern. The influence of the orientation of the geonet on its frictional properties has been observed by De & Zimmie (1998).
Effective and efficient capping design will require a good understanding of the various factors that may influence friction at the various interface combinations, geosynthetic placement being one of them. A typical geocomposite manufacturer's data sheet is presented in Appendix A.

2.8 Role of PWP on the Geosynthetic Interface:

One of the objectives of the capping system is to manage leachate production by controlling the ingress of rain and surface water into the underlying waste, and preventing any uncontrolled discharge or escape of perched leachate upslope.

Figure 1.1 shows that the drainage layer overlies the hydraulic/gas barrier in the cover system. The drainage layer and the effectiveness of the drainage system are vital to controlling infiltration of water into the waste mass. The geomembrane drainage barrier unless damaged, is completely impermeable to leachate from below it or infiltrating water from above, thereby preventing the downward migration of liquids. This could cause a build up of pore pressure.

Over time, the excess pore water pressure may dissipate, but the rate at which it dissipates depends on the permeability of the surrounding materials and the overall efficiency of the drainage system. Failure could occur along the geomembrane barrier layer if the excess pore water pressures do not dissipate.

Extensive testing has been conducted by various authors including Rankilor & Hieremans (1996) and Dixon et al., (2000) on the shear strengths of soil/geosynthetic and geosynthetic/geosynthetic interfaces that are common in modern landfills. The data generally suggests that the shear strengths of these interfaces are typically lower than the shear strengths of waste or soil and that the presence of liquids may reduce interface shear strength.

2.8.1 Shear Strength & Effective Stress

Shear strength depends fundamentally on effective stresses. The principle of effective stress, presented in Equation 2, strictly applies only to fully saturated soils.

\[ \sigma' = \sigma - u_w \]

Equation 2
Where \( \sigma' = \text{effective stress} \)

\( u_w = \text{pore water pressure} \)

\( \sigma = \text{total stress} \)

The Coulomb equation was modified by Terzaghi as shown:

\[
\tau' = c' + (\sigma - u_w) \tan \phi' \\
\tau' = c' + \sigma' \tan \phi'
\]

Equation 3
Equation 4

Where \( \tau' = \text{shear stress on the plane (in terms of effective stress)} \)

\( \sigma' = \text{effective stress normal to that plane} \)

\( u_w = \text{pore water pressure} \)

\( c' = \text{apparent cohesion} \)

\( \phi' = \text{angle of shear resistance} \)

This equation suggests that if pore water pressure \((u_w)\) is positive, \(\sigma'\) is reduced, thus effective shear strength is reduced. Pore water pressures influence stability by modifying the effective stresses within the materials and at interfaces between geosynthetics and soils.

Capping systems generally have low normal stresses. The presence of water at the critical interface could quickly lead to a slope failure if there is no sufficient in-plane drainage system for fairly rapid pore water pressure dissipation.

### 2.8.2 The Clay/Geomembrane Interface

For a geomembrane to be placed directly above a highly saturated cohesive mineral sealing layer, the consolidation of the soil has to be considered. Applying stress to the mineral sealing layer will lead to its consolidation under discharge of pore water. Excess pore water pressures are dissipated during consolidation. The magnitude of the excess pore water pressure depends on the
permeability of the soil, degree of saturation of the soil, proximity to a drainage layer, the magnitude of load, the rate of loading and time allowed for dissipation in relation to the rate of loading.

If a geomembrane is placed directly on top of a saturated cohesive soil and further layers are constructed on top of the geomembrane, the mineral layer undergoes consolidation as a result of loading. Excess pore water pressures are dissipated during consolidation. In practice, compacted soils are not saturated and this influences the magnitude of generated pore water pressures in response to loading and also influences the rates of dissipation of excess pore water pressures.

Marr (2001) has suggested that clays placed wet of the standard Proctor optimum moisture content and subjected to normal stresses above 100 kPa tend to generate positive excess pore pressures during shear, whereas clays placed dry of the standard Proctor optimum moisture content and those that become dessicated by wet-dry or freeze-thaw cycles tend to generate negative pore pressures. Depending on the weather and time of year of placement, clays tend to dessicate in landfill caps and also during the construction stage of the liner/side slopes.

The pressure of the pore water within the soil specimen is measured by means of a pressure transducer mounted as close as possible to the non-draining face of the specimen. The pressure transducer can also be embedded in the sample, close to the shear zone.

Typical results for normally consolidated and over consolidated clays are described by Craig (1999). Normally consolidated clays experience increases in pore water pressures during shearing while over consolidated clays experience an initial increase in pore water pressure followed by a decrease. Excess pore water pressures generated during shearing decrease as the over consolidation ratio increases.
2.8.3 Measurement of Pore water Pressure

Effective stress tests for shear strength with the measurement of pore water pressures have traditionally been carried out in the triaxial cell. Methods of measuring the pore pressure in the triaxial apparatus have been developed which are adequate for many research investigations and routine testing purposes.

The procedure for effective stress tests in the triaxial apparatus has been described in BS1377-8, 1990. Drainage conditions during triaxial compression are described by Head (1998). Figure 2.10 shows the various connections for pore water pressure measurement in the triaxial cell.

![Diagram of triaxial cell connections](image)

**Figure 2.10 Connections to the Triaxial Cell for effective stress tests (Head, 1998)**

Measurement of pore water pressures on a soil/soil or soil/geosynthetic interface in the DSA is a challenge because the sample is not in a closed system.

2.8.4 Types of Pressure Transducers

A pressure transducer is a transducer that converts pressure into an analog electrical signal. The strain-gage base transducer is the most common type of pressure transducer. Conversion of pressure into an electrical signal is achieved by the physical deformation of strain gages which are bonded into the diaphragm of the pressure transducer and wired into a Wheatstone bridge configuration.
Pressure applied to the pressure transducer produces a deflection of the diaphragm which introduces strain to the gages. The strain will produce an electrical resistance change proportional to the pressure.

Pressure transducers are generally available with three types of electrical output; millivolt, volt and 4-20mA (Iomega Inc, USA). A summary of the outputs and when they are best used is presented below:

- **Millivolt Output Pressure Transducers:** These are the most economical, with an output of around 30mV. The output is directly proportional to the pressure transducer input power or excitation. If the excitation fluctuates, the output will fluctuate too. These transducers need a regulated power supply and an electrically noise-free environment. For best results, a relatively short distance should be kept between the transducer and the readout instrument.

- **Voltage Output Pressure Transducer:** These include integral signal conditioning which provides a much higher output than a millivolt transducer (0-10Vdc). They can cope with unregulated power supplies as long as they fall within a specified power range. They are not as susceptible to electrical noise as the millivolt transducers and can, therefore be used in industry.

- **4-20mA Output Pressure Transducers,** better known as pressure transmitters. A 4-20mA signal is least affected by electrical noise and resistance in the signal wires. They are used when the signal must be transmitted over long distances.

**2.8.5 The Pore Pressure Transducer**

A schematic of the pore pressure transducer (PDCR81) manufactured by Druck Inc., Leicestershire, England, is shown in Figure 2.11. This type of transducer is commonly used in triaxial experimental set ups which involve pore water pressure measurement. A copy of the manufacturer’s specification is presented in Appendix A.
The transducer consists of a thin diaphragm on which electric strain gauge circuits are bonded and mounted in a rigid cylindrical housing. A porous filter protects the diaphragm, allowing it to be influenced by the pressure of the water. The resulting diaphragm deflection, though extremely small, gives rise to an out-of-balance voltage, which is amplified and converted to a digital display in pressure units.

![Schematic of the miniature pore pressure transducer PDCR81](image)

**Figure 2.11 Schematic of the miniature pore pressure transducer PDCR81 (Kutter et al., 1990)**

The response time of the transducer depends on the volume of water required to cause the small diaphragm movement. The physical displacements at the centre of the small diaphragm take place when pressures are applied to one side of it.

The centre of the diaphragm is linked to a group of four bonded strain gages of equal length; two of which increase in length and the other two decrease in length as pressure is applied to the diaphragm. The four strain gages are connected in the form of a Wheatstone bridge. This is so that any change in length, and consequently in the resistances in these gages will alter the electrical balance of the bridge. The result is a change in electrical signal in the output circuit, which can be calibrated against a known applied pressure.

The connection between the specimen and the transducer must be filled with de-aired water (produced by boiling water in a near vacuum) and the system should undergo negligible volume change under pressure. The presence of air in the
system or the use of a flexible connection between the sample and the transducer greatly increases the compliance of the system. The compliance is the volume displacement per unit change of pressure.

Kutter et al., (1990) used the PDCR81 and demonstrated both theoretically and experimentally that measurements of pore pressure transducers could be strongly influenced by stress concentration or arching around the transducer. Transducer readings overshot the true 'free field' pore pressure depending on the orientation of the transducer's porous stone relative to the direction of applied stress.

Kutter et al, (1990) found that the response time of a transducer depended on the compliance of the transducer, the surface area of the porous stone and the permeability and compressibility of the surrounding soil. Kutter et al, (1990) recommended that the transducer be mounted as close as possible to the sample and that the whole system be thoroughly de-aired, to enhance rapid transducer response to changes in pressure.

2.8.6 The Large Direct Shear Device for Effective Stress Tests

Drained and partially drained tests in the large direct shear apparatus require that both the soil and geosynthetic are conditioned (i.e. set to the right moisture content to reflect the expected worst conditions) and tested at a given temperature. The sample is normally hydrated and a seating load applied for a specified length of time. Marr (2001) has suggested different hydration and consolidation duration times for the various interfaces.

To obtain effective stress parameters in the direct shear device, the rate of shear of interfaces involving soil has to be extremely slow to allow for dissipation of excess pore water pressures. Hence, it is assumed that pore water pressures at the interface are zero.

Alternatively, the direct shear device could be specially adapted for measuring pore water pressures either at the interface or within the soil both during consolidation and in shear. The rate of shear could be increased if pore pressures could be measured on the interface during shear. The Author is not
aware of any work on the measurement of pore water pressures during direct shear in the large direct shear device.

2.9 Landfill Cover Stability Analysis

To evaluate the stability of the capping system, the shear strength parameters must be determined for each soil/geosynthetic, soil/waste and geosynthetic/geosynthetic interface in the capping system. This is in addition to analysing the landfill geometry and including for landfill gas pressure, water pore pressures, earthquakes and equipment loading during construction, in the slope stability analysis.

2.9.1 Use of Peak or Residual Shear Strengths

Stark & Choi (2004) recommend that the stability of landfill cover systems be analysed using peak shear strength of the weakest interface, or if necessary the weakest composite interface with a factor of safety greater than 1.5. The peak interface strength is recommended for the cover system because of the lack of or limited amount of large detrimental shear displacement along the weakest interface compared with a liner side slope. This can generate post peak strengths.

In cases where the average slope angle of the cover system is greater than the friction angle of the weakest interface, or large displacements such as construction-induced displacements or seismically induced displacements are expected, Stark & Choi (2004) suggest the use of residual shear strength with a factor of safety greater than 1, thus highlighting the importance of site specific testing.

2.9.2 Effect of Landfill Gas

In a municipal solid waste (MSW) landfill, landfill gas is generated as the waste decomposes. Even where gas extraction systems exist, the gas is not necessarily extracted at the rate at which it is generated. For covers incorporating geomembranes, there is the concern that an uplift pressure can be caused by the gas.
From a slope stability point of view, gas pressure is an excess pore pressure that serves to reduce the effective normal stress beneath the geomembrane cover. To prevent this, gas must be properly vented so that the geomembrane cap does not act like a balloon. A gas collection system including a geocomposite gas transmissivity layer, pipes and wells to withdraw gas can provide a gas escape route. The method for designing gas venting layers below landfill final covers has been developed by Thiel (1999).

2.9.3 Effect of Infiltrating Water
Water that accumulates above the geomembrane layer serves to increase the weight of the soil above the geomembrane and reduces the effective stress and shear resistance of the interfaces and materials above the geomembrane. The normal stress applied by the cover soil is low, usually equivalent to ~1m of soil, therefore, small liquid increases above the geomembrane can quickly cause instability. The effect of water on stability of the capping slope can be modelled using the pore water pressure ratio (Ru).

2.9.4 Effect of Shear Strength of Waste
The shear strength of waste is particularly important in capping systems because differential settlement of waste affects the integrity of the geosynthetic liner and may cause shearing in the mineral liner. Differential settlement of waste also affects the integrity and performance of the gas and leachate wells. Dixon & Jones (2004) have summarised the measurement and interpretation issues for the key engineering parameters used to define MSW. The appropriate values of shear strength of waste and Ru to be adopted in design are given in an Environment Agency Technical Report TR1 (2003).

2.9.5 Incorporating Equipment Loads
Koerner & Daniel (1997) recommend that cover soil placement on a slope with a relatively low shear strength component like a geomembrane, should always be from the toe upward to the crest. In this way, the gravitational forces of the cover soil and live load of the construction equipment, are compacting
previously placed soil and working with an ever-present passive wedge and stable lower portion beneath the active wedge.

Kerkes (1999) and Jones et al, (2000) have assessed the effects of equipment loading during construction, on the stability of landfill lining systems. These effects can be included in the slope stability calculations during the design stage.

2.9.6 Selection of Appropriate Material Factors
Limit equilibrium calculations can be carried out using a global factor of safety (traditional approach) and using partial factors on both resisting and disturbing forces (limit state approach, Eurocode 7: 1997). When using the global factor of safety (traditional approach) conservatively chosen mean values of shear strength are used.

In current engineering design practice, the limit state approach is used to assess the performance of a given geotechnical structure. Failure is defined in terms of ultimate limit state e.g. slope failure and serviceability limit state e.g. damage of liner due to waste settlement.

The Eurocode 7 (1997) approach involves obtaining characteristic values of shear strength. The material properties obtained have to be factored for conventional design situations. Where water pressures are encountered, the design incorporates the worst imaginable but unfactored value. For practical purposes, it can be assumed that the characteristic value (EC7) and the conservatively chosen value are equivalent (Schneider, 1997). Schneider( 1997) has proposed a statistical approach for determining the characteristic value using the mean value of the test results and the standard deviation of the test results .

When selecting whether to use peak or residual shear strengths, it is important to understand that the residual strength controlling stability of the whole lining system is not the lowest residual strength but the residual strength for the interface with the lowest peak strength (Gilbert, 2001).
2.9.7 Methods of Capping Stability Analysis
In a proposed limiting equilibrium stability analysis methodology suitable for capping systems, Jones & Dixon (1998b) adopt the Soong and Koerner (1996) equation but include the effect of a cover soil with cohesion, $c$ and an interface with a cohesion intercept of $\alpha$, which results in a change in the $b$ and $c$ terms of the quadratic equation, a closed form calculation. Jones & Dixon (1998b) suggest that the stress normal to the interface used in the calculation of the geoysynthetic tensile force should take account of the piezometric surface.

Jones & Dixon (1998b) propose that the stability of a cover soil over several layers of geosynthetics together with the tension developed in the geosynthetics can be obtained by:

- Calculating the factor of safety against cover soil sliding using the approach of Soong & Koerner (1995), modified to allow for $c$ & $\alpha$.
- Calculating the mobilised tension in the upper geosynthetic using Bourdeau et al (1993) with the modification for $\gamma_{sat}$ and $\gamma_d$. (where $\gamma_{sat}$ is the saturated unit weight of the cover soil and $\gamma_d$ is the dry unit weight of the cover soil).
- Calculating the mobilised tension in the remaining geosynthetics.

For tension (T) in the geosynthetics, if 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 upper geosynthetic to the lower geosynthetic with no tension induced in the upper geosynthetic. The calculation can be re-run to see if there is tension in the lower geosynthetic. If this tension is also negative, the lower geosynthetic can transfer the shear to the underlying materials without any tension (Jones & Dixon 1998b).
2.10 Influence of Site Practices

All possible mechanisms that could result in mobilisation of post-peak and residual strength conditions for the interfaces should be considered by the designer and care taken to minimise them on site.

Mechanisms for controlling post peak strengths have been identified by Dixon & Jones (2003) to include:

- Construction related activities e.g. dragging geosynthetic materials over one another in the process of positioning, construction plant loads, compaction of fine grained soils above geosynthetic layers, improper storage and handling of geosynthetics leading to loss of internal strength and
- Activities associated with landfill operations e.g. placement of veneer soil layers, short and long term settlement, which may cause creep and degradation of geosynthetics, and differential settlement of waste beneath a cap for landfill covers.

Dixon & Jones (2003) suggest a stringent construction quality assurance to control the method of placement to minimise any dragging of the geosynthetic, to specify a minimum soil cover over geosynthetics before being trafficked by limited plant, to specify methods of soil placement on slopes, minimise vehicle operations e.g. braking, control handling and storage of geocomposite materials so that internal strength is not compromised.

2.11 Summary

This section summarises the contents of the literature review by highlighting the key points and exposing the relevant areas where there is a lack of knowledge.

2.11.1 Key Points

- The large direct shear device is the most commonly used device for measuring interface shear strengths. Twelve factors controlling the measured interface shear strength have been identified and include the design of the direct shear device (e.g. design of the top box), the test set up and procedure (temperature, rate of shear, gap
size, loading plate, clamping, normal stress, etc), variability of the material (e.g. type of geosynthetic, direction of shearing, number of tests, etc) and in case of soils, volume changes, density, porewater pressure, consolidation, particle size, drained/undrained shearing, etc.

- For measuring interface shear strength, four standards are currently available. They give guidance on test procedures as well as evaluation of measured data, beneficial to both designer and operator.

- Modifications to the design of the large direct shear device to allow the top box to move freely vertically give better interface shear strength results.

- The interface or interfaces controlling stability should be identified from site specific interface shear strength tests. For design, suggestions from literature indicate the use of the peak interface shear strength of the weakest interface or the weakest composite interface with the factor of safety greater than 1.5. If the average slope angle of the cover system is greater than the lowest peak interface friction angle or construction-induced or seismically induced displacements are expected, a residual interface friction angle should be used for design. The integrity of the protection and drainage layers is particularly critical.

- Selection of design parameters for a stability analysis requires project specific engineering properties and configuration to accurately model the anticipated field conditions. The primary concern is the long-term stability of the cap, which requires an effective stress approach. Undrained failure is unlikely to occur because the drainage layer minimises the possibility of hydrostatic pressure build up in the cap. However, the shear strength between the various interfaces e.g. between geosynthetic/soil or geosynthetic/geosynthetic remains a key factor in cover system stability.

- Pore water pressures influence stability by modifying the effective stresses within the materials and at interfaces between geosynthetics and soils. Pore water pressures have been traditionally measured in the triaxial apparatus.

- In evaluating the stability of a capping system, the material properties, type of stability analysis to be performed, surface and subsurface drainage systems and pore...
pressure development must be considered. The forces responsible for instability in slopes and masses must especially not be ignored when incorporating geosynthetics.


- Construction and landfill related activities should be considered and controlled in order to minimise the possibility of mobilising large displacement (post peak) shear strengths.

2.11.2 Gaps in Knowledge

- The interface shear strength standards make no mention of the design of the top box of the shear device to be used, and the possible inaccuracies associated with using this device at low normal stresses relevant to capping systems.

- With the increasing use of geosynthetics in landfill capping systems, the Author has not come across any studies on the variability of geosynthetic interface shear strength data for tests conducted at low normal stresses, for design of geosynthetic capping systems.

- There are various types of geocomposites available on the market. These materials have been developed recently and are increasingly used in landfill cover slopes as drainage layers in place of sand or gravel, but there is hardly any research literature on their structural performance/shear behaviour.

- The author is unaware of any work involving the measurement of pore water pressure in the direct shear apparatus. Unlike the triaxial apparatus, drainage conditions in the large direct shear device cannot be controlled because the sample is not sealed within the shear box. The direct shear apparatus needs to be specially adapted for measuring pore water pressures either at the interface or within the soil
sample both during consolidation and shear, in order to obtain effective interface shear strength parameters.

These gaps in knowledge therefore justify the specific purposes of this research project.
3. Design of Laboratory Experiments

This chapter describes the research philosophy behind this study and validates the laboratory equipment and methods chosen for the measurement of interface shear strength tests and measurement of pore water pressure on the interface. The equipment and materials specifications are given together with the test procedures.

3.1 Research Philosophy

It was decided to use the large direct shear device for this study because it is currently the industry standard device for measurement of geosynthetic interface shear strengths. It is commonly used in laboratories in the USA and Europe and is rapidly gaining worldwide acceptance.

Repeatability interface shear strength testing is undertaken at a low normal stress range suitable for capping systems. At the commencement of this study, there was hardly any published work on interface testing, design and use of geosynthetics in landfill capping systems. The shear device used for the initial repeatability tests was manufactured by GeoDurham (USA) over 20 years ago. A new shear device from the same manufacturer was purchased in 1999, and used for the majority of tests conducted at Loughborough University. Following completion of the initial repeatability tests, the wide range of values obtained prompted a second series of tests. The device in this phase of tests is the brand new version of the one used in the preceding tests. This series of testing had additional special conditions including wetting and scratching on the smooth geomembranes. The aim of the investigation was to assess possible causes of scattering of interface shear strength data.

Reproducibility testing was carried out as part of an inter-laboratory British Council Academic Collaboration scheme between Loughborough and Hanover Universities. In this program, three large direct shear devices were used for...
these interface tests. The three shear boxes differ in the degree of freedom of movement of the top box. The fixed and floating top box designs were used at Loughborough University, and the vertically movable top box was used for tests carried out at Hanover University, Germany. The 100mm x 100 mm shear box manufactured by Wykeham Farrance, UK, was used for direct shear tests on Leighton Buzzard sand. The top box in this device also moves vertically in what could be referred to as a floating mechanism.

The main geosynthetic materials used in this study include LLDPE geomembranes, non woven geotextiles, and geocomposite drainage materials. These materials are typically used in landfill cover systems in the northern hemisphere.

Geocomposite drainage material is increasingly placed in landfill capping systems with little regard to the orientation of the core grid in relation to the direction of shear. Placement is usually down slope in the direction of the roll. This pattern is not necessarily followed at slope corners and in confined spaces, where the orientation of the grid can be any direction. This is bound to have an effect on the interface shear strength. The introduction of these materials also raises questions regarding internal stability, due to potential weak bonding between component layers (Bluemel et al., 1997). There is also presently a dearth of information on the interaction of these geocomposites in shear with other capping system members at the normal stresses associated with capping systems.

For the measurement of pore water pressure, the pore pressure transducer has been chosen because it is commonly used in geotechnical laboratories for pore water pressure measurement in the triaxial cell. It was considered that the use of three pore pressure transducers would give a more accurate value of pore water pressure on the interface. The adaptation of the pore pressure transducer for use in the large direct shear device is novel, and opens up possibilities of faster interface test times for saturated geosynthetic/soil interfaces.
The test programmes conducted are presented in Table 3.1.

Table 3.1  Interface Shear Strength Testing Programs

<table>
<thead>
<tr>
<th>Interface Test Program</th>
<th>Reason for Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Repeatability Tests</td>
<td>To ascertain the variability of interface shear strengths at low normal stresses</td>
</tr>
<tr>
<td></td>
<td>for the same interface, at the same normal stresses, using the same operator and</td>
</tr>
<tr>
<td></td>
<td>device.</td>
</tr>
<tr>
<td></td>
<td>To assess possible causes of scattering.</td>
</tr>
<tr>
<td>Reproducibility Tests</td>
<td>To ascertain reliability of interface shear strength data provided by use of</td>
</tr>
<tr>
<td></td>
<td>different designs of shear box, different laboratories and different operators.</td>
</tr>
<tr>
<td>Structural/shear behaviour of</td>
<td>To study the effect of the direction of the geocomposite grid on shear at low</td>
</tr>
<tr>
<td>Geocomposites</td>
<td>normal stresses</td>
</tr>
<tr>
<td>Measurement of PWP on a clay/geomembrane interface in the large DSA.</td>
<td>To confirm whether the clay/geomembrane interface is a drainage path</td>
</tr>
<tr>
<td></td>
<td>To obtain effective shear strength parameters</td>
</tr>
</tbody>
</table>

3.2  Equipment Specification

The shear box devices used for this study differ in the design of the top box. The designs of the shear devices include fixed top box (i.e. the top box cannot move vertically or rotate), tilting top box and vertically movable top box.

3.2.1 Fixed Top Large Direct Shear Device

Large direct shear devices with a top box plan area of 300mm x 300mm and a lower box area of 300mm x 400mm were used for these tests. The fixed top design is used worldwide for routine geosynthetic interface shear strength assessment, and is in accordance with the commonly referenced national test standards (e.g. BS 6906, ASTM D5321). This apparatus can be used for testing at normal stresses between 5 kPa and 600 kPa. The maximum horizontal travel is ~102 mm. A schematic of this device is shown in Figure 2.5.

The large direct shear device at Loughborough University was manufactured by Durham Geo (USA) and is a Brainard Kilman Model LG116 (Figure 3.1).
device provides a 305 mm x 305 mm shear zone for determining the interface friction characteristics geosynthetic/geosynthetic or geosynthetic/soil interfaces.

Figure 3.1  LG-116 Large DSA at Loughborough University (Durham Geo USA)

The device consists of the following:-

- A break-away box and drive system for enhanced efficiency
- A stainless steel tank for submerged sample testing to emulate field conditions
- The normal load is applied with a flexible bladder filled with compressed air, or a rigid platen, fitted to the upper box.
- A DC motor control system for precise speed control of applied force.
- Linear bearings to minimise horizontal movement friction.
- Data is displayed on four digital readouts for measuring load, displacement, settlement and pressure.
- Power, direction and speed controls, along with warning indicators protect the system from over-travel.
Pressure transducer with a quick connect to allow use with both a normal and fixed platen.

The fixed top shear box design challenges the accuracy of the capping system normal load application when the air bag is used. Its use in the range of normal stresses relevant for cover systems requires an assessment of the achievable accuracy and reproducibility of results. The airbag has been used for normal stresses >50 kPa, while the low load platen has been used for tests of normal stresses between 10 – 30 kPa.

For this study, normal stress was applied using a pneumatically operated piston reacting against the body of the shear device, and acting through a rigid load platen with the same plan area as the top box, also known as the Low Load System. The rigid low load system used with the fixed box incorporates a pneumatic loading device, a loading platen, a rolling neoprene diaphragm seal and a displacement transducer to measure the sample's vertical displacement.

The applied normal stress on top of the sample is kept constant, can be controlled to a resolution of 0.2 kPa and is recorded throughout each test. A displacement transducer is used to measure vertical displacement of the sample during application of normal stress and during shearing. Using a specialised data acquisition system, a personal computer logs this information at set time intervals.

3.2.2 Floating Top Large Direct Shear Device

The floating top device presented in Figure 2.6 is manufactured by Wykeham Farrance Ltd. The device was designed for use with granular soils. The top and lower boxes are both 300mm x 300mm.

For the geosynthetic/soil interface tests, the top and lower boxes of this device were adapted with clamps for fixation of the geomembrane and geotextile samples. The device was used in the reproducibility test series for the sand/geotextile and geomembrane/geotextile interfaces.
3.2.3 Vertically Movable Top Large Direct Shear Device

The vertically movable shear device, as schematic of which is shown in Figure 2.7, is a modification of the fixed box direct shear device, and is manufactured by Wille Fassertechnick of Germany. The device is designed to allow the average vertical stress acting on the interface during shear to be determined by measuring the vertical support forces to the top box. The pressure applied to the top of the sample is regulated during the test to keep the resulting vertical force on the interface at a constant value.

The vertically movable top box together with the control system ensures that the vertical stress applied to the interface remains constant during the testing process (Stoewahse, 2001). This construction was selected as the standard DSA design and incorporated into the German DIN 18 137-3.

This device was used in the repeatability, reproducibility and geocomposite grid direction tests presented in this report.

3.2.4 Electric Pore Pressure Transducers PDCR81

Three electrical pore pressure transducers similar to the one described and illustrated in Figure 2.11 were selected for use in the experimental set up to measure pore water pressure on the clay/geomembrane interface during shear. These are shown in Figure 3.2.
3.2.5 Brass Bosses

3 No. Brass Bosses were shaped to accommodate pore pressure transducers. The bosses had sintered porous bronze discs glued to the bottom and an O-ring to seal against the geomembrane, as shown in Figure 3.3.

![Figure 3.2 Pore water pressure transducers](image1)

![Figure 3.3 Brass boss fittings](image2)

The dimensions of the bosses are shown in Figure 3.4.

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3.3.6 Machined Flanged Ring

The figure is shown. The dimensions are as follows:

- **Material:** Brass
- **Drill and Tap:** Off side holes
- **Groove to fit 1/2" Ring
- **TAP 1/4 BSP X 12 DEEP**

**Figure 3.4 Dimensions of Brass Bosses**

- **CROSS SECTION A–A**
- **ALL DIMENSIONS IN mm**

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3.2.6 Machined Plastic Block

Four plastic plates were fixed together by use of a water proof adhesive such that the bottom and top where flat for even distribution of applied stress. The block was then machined to accommodate insertion of the three pressure transducers and bosses so that the bottom of the boss was flush with the base of the block. This enabled even normal stress distribution at the base of the block.

The top of the block was machined to allow the transducer cables to terminate at the side of the block. Channels were machined such that the transducer cables could sit lower than the top edge of the block. Photographs of the machined plastic block are shown in Figure 3.5.

Figure 3.5 300mm x 300mm machined plastic block

Figure 3.5 shows the position of the transducers in (a), the insertion of the transducers flush with the block base in (b) and the transducer cable arrangement on the top side of the block in (c).

3.3 Material Specification

Materials used in this test programme were selected as being typical for landfill lining construction. Apart from the HDPE geomembranes, all other materials used in the various testing programmes are suitable for use in capping systems.
3.3.1 Geomembrane/Geotextile Repeatability Tests

The repeatability tests are of geomembrane/non woven geotextile interfaces. The tests were carried out in two series which differ in the materials used, the range of normal stresses applied and the loading systems. The manufacturer’s product specification sheets are presented in Appendix A.

Series One & Two:
Smooth High Density Polyethylene (HDPE) geomembrane vs. a 2.5 mm thick non woven needle punched polypropylene (HP7) geotextile. The HDPE geomembranes used in this series of tests are typical of geomembranes used in basal and side slope landfill lining systems.

Series Three:
Smooth and textured 1mm thick flexible Linear Low Density Polyethylene (LLDPE) geomembrane against a needle punched non-woven 80 g/m² Polypropylene (PP) geotextile. The average asperity height for the textured geomembrane was 0.95 mm ± 0.2 mm. Materials used in this test programme were selected as being typical for cover system construction.

To reduce the number of variables, and hence variability of results, the repeatability test materials were restricted to geomembrane and geotextile (no cover soils were involved). All the samples used were taken from the same roll of material. One side of each of the geosynthetics was selected for testing, and all tests were carried out on the selected sides. The same direction of shearing was maintained for all the samples. Samples were stored for a minimum of 24 hours in a room with the temperature controlled at 20 ± 2°C, before testing.

3.3.2 HDPE Geomembrane/Geotextile Reproducibility Tests

The interfaces were dull side sprayed textured geomembrane (2 mm) and polypropylene 1200 g/m² non woven geotextile.
3.3.3 Geotextile/Sand Reproducibility Tests
A 335 g/m² non woven polyethylene geotextile vs. CEN sand (European standard sand) at 1.8 g/cm³. The sand density value of 1.8 g/cm³ was chosen according to the draft European standard EN 196-1.

3.3.4 Geocomposite/Sand Grid Direction Tests
The geocomposites chosen for this study include Terram 1B1, GSE Fabrinet and GSE Triplanar 300, herein after referred to as T, F & G respectively. They were sheared against Leighton Buzzard sand.

Geocomposite T comprises of a fairly uniform HDPE 3-D net structure laminated to and between two layers of a thermally bonded non woven geotextile (Terram 1000) of 729 g/m² mass per area. The product is altogether about 5 mm thick.

Geocomposite product G Triplanar 300 is an ultra high flow tri-planar drainage 7.6 mm thick geonet consisting of three layers of extruded ribs sandwiched between a 2 mm thick 200 g/m² polypropylene geotextile. The centre vertical ribs which create the flow channel are protected by top and bottom structural ribs that are meant to minimise the potential for intrusion into the flow channel from both the upper and lower substrata.

GSE fabrinet consists of a 5mm Bi-planar HDPE geonet consisting of a parallel set of extruded ribs overlying a similar set oriented in the direction of manufacturing. Non woven polypropylene geotextile fabric 2 mm thick is heat bonded to both sides of the geonet. The overlying ribs are thicker and form the flow channel.

Leighton Buzzard sand is a uniformly graded sand of an average particle size of 1.2 mm. D₁₀ = 1.18 mm, D₂₀ = 1.40 mm and D₆₀ = 1.6 mm. It has a minimum density of 1493 kg/m³, a maximum density of 1667 kg/m³ and a friction angle of 35°. Of the three geocomposites, only the results of the Terram geocomposite/sand interface are presented in this work.
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3.3.5 Geomembrane/Clay PWP Measurement Tests
For this test program, a 2.5 mm HDPE textured geomembrane with an impingement of \( \sim 55 \, \text{g/m}^2 \) (Textured HDPE geomembrane) was chosen. The average asperity height for the textured geomembrane was \( 0.95 \, \text{mm} \pm 0.2 \, \text{mm} \).

The geomembrane was in combination with Mercia Mudstone at a moisture content approximately 3-4% wet of optimum. Mercia Mudstone material has a Specific Gravity of 2.74. Atterberg limit tests on this material yielded Liquid limit 36\%, Plastic Limit 19\% and a Plasticity Index of 17\%. It can be classified as clay of low plasticity. Compaction tests gave an optimum moisture content of \( \sim 13.5\% \) at 1.87 Mg/m\(^3\) maximum dry density.

3.4 Test Procedures
The test procedures described include those for the repeatability, reproducibility, geocomposite grid direction and pore water pressure measurement tests. In the latter, a staged description is given from the preparation of the geomembrane, assembly of the transducers onto the geomembrane, compaction of clay into the shear box and fixation of the transducer assembly onto the shear box, in readiness for the consolidation and shearing stages.

3.4.1 HDPE Geomembrane/Geotextile Repeatability Tests
In this the first of the repeatability test series, over 50 tests were carried out at normal stresses of 10, 20 and 30 kPa. Nylon spacer blocks were placed in the lower and upper boxes. A pneumatic bag was used for loading at normal stresses of 10, 20 & 30 kPa.

The main test features are listed below:

- Virgin samples were used for each test
- Samples were conditioned in a temperature controlled laboratory at \( 20 \pm 2^\circ\text{C} \) for at least 24 hours before the start of the test. Shear testing was carried out at the above temperature.
• Spacer blocks with sand paper or textured nylon surface were in contact with the non shearing side of the geosynthetics, in order to minimise stretching of the geosynthetic material during shearing.

• Both geosynthetic samples were punched and clamped onto the leading edges of the top and lower boxes. None of the samples were glued onto the substrates.

• It was assumed that the normal stress reading on the dial gauge was equivalent to that on the interface.

• Each specimen was sheared at a constant rate of displacement (3 mm/min). Start of shearing was at least 10 minutes after application of normal stress.

• Direction of shear was the machine direction (i.e. direction in which the geosynthetic roll would be rolled down slope on site).

• A constant effective sample area of 300 mm x 300 mm was used

• All samples were sheared until a constant residual load was reached or a maximum displacement of at least 90 mm.

• One operator carried out all the tests (the Author).

The detailed test procedure for Series One repeatability tests is presented in Appendix B. The test results are presented in Sections 4.1.1, 4.1.2 and 4.1.3.

### 3.4.2 LLDPE Geomembrane/Geotextile Repeatability Tests

This is the main series of the repeatability tests. It was conducted to quantify the variation in data resulting from a carefully controlled test procedure, with the number of variables minimised. It was thought that repeatability could be improved by using one design of direct shear device and one operator.

**Experimental Set up**

Nylon spacer blocks were placed in the lower box such that the top surface of the upper spacer was flush with the top of the box. The upper surface of the top...
block was covered with a high friction coating to ensure that the overlying geomembrane did not stretch. A geomembrane sample with a shear area of 300 mm x 400 mm was clamped to the leading edge of the lower box using bolts acting through a spreader bar. The geotextile sample, with a shear area of 300 mm x 300 mm, was clamped to the leading edge of the top box using a similar system. In all tests the geotextile was attached to the top box and the geomembrane to the bottom as this configuration produces results most representative of field conditions (Jones & Dixon, 2000).

The top box was brought into contact with the lower box, and then raised by 1mm to ensure that a shear force was not generated between the top and lower boxes. Due to the top box being fixed, this gap was maintained throughout the test. Nylon spacer blocks were placed in the top box to transfer the normal stress to the interface. The side of lowest nylon block in contact with the geotextile was covered with a high friction coating to ensure that the underlying geotextile did not stretch.

Tests were conducted at normal stresses of 10, 20 & 30 kPa, and each test was conducted using virgin samples of geosynthetics. Normal stress was applied and held for 10 to 15 minutes before shearing the interface at a rate of 3mm per minute. The temperature during testing was maintained at 20 ± 2°C. A minimum shear displacement of 90 mm was achieved in all tests. A minimum of 11 tests was carried out at each normal stress for both the smooth and textured geomembranes. One operator (the Author) carried out all the tests, thus eliminating variations in the test procedure caused by different operating techniques. A summary of the testing program is given in Table 3.2.
Table 3.2  LLDPE Geomembrane/Geotextile Repeatability Testing Program

<table>
<thead>
<tr>
<th>Interface</th>
<th>Condition of Interface</th>
<th>Normal Stress (kPa)</th>
<th>No. of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>S&lt;sub&gt;mgm&lt;/sub&gt;/gt</td>
<td>Normal</td>
<td>10</td>
<td>13</td>
</tr>
<tr>
<td>S&lt;sub&gt;mgm&lt;/sub&gt;/gt</td>
<td>Normal</td>
<td>20</td>
<td>11</td>
</tr>
<tr>
<td>S&lt;sub&gt;mgm&lt;/sub&gt;/gt</td>
<td>Normal</td>
<td>30</td>
<td>11</td>
</tr>
<tr>
<td>S&lt;sub&gt;mgm&lt;/sub&gt;/gt</td>
<td>Scratched</td>
<td>10,20,30</td>
<td>6 (2 No. at each normal stress)</td>
</tr>
<tr>
<td>S&lt;sub&gt;mgm&lt;/sub&gt;/gt</td>
<td>Damp wipe</td>
<td>10,20,30</td>
<td>6 (2No. at each normal stress)</td>
</tr>
<tr>
<td>T&lt;sub&gt;xgm&lt;/sub&gt;/gt</td>
<td>Normal</td>
<td>10</td>
<td>13</td>
</tr>
<tr>
<td>T&lt;sub&gt;xgm&lt;/sub&gt;/gt</td>
<td>Normal</td>
<td>20</td>
<td>12</td>
</tr>
<tr>
<td>T&lt;sub&gt;xgm&lt;/sub&gt;/gt</td>
<td>Normal</td>
<td>30</td>
<td>11</td>
</tr>
<tr>
<td>T&lt;sub&gt;xgm&lt;/sub&gt;/gt</td>
<td>Preloaded</td>
<td>10,20,30</td>
<td>6 (2No. at each normal stress)</td>
</tr>
<tr>
<td>T&lt;sub&gt;xgm&lt;/sub&gt;/gt</td>
<td>Rubbed</td>
<td>10,20,30</td>
<td>6 (2No. at each normal stress)</td>
</tr>
</tbody>
</table>

For each of the types of geomembrane, two other conditions were investigated separately. For the smooth material the effects of scoring the surface and of wetting the surface were assessed. To investigate the effect of possible variations in the surface friction of the geomembrane caused by damage during manufacture and sample preparation, a pattern of shallow scratches was made across the geomembrane surface. The same regular pattern was used in each of the tests. The influence of moisture on shear strength was assessed by wiping the surface of the geomembrane with a wet cloth prior to covering it with the geotextile. For the textured geomembrane tests, conditions of pre-loading and geotextile damage were investigated.

The effect of pre-loading the interface before shearing was assessed as it was anticipated that activities such as dropping the nylon spacer blocks into the top box, or accidentally increasing the normal stress above the test stress during test set-up, might have increased the entanglement between the geotextile fibres and the geomembrane asperities. The normal stress was increased by 20 kPa above the test normal stress value and held for 10 minutes before reducing it to the test value and shearing the interface. To investigate possible damage to the...
geotextile during sample preparation (i.e. fibres being pulled out or broken), the geotextile was dragged across the surface of a separate piece of textured membrane, in the direction of shearing, under zero normal stress, prior to clamping it in the shear box.

Six tests were conducted for each of the special conditions investigated (i.e. two tests were carried out at each normal stress). The detailed test procedure is presented in Appendix B. The test results are presented in Sections 4.1.4 and 4.1.5.

3.4.3 HDPE Geomembrane/Geotextile Reproducibility Tests
The geomembrane was fixed to a rigid substrate and clamped on its tension side, to the lower box. The geotextile was clamped to the top box. The top box was then filled with a standard sand according to EN196-1. The sand was compacted to a density of 1.8 g/cm³. This is equivalent to 5 cm in the upper box. The general procedure used for the fixed and vertically movable devices was the same as that used in Section 3.4.2.

The materials were tested dry. The geomembrane and geotextile materials were tested in the direction of production at normal stresses of 10, 25, 50, 100 and 200 kPa. The three large direct shear devices used differed in the design of the movement of the top box. The devices were the fixed top box, the tilting top box and the vertically moveable top box. The rate of shear was 1 mm/min. The gap size was maintained at 1 mm. The test results are presented in Sections 4.2.2.

3.4.4 Geotextile/Sand Reproducibility Tests
Huesker B1200 geotextile was sheared against CEN sand in the in fixed, floating (Wykeham Farrance) and the vertically movable devices.

The density of Eurosand used for these tests was 1.8 g/cm³. This is equivalent to 5 cm of sand in the upper box. The gap size of 1 mm was set after the sand was placed in the upper box. A solid flat steel loading plate was placed
immediately on top of the sand followed by plastic spacers which rose above the top box by about 5 cm for the pneumatic bag normal stresses (50-200 kPa). The interface was tested dry at a rate of shear of 3 mm/min. The rest of the test procedure is generally similar to that in Section 3.4.2. The test results are presented in Section 4.2.1.

### 3.4.5 Geocomposite/Sand Grid Direction Tests

The geocomposites (see Section 3.3.4) were cut in three different orientations of the grid and in the direction of the roll.

![Geocomposite grid orientation](image)

Geocomposite placement on landfill covers is typically down slope in the direction of the roll. In the field, after unrolling the material, sand would generally be placed on top of the geocomposite. This was a major consideration in choosing the side of the geocomposite to be sheared against sand.

The lower box of the shear box was filled with plastic spacer blocks. The uppermost spacer block was wooden, 1 mm proud of the height of the box with its top side covered with sand paper. The purpose of the sand paper was to grip the geocomposite from the underside. The geocomposite was then clamped to the leading edge of the lower box. The top box was lined with frictionless tape and lowered onto the geocomposite, clamped and the sand compacted to its maximum density, of ~ 1667 kg/m³. The loading system was attached; the top box unclamped and raised by approximately 1mm above the interface. The
normal load was then applied and a seating time of 10 - 15 minutes allowed before shearing at 1 mm/min.

For each of the three grid directions, shearing was undertaken at normal stresses of 10, 20 & 30 kPa. These stresses are within the range for typical capping systems. It was also attempted to ascertain the normal load on the interface during shear by the use of load cells located in the lower box for one of the tests. The testing program is shown in Table 3.3.

Table 3.3 Geocomposite/Sand Testing Programme

<table>
<thead>
<tr>
<th>Interface Tested</th>
<th>Design of Top box of DSA</th>
<th>Normal Stress</th>
<th>Loading Variations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terram 1B1/LB Sand</td>
<td>Fixed</td>
<td>10, 20, 30</td>
<td>Low load system</td>
</tr>
<tr>
<td>Terram 1B1/LB Sand</td>
<td>Vertically Movable</td>
<td>10, 20, 30</td>
<td>Pneumatic Bag</td>
</tr>
<tr>
<td>GSE Triplanar/LB Sand</td>
<td>Fixed</td>
<td>10, 20, 30</td>
<td>Low load system</td>
</tr>
<tr>
<td>GSE Fabrinet/LB Sand</td>
<td>Vertically Movable</td>
<td>10, 20, 30, 50</td>
<td>Pneumatic Bag</td>
</tr>
<tr>
<td>LB Sand/LB Sand</td>
<td>Floating</td>
<td>10, 20, 30, 50, 100, 200</td>
<td>Loading Yoke</td>
</tr>
</tbody>
</table>

* = Load cells in lower box

The results are presented in Section 4.3.

3.4.6 Pore water Pressure Measurement on the Interface

This section describes in stages, the procedure used for the pore water pressure measurement tests. The tests were conducted on a clay/textured geomembrane interface.

The procedures described below include compaction tests on the clay, oedometer consolidation tests on the clay, consolidated drained tests in the small shear box, preparation of the textured geomembrane, preparation and assembling of the transducers onto the geomembrane, compaction of clay in the large direct shear device and fixation of transducer assembly onto the surface of the compacted clay, in readiness for the consolidation stage.

Investigating the stability of geosynthetic landfill capping systems
Compaction Tests

Compaction tests were required to determine the optimum moisture content and maximum dry density of the Mercia Mudstone sample.

The laboratory compaction tests were conducted on the clay as described in BS1377: 1990, Part 4. This test covers the determination of the dry density of soil passing a 20mm test sieve when it is compacted in the specified manner described below over a range of moisture contents. A brief description of the test procedure is presented below:

- The mould with the base plate attached was weighed to 1g. The internal dimensions of the mould were weighed to within 0.1mm.
- The mould extension was attached and the assembly placed on a solid base.
- Water was added to the prepared soil and the soil and water mixed thoroughly.
- A quantity of moist soil was placed in the 1L (one litre) mould. The sample was moulded such that it occupied a little over 1/3 of the height of the mould when compacted.
- 27 No. blows were applied onto the sample from the free falling 2.5kg rammer, being dropped from ~ 300mm height above the soil. The mould was rotated around the base during the application of the blows to distribute the blows over the whole surface.
- Another quantity of moist soil was placed in the mould such that it occupied just over 2/3 of the height of the mould when compacted.
- 27 No. blows were applied as previously using the 2.5 kg rammer.
- A third quantity of moist soil was placed in the mould such that the amount of soil used filled the mould body, with the surface not more than about 6mm proud of the upper edge of the mould body.
The extension was removed and the excess soil cleaned off. The surface of the compacted soil was levelled off using a straight edge. Any loose materials that may have been removed during levelling were pressed back in.

- The soil and mould with base plate were weighed.
- The compacted soil was removed from the mould and placed on a tray. A small sample was taken from it for determination of moisture content according to BS1377:1990 Part 2.
- The remainder of the soil was broken up, and mixed with another test sample.
- An adequate amount of water was added to the soil and the soil and water thoroughly mixed. This sample was made slightly wetter than the previous.
- Compaction was conducted as described for the first sample and the moisture content also determined.
- Seven tests were conducted as described above.
- The values of moisture content corresponding to each volume of compacted soil were compiled. A plot of dry density vs. moisture content was used to determine the optimum moisture content and maximum dry density of the clay sample.

The results are presented in Figure 4.21.

Clay Consolidation in Oedometer
This test was carried out to determine the coefficient of consolidation (c_v) and the theoretical rate of consolidation. The procedure is detailed in BS1377:1990 but is briefly described below.

- A cylindrical sample of the clay (75mm diameter and 20mm thick) was enclosed in a metal ring with porous plates above and below it.
The specimen was saturated with water and subjected to a series of increasing vertical static loads at regular time intervals.

- The load was doubled with each increment i.e. 50kPa, 100 kPa & 200 kPa.
- Changes in vertical displacement of the sample were recorded against time during and after the application of each load.
- After full consolidation under the final load, the vertical static loads were removed in decremental stages, over a period of two to three days, to allow the sample to complete swelling after each load decrement.

The clay sample was removed and its thickness and water content determined.

- The change in thickness of the sample was plotted against time during the load stage to determine the rate of consolidation and the coefficient of consolidation ($c_v$).

The results are presented in Figure 4.22.

**Consolidation & Drained Shear Tests in 100mm x 100mm DSA**

This test was used to determine the consolidated drained shear strength parameters $c'$ and $\phi'$ of the clay, to enable interpretation of the large DSA tests with pore water pressure measurement. The test procedure is fully described in BS1377.

- The Mercia Mudstone sample at moisture content of ~17% and dry density $1.81\text{Mg/m}^3$ was placed in the small direct shear device, sandwiched between two porous plates and a normal stress applied.
- Water was added to the outer container immediately before loading the sample.
- The frames that hold the sample were unlocked and a shear commenced, essentially displacing the top box with respect to the bottom box, at a constant rate of shearing.
Data logging of the vertical movement of the dial gauge commenced as soon as the normal load was applied to the sample.

The strain rate used was calculated from the oedometer consolidation tests.

The shear force and horizontal displacements were recorded as the sample was sheared.

Three clay samples were each consolidated under a different normal load to determine the shear strength.

The results are presented in Section 4.4.3

**Textured Geomembrane Preparation**

- The textured geomembrane sample was shaped and punched for clamping onto the upper box of the fixed DSA.

- A wooden template was used to locate the 3 No. positions for the fixation of the transducer fittings. The fittings were approximately 100 mm from each other, in a triangular shape. All three transducers were approximately 100 mm from the edge of the 300 mm x 300 mm area of the geomembrane.

- The geomembrane at the location of each transducer was smooth to ensure intimate contact with the transducer boss and hence formation of a competent seal.

- Approximately 20 pin sized holes were drilled through the geomembrane at the centre of each proposed boss location, in the position of the porous disc, to allow pore pressure transmission from the clay at the interface to the transducer.

- Six holes per fitting were drilled in the geomembrane and counter sunk for the attachment of the screws onto the brass bosses.

**Transducer Assembly onto Geomembrane**

- Transducers were calibrated using the Budenburg gauge.
The brass bosses with sintered bronze discs (bonded onto the lower side of the bosses) were cleaned in an ultrasonic bath, rubber o-rings inserted into the grooves and the whole set up de-aired in the vacuum chamber.

De-aired water was injected into the pore pressure transducers to displace any air.

The transducers were fixed onto the bosses under de-aired water in a big basin full of de-aired water. Care was taken to keep the electrical connections to the transducers free of water.

All three transducers with fixings were transferred into a vacuum chamber for further de-airing pending their fixation onto the geomembrane.

The transducers with brass bosses were then screwed onto the geomembrane under de-aired water. Care was taken to keep the top end of the transducer with electric wires dry. The o-rings previously attached to the bosses were compressed on fixation of the bosses onto the geomembrane. They served to form a seal so that no flow of water was allowed on the upper surface of the geomembrane into the transducer assembly.

The heads of the screws attaching the brass boss to the geomembrane were carefully countersunk.

The geomembrane with transducer assembly was quickly placed onto the previously prepared clay sample which was in the lower box of the DSA. The transducer assembly is shown in Figure 3.7.
Clay Compaction and Set up of Experiment in the Large DSA

- A 300 mm x 300 mm piece of Terram 1B1 geocomposite material was inserted in the lower box for use as a drainage layer.

- To keep the shear area at 300 mm x 300 mm, a 100 mm x 300 mm steel block was placed inside the lower box at the leading edge.

- Mercia Mudstone at a moisture content of ~17% was carefully placed and lightly compacted into the lower box using a tamping rod to achieve an approximate dry density of 1.81 Mg/m³. The soil was compacted flush of the lower box. The moisture content of the sample after compaction was recorded. The top of the sample was covered with a wet paper towel to prevent the shear surface from drying out, in readiness for the geomembrane/brass boss set up.
• The geomembrane with transducer fittings was then carefully clamped onto the top box.

• The 300 mm x 300 mm plastic block shaped to accommodate the brass bosses and transducers with their electric wires was carefully lowered into position on the geomembrane.

• The transducer wires were carefully fed through the channels in the plastic block, through a round hole in the top box previously drilled to accommodate them and then connected onto a data logger.

• The loading plate with pneumatic bag was then fitted onto the top box. The design of the experiment for measurement pore water pressure did not allow the use of the low load platen.

• The sample was submerged in water up to about 10 mm above the geomembrane/transducer interface.

• Data logging of the pore water pressure was commenced. The response of the transducers to the applied load would be used to measure the value of the pore pressure coefficient, B.

• A given normal stress was applied to the top of the sample via the pneumatic bag for 24 hours, to consolidate the sample and to allow time for pore water pressure dissipation before drained shearing was commenced.

• The sample was sheared at a very slow rate of 0.001 mm/min for approximately 10 mm (this was as close as possible to 0.00096 mm, the rate used in the 100 mm x 100 mm shear box) to monitor changes in pore water pressure and how closely the shear strength matched that obtained from the small shear box. (Faster rates of shear would be used after it was established that the above method, with measurement of pore water pressures, was working well.)

• A record of the moisture content of the sample after shear was taken.
The results of the pore water pressure trial tests are presented in Section 4.4.

3.5 Summary

This chapter has presented details of the various interface test programs with descriptions of the equipment used and the characteristics of the materials involved.

The test programs included geomembrane/geotextile repeatability tests, geotextile/sand and geomembrane/geotextile reproducibility tests, geocomposite/sand grid direction tests and pore water pressure measurement tests in the large direct shear. The different designs of large direct shear devices used include the fixed top, floating top and the vertically movable top. The materials used are typical landfill interfaces.

Repeatability tests were carried out to understand the causes of scattering of interface shear strength data and to ascertain this variability at low normal stresses suitable for capping systems. Reproducibility tests were undertaken to investigate and compare the behaviour of the materials using different devices, different laboratories and different operators. It was necessary to ascertain and validate the reliability of the resultant data. The structural/shear behaviour of geocomposites was investigated to study the effect of the direction of the grid on the interface shear strength, especially at low normal stresses. The pore water pressure measurement trial tests were undertaken to confirm whether the clay/geomembrane interface is a drainage path and also to obtain effective shear strength parameters.

The results of all the tests are presented in Chapter 4 and discussed in Chapter 5.
4. Laboratory Test Results

All the results of the laboratory testing are described and presented in this chapter. They include interface shear strength test results for the geomembrane/geotextile repeatability tests, geomembrane/geotextile and geotextile/sand reproducibility tests, geocomposite/sand grid direction tests and the pore water pressure measurement tests. For each interface shear test, the mobilised shear stresses are plotted against displacement. These show how quickly peak and residual shear strengths are reached and also whether the interface is strain softening. The shear strengths are plotted against the normal stress and a best-fit straight line drawn through the points. The interface shear parameters (friction angle and cohesion) are given by the gradient of the best fit line.

The pore water pressure measurement test results include classification tests for the clay sample, oedometer & shearbox consolidation and pore pressure transducer readings on the clay/geomembrane interface during staged consolidation and shear.

4.1 Repeatability Tests

The repeatability test results are divided into two series. The first series results are for a smooth HDPE geomembrane/HP7 non-woven geotextile while the second series results are for both smooth and textured LLDPE geomembrane vs. HP3 Non woven geotextile.

4.1.1 Series S1-A: Smooth HDPE GM/HP7 GT (10 kPa fixed top)

Smooth HDPE geomembrane was sheared against HP7 non woven geotextile at a normal stress of 10 kPa and a rate of shear of 3 mm/min, in a fixed top large DSA. The normal stress was applied using the air bag. Thirty five tests were carried out. The interface was tested dry. Thirty five tests were carried out. The results are presented in Figure 4.1
Repeatability Testing  
June 1999

Interface: Smooth Geomembrane/Non Woven Geotextile  
Normal Stress = 10 kPa  
Rate of Shear = 3 mm/min  
Operator: pk

There are differences in the shapes of the shear stress vs. displacement plots Figure 4.1. The shear stress-displacement plot shows strain softening behaviour typical of many geosynthetic interfaces. Peak shear stress is achieved within the first 5-10mm of shear. The shear stress then reduces markedly in the tests with peak shear stresses above 6 kPa.

The shear stress vs. normal stress plots for all the tests show that peak shear stress values range from 2 kPa to 13 kPa for the same interface under the same testing conditions. A histogram of the data is presented in Figure 4.2.
Chapter 4: Laboratory Test Results

Figure 4.2 Histogram of S1-A GM/GT Results

The histogram shows that over 80% of the tests achieved peak stress between 2 kPa and 6 kPa.

4.1.2 Series S1-B: Smooth HDPE GM/HP7 GT (10 kPa fixed top)

As in Series S1-A, smooth HDPE geomembrane was sheared against HP7 non-woven geotextile at normal stress of 10 kPa, but with the normal stress applied using weights. The shear stress-displacement and shear stress-normal stress plots presented in Figure 4.3 are for tests at 10 kPa only.

The shapes of the shear stress vs. displacement plots are very similar. Peak shear stresses were mobilised generally within the first 10 mm of displacement. Shear stress gradually reduces with increasing displacement until there is little further change in shear stress with subsequent displacement.
Repeatability Testing
September 1999

Interface: Smooth Geomembrane/Non Woven Geotextile
Normal Stress = 10 kPa
Rate of Shear = 3 mm/min
Operator: pk

All twenty tests generally show a strain softening behaviour and yield peak shear stresses between 4 kPa & 6 kPa. The average large displacement shear stress was 3.7±0.5 kPa.

4.1.3 Series S1-C: Smooth HDPE GM/HP7 GT (20, 30 kPa fixed top)
In this series, the same interface as S1-B was tested at 20 and 30 kPa using the air bag as the loading system. Four tests were carried out at 20 kPa and Thirteen tests at 30 kPa. The shear stress-normal stress plot of results presented in Figure 4.4 includes data from S1-B at 10 kPa.
Figure 4.4 Shear Stress/Normal Stress Plots for S1-B and S1-C

The average peak shear strengths obtained at 20 kPa and 30 kPa are 8.18 kPa and 7.21 kPa respectively. The ringed results at 30 kPa are noticeably lower than those obtained at 20 kPa, indicating a possible problem with the test procedure.

4.1.4 Series S2-A: LLDPE GM/HP3 GT (fixed top)

From the test results obtained in Series S1, it was necessary to increase accuracy of normal stress loading especially at 20 and 30 kPa, and to specify a test procedure. Apart from the use of different materials for this series of repeatability tests, the loading system was changed from the pressure bag to a low load system with a load platen, suitable for loading at the lower normal stress range i.e. (10 – 50 kPa).

The materials used are smooth and textured linear low density polyethylene (LLDPE) geomembranes and HP3 Non woven geotextiles. Shear stress vs. displacement plots
for the smooth geomembrane vs. non woven geotextile in the fixed top box are presented in Figure 4.5.

Figure 4.5  Shear stress vs. displacement plots: Smooth LLDPE GM/HP3 GT

Separate plots are presented for the tests at the three normal stresses of 10, 20 & 30 kPa. The shapes of the plots are generally similar, with post peak shear stresses remaining generally constant until the end of the test. Shear stresses are shown to increase with increasing normal stress as expected.

Figure 4.6 shows the shear stress/displacement plots for the textured geomembrane/non woven geotextile interface at normal stresses of 10, 20 and 30 kPa.
The strain softening behaviour is more evident in the textured geomembrane/geotextile interface as shown in Figure 4.6. The shapes of the plots are generally similar. Peak shear stress is achieved within the first 5mm of displacement. Shear stress continues to decrease with increasing displacement until the test is stopped.

Special condition tests were undertaken on both the smooth and textured geomembranes to assess the possible causes of scattering of shear stress results. For each geomembrane type, two conditions were investigated separately. On the smooth geomembrane, the effects of scoring and of wetting the surface were assessed. For the textured geomembrane, conditions of pre-loading and geotextile damage were investigated.

The shear stress/normal stress plots for the smooth geomembrane/geotextile interface are shown in Figures 4.7 and 4.8. The tests in which the surface of the geomembrane was wetted show a decrease in the angle of friction whereas the scratch tests and the normal dry condition tests give the same friction angle.
Chapter 4: Laboratory Test Results

Figure 4.7 Smooth GM/GT: Peak shear strength

Figure 4.8 Special Conditions Smooth GM/GT: Peak Shear Strengths
Figures 4.9 and 4.10 show the shear stress vs. normal stress plots for the textured geomembrane interface in the normal and special conditions, respectively.

![Figure 4.9 Textured Geomembrane/Geotextile: Peak Shear Strength](image-url)

- Friction Angle = 34 deg
- Y-Intercept = 4 kPa
Figure 4.10 Special Conditions Textured GM/GT: Peak shear strength

The textured geomembrane/geotextile Coulomb plots show a decreased angle of peak friction for the preloaded interface. The large displacement shear strengths for the preloaded interface also increased at 10 and 20 kPa.

4.1.5 Series S2-B: LLDPE GM/HP3 GT (vertically movable top)
Both the smooth geomembrane/non woven geotextile and textured geomembrane/non woven geotextile interfaces were tested in a vertically movable top large direct shear device. The shear stress/displacement and shear stress/normal stress plots are presented in Figure 4.11.
Figure 4.11 Smooth & Textured GM/GT results in Vertically Movable top box DSA.

The shear stress vs. displacement plots generally show good repeatability for both the smooth and textured interfaces in the vertically movable top box design.
4.1.6 Summary of Repeatability Test Results

Table 4.1 provides a summary of the repeatability test results including interface type, shear box design, stress range and shear strength parameters.

Table 4.1 Summary of Repeatability Test Results

<table>
<thead>
<tr>
<th>Interface</th>
<th>Shear Box Design</th>
<th>Condition</th>
<th>Normal Stress (kPa)</th>
<th>Peak Shear Stress (kPa) or Peak Friction Parameters (Degrees, kPa)</th>
<th>Residual Shear Stress (kPa) or Residual Friction Parameters (Degrees, kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HDPE smgm/gt</td>
<td>Fixed</td>
<td>Dry</td>
<td>10</td>
<td>4.06</td>
<td>3.23</td>
</tr>
<tr>
<td>HDPE smgm/gt</td>
<td>Fixed</td>
<td>Dry</td>
<td>10</td>
<td>4.6</td>
<td>3.7</td>
</tr>
<tr>
<td>HDPE smgm/gt</td>
<td>Fixed</td>
<td>Dry</td>
<td>20</td>
<td>8.18</td>
<td></td>
</tr>
<tr>
<td>HDPE smgm/gt</td>
<td>Fixed</td>
<td>Dry</td>
<td>30</td>
<td>7.21</td>
<td></td>
</tr>
<tr>
<td>LLDPE Smgm/gt</td>
<td>Fixed</td>
<td>All</td>
<td>10, 20, 30</td>
<td>16, 1</td>
<td>15, 1</td>
</tr>
<tr>
<td>LLDPE Smgm/gt</td>
<td>Fixed</td>
<td>Damp wipe</td>
<td>10, 20, 30</td>
<td>9, 2</td>
<td>8, 2</td>
</tr>
<tr>
<td>LLDPE Smgm/gt</td>
<td>Fixed</td>
<td>Scratched</td>
<td>10, 20, 30</td>
<td>16, 2</td>
<td>14, 3</td>
</tr>
<tr>
<td>LLDPE Smgm/gt</td>
<td>Vertically Movable</td>
<td>Normal</td>
<td>10, 20, 30</td>
<td>10, 1</td>
<td></td>
</tr>
<tr>
<td>LLDPE Txgm/gt</td>
<td>Fixed</td>
<td>All</td>
<td>10, 20, 30</td>
<td>34, 4</td>
<td>27, 3</td>
</tr>
<tr>
<td>LLDPE Txgm/gt</td>
<td>Fixed</td>
<td>Rub Together</td>
<td>10, 20, 30</td>
<td>28, 9</td>
<td>25, 4</td>
</tr>
<tr>
<td>LLDPE Txgm/gt</td>
<td>Fixed</td>
<td>Preload</td>
<td>10, 20, 30</td>
<td>33, 5</td>
<td>22, 6</td>
</tr>
<tr>
<td>LLDPE Txgm/gt</td>
<td>Vertically Movable</td>
<td>Normal</td>
<td>10, 20, 30</td>
<td>25, 4</td>
<td></td>
</tr>
</tbody>
</table>
4.2 Reproducibility Tests

The results of the intercomparison geosynthetic interface testing program between the Hanover and Loughborough universities is presented in this section. The interfaces tested are sand/non woven geotextile interface and geomembrane/non woven geotextile interface with sand in the top box.

Between the two institutions, the interfaces were tested in three different designs of top box of the DSA. The designs include the fixed top, floating top and vertically movable top box. The results are presented in Sections 4.2.1 and 4.2.2 below.

4.2.1 Naue Depotex Geotextile/Eurosand Interface

Shear stress/displacement and shear stress/normal stress plots for this interface in the fixed top DSA are presented in Figure 4.12. The results also include plots of shear stress/normal stress vs. displacement and a measurement of the vertical displacement of the sand in the top box throughout the duration of shear.

The plots are described as follows:

a) Shear stress vs. displacement plots,

b) Shear stress/normal stress vs. displacement,

c) Change in vertical height of sand vs. displacement and

d) Shear stress vs. normal stress.

At the commencement of shear, the vertical height of the sand is shown to generally decrease and then increase with increasing displacement throughout the duration of peak shear and then it decreases with subsequent horizontal displacement.
ARC Interface Shear Testing - Loughborough Univ.

Eurosand - Naue Depotex 335R Geotextile
Durham Geo 300 x 300 mm²
Fixed Top

\[ \sigma = 10/25/50/100/200 \text{ kN/m}^2 \]

---

**Figure 4.12 Geotextile/sand Intercomparison Results in Fixed Top DSA**

*Investigating the stability of geosynthetic landfill capping systems*
The summary of results from all the devices tested is presented in Table 4.2. The shear strength parameters are taken from a Coulomb best fit failure envelope for the range of normal stresses tested i.e. 10 – 200 kPa.

Table 4.2 Results summary of geotextile/sand intercomparison tests

<table>
<thead>
<tr>
<th>DSA top box design</th>
<th>Shear Strength Parameters (deg, kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floating Top (Wykeham Farrance)</td>
<td>37, 0</td>
</tr>
<tr>
<td>Fixed Top (Lboro Univ)</td>
<td>39, 22</td>
</tr>
<tr>
<td>Fixed Top (Hanover Univ)</td>
<td>35, 13</td>
</tr>
<tr>
<td>Fixed Top Normal Load Controlled (Hanover)</td>
<td>36,11</td>
</tr>
<tr>
<td>Vertically Movable Top (Hanover)</td>
<td>37, 0</td>
</tr>
</tbody>
</table>

The results show similar shear strength parameters obtained for the vertically movable top and floating top direct shear devices. The fixed top boxes generally show high y-intercept values.

4.2.2 Textured Geomembrane/Geotextile Interface

This interface was tested with sand in the top box instead of spacer blocks. The plots of results from both the fixed and vertically moveable devices are presented in Figures 4.13 and 4.14 respectively. The behaviour of the sand is also presented in the vertical displacement vs. horizontal displacement plots on the same figures.
ARC Interface Shear Testing - Loughborough Univ.

Textured GSE Geomembrane - Huesker 1200g/m² Geotextile
Durham Geo 30 x 30 cm²
Fixed Top

$\sigma = 10/25/50/100/200$ kN/m²

Figure 4.13 Textured GM/GT/Sand Results in fixed top DSA

Investigating the stability of geosynthetic landfill capping systems
The summary of results obtained from the different devices is presented in Table 4.3.
### Table 4.3 Results Summary for Textured GM/GT Interface Reproducibility Tests

<table>
<thead>
<tr>
<th>Device (top box design variations)</th>
<th>Shear Strength Parameters (deg, kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floating (Wykeham Farrance)</td>
<td>9,20</td>
</tr>
<tr>
<td>Fixed (Lboro Univ)</td>
<td>23,12</td>
</tr>
<tr>
<td>Fixed (Hanover Univ)</td>
<td>22,28</td>
</tr>
<tr>
<td>Fixed (Normal Load Controlled - Hanover)</td>
<td>30,17</td>
</tr>
<tr>
<td>Vertically Movable (Hanover)</td>
<td>28,12</td>
</tr>
</tbody>
</table>

Results for the textured geomembrane/geotextile interface obtained using the Wykeham Farrance device are considerably lower than the those obtained using other devices, suggesting the possibility of procedural failure in this instance.

#### 4.2.3 Summary of All Reproducibility Tests Results

A summary of all the results obtained from the UK-German intercomparison tests for the geomembrane/geotextile and geotextile/sand interfaces is presented in Table 4.4 below.

### Table 4.4 Summary of results of intercomparison tests

<table>
<thead>
<tr>
<th>Shear Devices</th>
<th>Huesker Textured Geomembrane/GSE Non Woven Geotextile 1200 g/m²</th>
<th>Standard sand (Eurosand)/Naue Non Woven Geotextile 300 g/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \delta_p ) (deg) ( \alpha_p ) (kPa)</td>
<td>( \delta_r ) (deg) ( \alpha_r ) (kPa)</td>
</tr>
<tr>
<td>Tilting (Wykeham Farrance)</td>
<td>9, 20</td>
<td>6, 8</td>
</tr>
<tr>
<td>Fixed (Durham Geo)</td>
<td>23, 12</td>
<td>14, 10</td>
</tr>
<tr>
<td>Fixed (Hanover)</td>
<td>22, 28</td>
<td>14, 10</td>
</tr>
<tr>
<td>Fixed + Load Control (Hanover)</td>
<td>32, 15</td>
<td>36, 11</td>
</tr>
<tr>
<td>Vertically Movable (Hanover)</td>
<td>30,4, 8</td>
<td>20, 5,1</td>
</tr>
</tbody>
</table>
4.3 Geocomposite/Sand Grid Direction Tests

The geocomposite/sand interface testing program is presented in Table 3.3. Only the results of the Terram geocomposite are presented below.

4.3.1 Terram GC/Sand in Fixed Top DSA

The shear stress vs. displacement and normal stress vs. displacement results for Geocomposite T in different grid directions are presented in Figures 4.14 and 4.15.

![Shear Stress/Displacement Plots](image1)

![Normal Stress/Displacement Plots](image2)

The normal stress in each of the plots in Figure 4.16 corresponds with the shear stress in each plot in Figure 4.15, for the relevant direction of grid.
The plots show separate shear stress vs. displacement and normal stress vs. displacement plots for geocomposite T in three different grid directions in the fixed DSA. The grid orientations are 60°, 90° and 180° to the direction of shearing. The orientations are described in Section 3.3.4. A plot showing both the normal stress and shear stress during the first 10mm of shearing is shown in Figure 4.17.

Figure 4.17  Shear stress and normal stress/displacement plots for GC T60 (fixed top)

Figure 4.17 shows that the normal stress at the interface increases in the first 2mm of shearing. Although peak shear stress is, in all cases, obtained in the first 10mm of shearing, it does not necessarily coincide with the highest normal stress.

Figure 4.18 shows the stress paths for Geocomposite T60 tests. The results demonstrate the increase in normal stress during the first 10mm of shear displacement in the fixed top design of the shear box. The normal stress continues to increase even after the peak shear stress has been attained. A maximum normal stress of nearly 40 kPa is recorded for the test set at 30 kPa.
4.3.2 Geocomposite/Sand in Vertically Movable Top DSA

As in the fixed top box, Terram geocomposite samples were sheared against Leighton Buzzard sand in the vertically movable top DSA at Hanover University. The shear stress/displacement plots are shown in Figure 4.19, for different orientations of the primary remembers of the drainage core to the direction of shearing.

Investigating the stability of geosynthetic landfill capping systems
Each direction of grid was sheared at 10, 20 and 30 kPa. The repeat tests at the same normal stress are consistent and reproducible. Maximum shear stresses are generally higher with T90 compared to T60 and T180.

4.3.3 Results Summary for Geocomposite T60/sand Interface

A summary of the results for geocomposite T performed using two different designs of devices is presented in Table 4.5.

<table>
<thead>
<tr>
<th>Shear box design (normal stress status)</th>
<th>T60</th>
<th>T60</th>
<th>T60</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Angle of friction (deg), apparent adhesion (kPa)</td>
<td>Angle of friction (deg), apparent adhesion (kPa)</td>
<td>Angle of friction (deg), apparent adhesion (kPa)</td>
</tr>
<tr>
<td>Fixed Top (assumed constant)</td>
<td>39, 8.4</td>
<td>38, 3.5</td>
<td>34, 6.5</td>
</tr>
<tr>
<td>Fixed Top (measured at interface)</td>
<td>42, 2.7</td>
<td>46, -0.6</td>
<td>42, 1.6</td>
</tr>
<tr>
<td>Vertically Movable</td>
<td>33, 2.9</td>
<td>35, 2.8</td>
<td>31, 3.6</td>
</tr>
</tbody>
</table>

The results show generally higher angles of friction for all grid directions in the fixed top box compared to the vertically movable box.

Figure 4.20 shows the Coulomb failure envelopes for geocomposite T in the direction of the roll (T60) for the different scenarios – i.e. fixed top box with normal stress assumed constant, fixed top box with normal stress on the interface measured using load cells and the vertically movable device with normal stress on the interface constant.
4.4 PWP on the Clay/Geomembrane Interface

Trial tests to measure the pore water pressure on the clay/geomembrane interface during shear were conducted.

The tests commenced with compaction tests on the Mercia Mudstone, followed by consolidation of the material in the oedometer and consolidated drained shear tests in the 100 mm x 100 mm shear box. The clay sample was compacted into the lower box of the large DSA, the transducer assembly attached onto the shear box. The material was consolidated over a given range of normal stresses with measurement of pore water pressure and then sheared. The test results are presented below.

4.4.1 Mercia Mudstone Compaction Tests

The compaction tests on the Mercia Mudstone sample was carried out according to the Proctor test detailed in BS1377: Part 4 (1990) using the 2.5 Kg rammer. A plot of the values of dry density and moisture content is presented in Figure 4.21.
The peak value of dry density is 1.87 Mg/m³ and the corresponding optimum moisture content (OMC) is 13.5%.

4.4.2 Staged Consolidation of Mercia Mudstone in Oedometer

Figure 4.22 shows the experimental curves for vertical displacement (dial gauge reading) vs. root time for consolidation of Mercia Mudstone in an oedometer. The sample was initially consolidated at 50 kPa. The normal stress was then increased by 50 kPa to 100 kPa after 24 hours and after another 24 hours the stress was increased by 100 kPa to 200 kPa. These tests were undertaken to calculate the coefficient of consolidation, \( c_v \), and the theoretical rate of drained shearing.
Chapter 4: Laboratory Test Results

Marcia Mudstone Oedometer Consolidation

Figure 4.22 Marcia Mudstone oedometer consolidation at 50, 100 & 200 kPa

The results show changes in vertical displacement of between 1.15 and 1.28 mm for the three normal loads. The large initial compression is due to the compacted sample being partially saturated, as there is compression of air.

The coefficient of consolidation for each of the normal stresses has been calculated using the Root time method as suggested by Atkinson (1993). The sample was assumed permeable above and below because it was sandwiched between two porous plates. The results are presented in Table 4.6 below.

Table 4.6 Coefficient of Consolidation from Oedometer tests

<table>
<thead>
<tr>
<th>Normal stress</th>
<th>Coefficient of Consolidation (m²/yr)</th>
<th>Theoretical time to 90% consolidation (mins)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>3.2</td>
<td>4</td>
</tr>
<tr>
<td>100</td>
<td>0.80</td>
<td>49</td>
</tr>
<tr>
<td>200</td>
<td>0.39</td>
<td>100</td>
</tr>
</tbody>
</table>

Investigating the stability of geosynthetic landfill capping systems
4.4.3 Consolidated Drained Shear Tests in 100mm x 100mm DSA

Consolidated drained shear tests for Mercia Mudstone were carried out in the 100 mm x 100 mm shear box. The first trial consolidated drained test was carried out at 200 kPa at the slowest rate the shear box could go (0.00048 mm/min). The consolidation test is presented in Figure 4.23.

![Figure 4.23 Mercia Mudstone Shearbox Consolidation at 200 kPa.](image)

The shear test (Figure 4.24) showed the Mercia Mudstone sample to have reached its peak shear stress by 0.8 mm of displacement. This was the assumed displacement to failure used to calculate the rate of shear for all the drained tests in the 100mm x 100 mm shear box.

In the tests that followed, the samples were separately consolidated and sheared at the separate normal stresses of 50, 100 & 200 kPa.
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Mercia Mudstone Shear @ 200 kPa  100mm x 100 mm Shear Box
Rate of shear = 0.00048 mm/min

Figure 4.24 Mercia Mudstone Drained Shear at 200 kPa

Vertical displacement was plotted against the square root of time to calculate the
coefficient of consolidation ($c_v$) and the rate of displacement for drained shear using
the method suggested by Bishop & Henkel (1962) as described by Head (1986). The
results are presented in Table 4.7.

Table 4.7 Coefficient of Consolidation from 100 mm x 100 mm DSA

<table>
<thead>
<tr>
<th>Normal stress (kPa)</th>
<th>Coefficient of Consolidation ($m^3$/yr)</th>
<th>$T_{100}$ (minutes)</th>
<th>Time to failure (mins)</th>
<th>Rate of shear to failure (mm/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>2.02</td>
<td>20.25</td>
<td>257.18</td>
<td>0.0031</td>
</tr>
<tr>
<td>100</td>
<td>0.64</td>
<td>64</td>
<td>812</td>
<td>0.00098</td>
</tr>
<tr>
<td>200</td>
<td>0.12</td>
<td>324</td>
<td>4114.80</td>
<td>0.00019</td>
</tr>
</tbody>
</table>

A shearing rate of 0.00096 mm/min was chosen for use in all the direct shear tests in
the small shear device. This value was taken from the consolidation results obtained
from the shear box as they are more conservative and therefore more likely to ensure a drained interface. The tests were undertaken at normal stresses of 50, 100 and 200 kPa. The results are presented in the form of a shear stress/displacement plot in Figure 4.25.

![Mercia Mudstone Drained Shear Test](image)

**Figure 4.25** Drained shear stress/displacement plots for Mercia Mudstone

The results indicate that peak shear stress was achieved between 1.5 mm and 5 mm. The maximum displacement achieved was 10 mm. Effective peak shear stresses of ~45 kPa, 72 kPa and 120 kPa were obtained at normal stresses of 50, 100 & 200 kPa respectively.

### 4.4.4 PWP Dissipation on Clay/GM Interface

Transducer responses of the initial trials of pore water pressure measurement during staged consolidation in the large shear device (300mm x 300mm) at normal stresses from 50, 100, and 200 kPa are presented in Figure 4.26.
The three transducers are represented as channels 1, 2 and 3, for each normal stress. The results indicate an immediate increase in pore water pressure on application of the normal stress. The pore water pressure is shown to decrease over a period of time until the next normal load application.

![Figure 4.26 Transducer responses during staged loading (Trial 1)](image)

The transducers respond differently to the same changes in normal stress. For example, Transducer 2's results indicate that the pore water pressure rose by about 27 kPa, 8 kPa and 7 kPa on application of 50 kPa, increase by 50 kPa to 100 kPa and another increase by 100 kPa to 200 kPa, respectively. Transducer 3's response to loading shows much lower pore water pressure changes compared to Transducers 1 & 2.
Figure 4.27 shows the second experimental trial on the same interface with pore water pressure monitoring at 60 kPa, 100 kPa, 125 kPa, 150 kPa, 200 kPa and during a short shearing phase at 200 kPa normal stress.

The results show an immediate response of the transducers to changes in normal stress. The pore water pressure initially increases and then decreases with time. In this case, the response of the three transducers is similar with pore water pressure changes varying by generally the same amount on loading.

![Porewater Pressure on a Clay/Geomembrane Interface](image)

**Figure 4.27** PWP Measurement during staged consolidation and shear (Trial 2)

Transducer 1 shows an increase of ~180 kPa when 60 kPa normal stress is applied. An increase in normal stress of 40 kPa shows pore water pressures changing by ~120 kPa. The pore water pressure change was expected to increase by about the same normal stress increase. This indicates a possible anomaly. At the commencement of the shearing stage at 200 kPa, there is also an immediate initial increase in pore water pressure but this is shown to decrease with time, as expected.

Investigating the stability of geosynthetic landfill capping systems
4.5 Summary

This chapter has presented the results of the geomembrane/geotextile repeatability
tests, the geomembrane/geotextile and geotextile/sand reproducibility tests, the
gecomposite/sand reproducibility tests and the trial pore water pressure measurement
tests on the clay/geomembrane interface in the large DSA.

Results of consolidation tests in both the oedometer and small DSA, and drained shear
tests in the small DSA, have been presented. These results are discussed in Chapter 5.
Chapter 5: Results analysis and interpretation

5. Results Analysis and Interpretation

In this chapter the methodology used in the analyses and interpretation of the repeatability, reproducibility, geocomposite grid direction and pore water pressure measuring test results presented in Chapter 4 are discussed. Crystal Ball, a Microsoft Excel based analysis program that uses the Monte Carlo simulation has been used to assess the implications for design of the repeatability shear strength data and is also discussed.

5.1 Repeatability Testing

The first series of geomembrane/geotextile repeatability tests was carried out on HDPE geomembrane/HP7 non woven geotextile, materials suitable for side slopes and basal landfill lining systems. The second series was carried out on LLDPE geomembrane/HP3 non woven geotextile in the fixed and vertically movable designs of the large DSA. The results are presented Section 4.1 and discussed below.

5.1.1 Series One

During this series of testing in the fixed shear device using the pressure bag as normal stress loading system, it was noted that at normal stresses between 10 – 30 kPa, suitable for capping systems, the pressure bag was not keeping the pressure constant. The dial gauge showed pressure loss during the test. It was decided to use dead weights for all the tests at 10 kPa and some at 20 kPa. The results are presented in Figure 4.4. Comparison of the shear strength results for 20 kPa using the pressure bag and dead weights showed higher shear strengths with the dead weights.

It can be seen in Figure 4.4 that some of the shear strength results at 30 kPa are equal to or less than those obtained at 20 kPa. This discrepancy was thought to have been due to the fact that the spacer blocks in the top box just beneath the pressure bag were not located above the edge of the top box and as a result, the pressure bag was left sagging. As such, the pressure bag was not occupying the entire area of the box and hence not fully effective. The pressure reading on the gauge did not indicate the true
normal stress being transferred on to the interface. The two results above the ‘ringed’ set of results at 30 kPa in Figure 4.4 were carried out with the spacer blocks a little proud of the top box and they show peak shear strengths nearly 3 kPa higher than previously obtained.

These tests were carried out in a fixed shear box, with no measurement of normal stress at the interface. The normal stress indicated by the dial gauge was not being fully transferred to the interface. It was necessary to find an accurate method of application of low normal stress and of measuring the normal stress on the interface in the fixed top shear device, in order to apply the required normal stress and to interpret the results accurately.

5.1.2 Series Two

Figure 4.5 and Figure 4.6 show consistent shear stress vs. displacement relationships for the two interfaces investigated. Both plots demonstrate a degree of strain softening behaviour that becomes more marked with increasing normal stress, and texturing. Most of the smooth geomembrane/geotextile tests appear to have reached their residual shear strength. Figure 4.6 suggests that the textured geomembrane/geotextile tests have not reached full residual conditions. For this reason, the lowest values of shear strength will be referred to as ‘large displacement shear strength.

Although no two interface shear strength tests are expected to give exactly the same result, it is evident from the above mentioned figures that at a given normal stress, there is significant variability between the tests. A very basic statistical evaluation of the peak shear strength values obtained is presented in Table 5.1. The Coefficient of Variation (calculated as a percentage of the standard deviation/mean) values in the order of 15% and 23% have been calculated for the smooth and textured test series respectively. The results show how wide the range of peak and residual shear strength parameters obtained is. These values are consistent with those reported by Blümel et al. (2000) for low normal stresses. Such data can be used in probabilistic type stability assessments.
Table 5.1  Basic statistical evaluation of repeatability tests

<table>
<thead>
<tr>
<th>Interface</th>
<th>Normal Stress (kPa)</th>
<th>Mean of Peak shear Strengths (kPa)</th>
<th>Standard Deviation of Peak Shear Strengths</th>
<th>Coefficient of Variation of Peak Shear Strengths (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smgm/gt</td>
<td>10</td>
<td>4.2</td>
<td>1</td>
<td>27</td>
</tr>
<tr>
<td>Smgm/gt</td>
<td>20</td>
<td>6.0</td>
<td>1.2</td>
<td>19</td>
</tr>
<tr>
<td>Smgm/gt</td>
<td>30</td>
<td>10.3</td>
<td>3</td>
<td>27</td>
</tr>
<tr>
<td>Txgm/gt</td>
<td>10</td>
<td>10.5</td>
<td>1.6</td>
<td>15</td>
</tr>
<tr>
<td>Txgm/gt</td>
<td>20</td>
<td>19</td>
<td>3</td>
<td>15</td>
</tr>
<tr>
<td>Txgm/gt</td>
<td>30</td>
<td>18</td>
<td>2</td>
<td>11</td>
</tr>
</tbody>
</table>

In consideration of the consistent and careful test procedure and material handling, this scatter is thought to be high. The test results presented are for index tests with no soil involved. It is understood that the scatter would be even higher when soils are involved (Snow et al., 1998).

5.1.3 Special Condition Tests

The special condition results seem to indicate that at low normal stresses, minor factors can have a significant influence on the measured shear strength. Scratching the smooth geomembrane indicates that scratching increases the shear strength. Wetting the geomembrane gives lower values compared with the normal dry and unscratched test conditions.

The film of water present on the surface of the smooth geomembrane was very thin but probably had a lubricating effect on the interface, thus the apparent change in shear strength, which could have a destabilising effect, is significant considering that there was no soil above or beneath the interface, that could have accounted for generation of positive pore pressures on the interface.

Special condition results for the textured geomembrane suggest that rubbing or dragging the textured geomembrane against the geotextile before testing does not affect the friction angle very much. In fact the results compare very well with the 'normal' condition results. Fibre damage during handling (i.e. resulting in their realignment in the direction of shearing, and hence in a reduction in interface shear
strength) is unlikely to be significant. Therefore, this mechanism is unlikely to have contributed to the scatter of the data.

Preloading of the textured geomembrane/geotextile interface seems to significantly affect the mobilised shear strength up to about 30 kPa. The applied preload of 20 kPa before shearing gave higher values at 10 and 20 kPa compared to the 'normal' tests. It seems that at 30 kPa, there is minimal effect of the preload, as the results obtained with the 20 kPa preload are nearly equal to those obtained for the normal test at 30 kPa. At normal stresses higher than 20 kPa, it is thought that the geotextile fibres interlock more with the asperities on the textured geomembrane, thus giving higher shear strength parameters. It is possible that if tests at 30 kPa had been carried out with a higher preload, the results would indicate the same trend as at 10 and 20 kPa, unless there is a threshold normal stress of about 30 kPa required to get full interlocking between fibres and asperities.

While the 'special condition' tests give an indication of possible factors that can cause the scatter of data, they cannot be used to explain all the observed variation in results. It is believed that much of the scatter is due to variation of the geosynthetics, and hence the repeatability tests could be giving a true reflection of the range of shear strengths likely to be mobilised in the field.

5.1.4 Simple Statistical Evaluation
A simple statistical evaluation of the failure envelope of the textured geomembrane/geotextile interface under 'normal' conditions was attempted and is shown in Table 5.2.

The minimum, maximum and median peak shear stress values obtained at each of the three normal stresses (10, 20 & 30 kPa) were computed and the Coulomb failure criterion applied in each case to find the shear strength parameters i.e. slope angle and y-intercept. A best fit straight line was drawn through the average values and three other variations shown in the last three columns of Table 5.2.
Table 5.2 Simple statistical analysis of textured gm/gt failure envelope

<table>
<thead>
<tr>
<th>Normal Stress (kPa)</th>
<th>Lowest</th>
<th>Highest</th>
<th>Median</th>
<th>Average</th>
<th>Max10</th>
<th>Min10</th>
<th>Avg10</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>8.21</td>
<td>11.14</td>
<td>10</td>
<td>9.83</td>
<td>11.4</td>
<td>8.21</td>
<td>9.8</td>
</tr>
<tr>
<td>20</td>
<td>16.1</td>
<td>25.6</td>
<td>17.6</td>
<td>19</td>
<td>17.6</td>
<td>17.6</td>
<td>16.1</td>
</tr>
<tr>
<td>30</td>
<td>21.9</td>
<td>26.8</td>
<td>23.1</td>
<td>23.6</td>
<td>21.9</td>
<td>26.8</td>
<td>23.1</td>
</tr>
<tr>
<td>Slope Angle (°)</td>
<td>34</td>
<td>38</td>
<td>33</td>
<td>35</td>
<td>38</td>
<td>43</td>
<td>34</td>
</tr>
<tr>
<td>y-intercept</td>
<td>1.7</td>
<td>5.9</td>
<td>3.8</td>
<td>3.7</td>
<td>6.5</td>
<td>-1.05</td>
<td>3</td>
</tr>
</tbody>
</table>

The results show a range of slope angles between 28° and 43° for the peak shear stress. A similar analysis using large displacement shear stress values gave 19° to 36° for the same interface. The same analysis was carried out for the peak and large displacement shear strength values of the smooth geomembrane interface, which gave slope angles between 3° to 35° and 5° to 35° respectively. The results indicate that there could be a large variability of shear strength parameters when only one test is conducted at each normal stress.

5.1.5 Monte Carlo Simulation of Laboratory Peak Shear Strengths

A Monte Carlo simulation was carried out on the data obtained at the various normal stresses to obtain the distributions of peak strength parameters ($\alpha_p$ and $\delta_p$) that are calculated when sets of three strengths are selected randomly (i.e. one from each normal stress) and a best-fit straight line calculated.

The Monte Carlo simulation is a sampling method that uses random numbers to measure the effects of uncertainty in a spreadsheet model. The simulation calculates
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numerous scenarios of a model by repeatedly picking values from the probability distribution for the uncertain variable and using those values for the cell. The procedure used is as follows:-

1. Find mean & standard deviation of the measured shear strength data for each normal stress

2. Define an assumption about the mean shear strength using the calculated standard deviation. This is done by choosing the probability distribution that best describes the uncertainty of the data in the cell. The normal distribution is generally taken for shear strength data because it is assumed that
   a) Some value of the shear strength (the uncertain variable in this case) is the most likely (the mean of the distribution)
   b) The uncertain variable is symmetrical about the mean (i.e. it could be above or below the mean)
   c) The uncertain variable is more likely to be in the vicinity of the mean than far away.

3. Define the decision variable cell - this contains numeric values e.g. normal stress at 10, 25 or 30 kPa, which can be changed.

4. Define the forecast cells. Forecast cells (dependent variables) contain formulas that refer to one or more assumption and decision variable cells. Cells containing formulas for the slope, intercept and shear strength at a particular normal stress are defined.

5. The pairs of shear strength parameters i.e. intercept ($\alpha$) and slope ($\delta$) values are defined in terms of mean and standard deviation. The pairs that define each best fit straight line are used to calculate the shear strength ($\tau$) at the desired normal stress.

6. Set the preferences e.g. number of trials required.

7. From the simulation, the statistical mean of the forecast cell for shear strength, is then taken as the design shear strength value.
For the Series 2 repeatability tests, a total of 1000 trials were conducted for each interface. A summary of the results from simulations in terms of mean and standard deviation of the calculated parameters is shown in Table 5.3. The pairs of shear strength parameters that define each best fit line have been used to calculate the shear strength for a normal stress of 20 kPa, typical for capping systems.

Table 5.3 Distribution of Peak Shear Strength Parameters

<table>
<thead>
<tr>
<th>Statistics</th>
<th>Smooth gm/gt</th>
<th>Textured gm/gt</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha$ mean (kPa)</td>
<td>1.0</td>
<td>3.7</td>
</tr>
<tr>
<td>SD* (kPa)</td>
<td>2.4</td>
<td>1.9</td>
</tr>
<tr>
<td>$\delta$ mean (°)</td>
<td>6.9</td>
<td>17.5</td>
</tr>
<tr>
<td>SD (°)</td>
<td>1.1</td>
<td>1.4</td>
</tr>
<tr>
<td>$\tau @\sigma_n = 20$ kPa mean (kPa)</td>
<td>16.2</td>
<td>34.5</td>
</tr>
<tr>
<td>SD (kPa)</td>
<td>7.8</td>
<td>3.5</td>
</tr>
</tbody>
</table>

* SD is standard deviation

An example of results obtained from the Monte Carlo simulation for the textured geomembrane/geotextile test data for 750 trials is shown in Figure 5.1, 5.2 and 5.3, for the slope ($\delta_p$), intercept ($\alpha_p$) and shear strength ($\tau$) at 20 kPa normal stress, respectively.

Figure 5.1 Frequency chart of slope forecast for Tgm/gt (750 trials)
From these results, three random tests carried out one each at the selected normal stresses would produce shear strength parameters within a range (i.e. it would not be known from just one set of tests, whether the shear strengths measured were high, medium or low values). Data that produces high shear strengths based on one test at each normal stress, could overestimate field values by up to 20%.

These results indicate that contrary to current testing and design practice, more than one test per normal stress is necessary if a more accurate and reliable interface shear
strength value is to be obtained for a particular design problem. This would also allow uncertainty in the design process to be considered.

5.2 Reproducibility Testing

Shear box reproducibility testing carried out as part of a joint research project between Loughborough and Hanover Universities involved geotextile/sand and geomembrane/geotextile interfaces tested using four different devices. The results of this testing program have been presented in Section 4.2. The results highlight variability in test data that could be due to shear box design and/or material variability.

5.2.1 Geotextile/Sand Interface

The results of the 1995/1996 German intercomparison laboratory testing program presented in Figure 2.8 have been plotted with those obtained for the UK-German reproducibility testing program involving the Loughborough and Hanover universities. These are presented in Figure 5.4.

Figure 5.4  Intercomparison shear stress/normal stress plots for sand/geotextile
The materials used in the three test programs i.e. 1995, 1996 and 2000 are the same. For the same normal stress, the UK-German data generally shows higher peak shear stresses. For both sets of data, coefficient of variation (CoV) values were calculated for the normal stresses 25 - 200 kPa as shown in Figure 5.5.

![Figure 5.5 Plot of CoV vs. Normal Stress for sand/geotextile interface](image)

The results indicate a generally higher CoV at the lower normal stresses for both sets of data. The lower CoV values shown by the UK-German tests could be due to a detailed strict testing procedure and a limitation on the number of operators and devices, whereas the 1995/1996 data set is for twenty laboratories using a range of DSA designs and with different operators.

A shear stress/normal stress plot for the sand/geotextile interface using all the devices involved in the UK-German intercomparison tests is presented in Figure 5.6.
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5.2.2 Textured Geomembrane/Geotextile Interface

This interface was tested with sand in the top box above the geotextile, instead of spacer blocks. The results have been presented in Section 4.2.2. Peak strengths for this interface for all the devices used are shown in Figure 5.7.

Figure 5.6 UK-German sand/geotextile interface results for all devices used

The results indicate that the fixed top devices consistently yield relatively higher peak shear strength values compared to other devices. This is because the fixed top box controls the position and formation of the shear plane in the tests involving soil on the interface. This influences the shear strength results as well as the change in normal stress, and results in an over estimation of the shear strength compared with the other devices. Results of the floating and vertically movable top box designs are generally similar.
The results still show variability in the data obtained for the three fixed top boxes and the vertically movable top design DSA. This is possibly due to differences in the normal stress being transmitted to the interface from the top of the sample, during shear.

5.2.3 Interpretation of UK-German Reproducibility Test Data

A detailed evaluation of the intercomparison results indicates that one of the main reasons for the scatter of measured strength data is the different designs of shear devices used. The higher stresses yielded by the fixed top device for the sand/geotextile interface compared with other devices are likely to be due to the amount and time dependent variation of the normal stress on the interface during shear. The normal stress, unless measured on the interface, is only known at the point of application i.e. the top of the sample. Friction between the test material and internal walls of the top box both during application of the normal stress and during shear will alter the amount of normal stress acting in the shear plane by an amount that is not known.
Stoewahse (2001) modified the fixed top box to measure the average normal stress acting on the interface. He separated the upper box from the loading system, to allow the upper box to move vertically, but not to rotate during the test. The vertically movable top box together with a control system ensures that the vertical stress applied at the interface remains constant during the shear test. This construction was selected as the standard DSA design and incorporated in the German DIN 18 137-3. The results have been reported by Stoewahse et al, (2002).

5.3 Selection of Appropriate Material Factors

Current practice is to carry out one, at best two, site-specific tests at each normal stress. This is not adequate to allow a statistical analysis of the results and hence to enable characteristic shear strength parameters to be obtained.

The extensive programme of repeatability and inter laboratory comparison interface shear strength testing has provided evidence of large variability in measured geosynthetic interface shear strengths.

5.3.1 Obtaining Characteristic Shear Strengths Parameters

Characteristic shear strength parameters from the laboratory tests could be obtained by:

- Carrying out enough site specific tests to enable a statistical analysis of the characteristic shear strength parameters. This approach is expensive and errors could result from carrying out an inadequate number of tests. The approach relies on the experience of the engineer interpreting the test results.

- Taking the best fit straight line through the lowest measured shear strength at each normal stress (for three tests conducted at each of three normal stresses). This is consistent with guidance in Eurocode 7 (1997) Part 2, Table A.9.2. While a smaller number of tests can be carried out than in the previous approach, it could lead to over conservative (i.e. low) strength parameters being calculated.
Obtaining cautious characteristic values using a limited number of site-specific tests. This approach was used in a study by Dixon et al. (2002) to analyse the repeatability and reproducibility (intercomparison) test data. The variability of the measured interface shear strengths from these extensive tests, in which the Author participated, was analysed to provide statistical information on the magnitude of scatter of measured shear strengths. Limit strength test data was corrected using a proposed relationship between the normal stress and standard deviation of the results. This quantified the variability of typical test data using the results from the extensive repeatability test programme.

The main interfaces considered were:

a) Non woven needle punched geotextile vs. geomembrane

b) Non-woven needle punched geotextile vs. sand.

Dixon et al. (2002) advise that use of the measured strengths as mean values is dependent upon an assessment by an experienced engineer. If there is limited prior information on an interface, this approach should not be used.

5.3.2 Importance of Compliance Testing during Construction

In general, selection of characteristic values for soil and geosynthetic properties should consider the inherent variability of soil and manufactured geosynthetics, measurement errors (due to equipment, procedure, operator and etc), and the extent of the zone governing the behaviour of limit state being considered.

The properties of compacted soils are easily and significantly influenced by construction processes. Increasing the moisture content of a cohesive soil that is in close proximity to a geosynthetic can result in large reductions in interface shear strength. Compliance testing should, therefore be undertaken frequently during construction, to validate the assumed values of shear strength used in design and to ensure the stability and integrity of the geosynthetic/soil system. The testing must consider the area of the interface tested, the number and complexity of factors influencing interface shear strength and the implication of failure.
5.4 Geocomposite Grid Direction

Geocomposite grid direction tests were carried out on the Terram 1B1 geocomposite/Leighton Buzzard sand interface in both the fixed and vertically movable devices. The results of this test series are presented in Section 4.3.

5.4.1 Geocomposite T Results Analysis

Figure 4.15 shows that the shear stress for T90, the geocomposite grid perpendicular to the direction of shear gives the highest shear strength of the three directions, with a peak shear stress of 41 kPa compared to 34 kPa and 31 kPa for T60 and T180 respectively at 30 kPa normal stress. Figure 4.16 shows an increase in normal stress within the first 10 - 20mm of shear. The shear stress/normal stress vs. displacement plot presented in Figure 4.17 shows that the highest normal stress does not necessarily coincide with the highest shear strength for geocomposite T60.

5.4.2 Interpretation of Geocomposite Grid Direction Tests

The results obtained from the fixed and vertically movable devices are consistent with those found by Stoewahse (2001) from tests on sand/geotextile interfaces at higher normal stresses. Results obtained from the vertically movable top box which has a constant normal stress on the interface show very good repeatability and the lowest interface friction angle.

The fixed top box is shown to significantly over estimate the derived interface shear strength parameters, especially when the normal stress at the interface is assumed to be equivalent to that on the interface. Measuring the normal stress on the interface results in slightly lower shear strength parameter results in comparison with those undertaken in the fixed box without measurement of normal stress, but these are still higher than in the vertically movable device.

Stoewahse (2001) has shown that the vertically movable top box design gives correct interface shear strengths while the fixed top box design consistently over estimates the shear strength. It is apparent that the fixed top design constrains the position and hence formation of the shear surface. It is thought that as the sand penetrates into the troughs between the grid ribs, it causes the formation of a corrugated and hence
complex shear surface. A planar shear surface is unable to form higher up (in the sand) due to the constraint applied by the fixed box, resulting in the measurement of higher and hence unconservative shear strengths.

The tests conducted in this study also indicate that at low normal stresses the angle of friction of the Terram 1B1 geocomposite/sand interface can vary by as much as 4° depending on the direction of shearing in relation to the orientation of the core.

Examination of geocomposite T samples after shear showed that the sand grains often got stuck in the woven fibres and pulled the fibres along as the bottom box moved, sometimes giving a strain hardening effect (i.e. no distinct peak). Similar studies undertaken on two other geocomposites of different grid design but sandwiched in a non woven geotextile did not have the same effect. A difference in girth and height, as a distinction between the major and minor grid is thought to encourage a sinusoidal shear plane, with activity on top and between the main grids, i.e. pushing the shear plane into sand, leading to higher shear strengths. The non-woven geotextile is flexible to allow this effect.

5.5 Measuring PWP on a Clay/Geomembrane Interface

It was attempted to measure pore water pressures on the clay/geomembrane interface during shear, to establish if the clay/geomembrane interface was a drainage path and to obtain effective stress parameters for this interface at the normal stresses used.

5.5.1 Data Analysis

The responses of the transducers indicate that pore water pressures in the sample responded to increments of normal stress application, and dissipated during shear. A plot of the midpoint of stress range vs. pore pressure coefficient B for the first trial shown in Figure 4.26 is presented in Figure 5.8. Pore pressure coefficient B is the ratio of change in pore pressure to change in the applied normal stress.
Change in Pore Pressure Coefficient B with Normal Stress in large DSA

Figure 5.8 Plot of variation of B value with normal stress (Trial 1)

The results show that the B values were all significantly less than 1, suggesting that the sample was unsaturated. Figure 5.8 also shows that the B values are generally decreasing with increasing normal stress. This suggests that there is an increasing amount of air in the system, probably in the transducer assembly and/or at the interface.

A plot of the mid-point stress vs. pore pressure coefficient B for the results of the second experiment (Trial 2) shown in Figure 4.27 is presented in Figure 5.9. The results show B-values greater than 1, indicating an anomaly in the test, as for a saturated soil, B = 1. A review of the shape of the plots presented in Figure 4.27 and 5.9 suggests that there must have been stress concentrations under the transducer assembly in the plastic block, which led to exceptionally high increases in pore water pressure, as measured by the transducers.
5.5.2 Interpretation of PWP Measurement Test Results
The uncertainty introduced by the challenges of keeping the transducers de-aired and ensuring they do not modify the stress conditions at the interface means that there can be little confidence in the results obtained. The results presented in Figures 5.8 and 5.9 show improvement in pore water pressure response. The improvement is shown by the calculated increased B values and the rapid response in Trial 2, indicating that with great care it is possible to keep the pore water pressure system de-aired.

However, the calculation of B values significantly greater than 1 in Trial 2 can only have been caused by stress concentrations under each transducer. This is despite care being taken to make the transducer assembly flush with the underside of the loading block. The results in Trial 2 indicate that the method used is flawed. Further trials must overcome this problem.
Due to the lack of confidence in the measured pore water pressures for the reasons discussed above, it is not possible to interpret the shearing phase with any certainty. The key question is whether the clay/geomembrane interface is a drainage path. If it is, then potentially shearing rates can be increased, thus reducing testing times, to obtain effective shear strength parameters. The results are inconclusive but indicate that it this interface might be a partial drainage path (i.e. some dissipation of pore water pressures by flow along the interface but retarded). This would occur if the hydraulic conductivity of the interface is greater than the clay (i.e. controlling vertical flow to the drainage boundary at the base of the clay).
6. MAIN FINDINGS

This section reviews the original aims and objectives of this research, the findings from the laboratory experiments, and the implications of these findings on theory, industry and testing. The challenges encountered are discussed and recommendations for further research put forward.

6.1 Original Aims

The original aims of this research project were:

- To develop conclusive methods for obtaining interface shear strength parameters at low normal stresses relevant to capping system design.
- To investigate the structural performance/shear behaviour of recently developed geocomposite drainage materials.
- To assess the role played by time dependent pore water pressures generated at the interfaces during shear, on the structural stability of cover systems.

6.2 Main Findings

1) The high degree of scatter has been achieved despite tight controls on the test procedure. Tests conducted to investigate reasons for the scatter have shown that scratching the surface of the smooth geomembrane, and dragging the geotextile over a textured geomembrane to cause light damage, do not significantly affect the measured shear strengths of the respective interfaces. However, introducing a thin film of water onto the surface of the smooth geomembrane tends to reduce the shear strength, and pre-loading the textured geomembrane/geotextile interface before shearing produces an increase in shear strength for the 10 and 20 kPa tests. These factors contribute to the scatter of data but do not fully explain it. Variability of the geosynthetics material properties is considered to be the main cause of scatter.
2) The variability demonstrated in the repeatability and reproducibility tests undertaken in this study suggests that great emphasis should be placed on the accurate site and project specific determination of interface shear strength parameters.

3) Given the unknown degree of variation in measured strengths if a standard set of three normal stresses is used for design, it is not possible to assess the reliability of shear strength parameters obtained. The repeatability test results have been used to demonstrate possible variability of shear strength parameters, and hence of calculated shear strength. These results have an important implication for designers who have to select appropriate factors for use in design. Further work is required to assess other interfaces.

4) Intercomparison test results highlighted inconsistencies and variability in test results obtained by different laboratories. The results show that the support or design of the top box has a significant effect on the test results. Kinematical restrictions of the upper box in the fixed top design have been shown to cause constraint forces in the system and this leads to unconservatively high values of measured interface shear strengths. As a result, a direct shear apparatus with a vertically movable top box was developed at Hanover University.

5) In this prototype vertically movable device, the weight of the box has an influence on the normal stresses. For the box to move vertically (i.e. float) the vertical reaction from the shear force on the front side of the box must be larger than its weight. To use this device at low normal stresses suitable for capping systems, calculations showed the top box would float if the normal stress is greater than 12 kPa.

6) The results obtained from the geocomposite grid orientation tests suggest the frictional resistance of a given geocomposite interface varies with the orientation of its core grid in relation to the direction of shear. Measurement of the normal stress on the interface showed a difference between the normal stress on top of the sample and the normal stress along the interface. Results show that at low normal stresses...
the angle of friction varied by as much as 4° depending on the direction of shearing in relation to the orientation of the core.

7) Results obtained from tests to measure pore water pressure on the clay/geomembrane interface showed that the method used was flawed as it was not possible, with the approach taken, to measure correctly and accurately the pore water pressure on the interface. However, the response of the transducers suggests that the interface might be a partial drainage path.

### 6.3 Implications of Findings on Theory, Industry & Testing

The implications of each of the test programs conducted in this study, on theory, industry and testing are presented below.

#### 6.3.1 Interface Shear Strength Repeatability Test Program

The variations in interface shear strengths observed for the same interface at the same normal stress for the same device, indicate that for design, care should be taken when selecting appropriate factors to apply to the shear strength parameters obtained from standard tests.

In the interests of best practice laboratory testing procedures, it is clear from the special condition smooth geomembrane/geotextile tests that every attempt must be made to ensure that all moisture is removed from the surface of the geomembrane before testing, unless submerged tests are required. This includes perspiration transferred to the surface during handling.

The results also indicate that contrary to current testing and design practice, more than one test per normal stress is necessary if a more accurate and reliable interface shear strength value is to be obtained for a particular design problem. Interface shear strength values used on a given project must be specific to that particular project and its site conditions.

The results of the pre-load tests have important implications for development of laboratory testing procedures. Care must be taken to ensure that the normal
stress at which shearing takes place is not exceeded during test set up. Filling of the top box (i.e. with spacer blocks or cover soil if used in the test) and application of the normal stress must not result in the application of an excess stress. Otherwise, increased shear strengths will be measured leading to unconservative (i.e. high) values being used in design. The implications of pre-loading during construction are not important because any gain in strength will be beneficial.

6.3.2 Interlaboratory Interface shear strength comparison test program

From the inter-laboratory comparison test programmes, the results indicate that small changes in testing conditions could significantly affect the results, much more so at the lower range of normal stresses suitable for capping systems.

It is recommended that all test results must be accompanied by a detailed test report to include the following information:

- A description of the test device including the design of the top box, how the normal load was applied, etc.
- Full material descriptions to include manufacturer, mass per unit area, polymer thickness, structure etc for geosynthetics and origin, soil mechanical classification and other mechanical parameters for soils.
- Describe how selection of samples was undertaken, how samples were prepared e.g. if soaked prior to testing. Describe any form of pre-treatment of soils e.g. crushing of aggregates, drying, adding water – if applicable.
- The test set up and boundary conditions including method of placement of soils (e.g. compactive effort and layer thickness), how the geosynthetics were fixed, consolidation time of soil, seating time, pre-loading/hydration time, density and moisture content before and after the test.
- Test results to include shear stress vs. displacement curves, peak shear stress vs. normal stress plots, large displacement shear stress vs. normal...
stress plots, volumetric changes vs. displacement if relevant, soil mechanics parameters at the beginning and end of test, shear parameters and the method of derivation (e.g. linear regression).

- A description of the state of the materials after the test e.g. if the geosynthetics stretched during shearing, abrasion of geomembrane textures, orientation of geotextile fibres, post shearing damage e.g. development of additional shear zones, changes in soil moisture content, etc.

- These recommendations have been reported by Stoewahse et al., (2002), in joint publication with the Author. It is emphasized that the interface shear strength values used on a given project must be specific to that particular project and its site conditions.

6.3.3 Geocomposite Grid Orientation

Grid orientation in the field becomes particularly important after sliding is initiated. When the direction of the main grid is transverse to the direction of movement, the magnitude of slip that occurs after shear strength at the interface is mobilised is likely to be relatively small. On the other hand, if the direction of the main grid is parallel to the slope, larger displacements can result following slippage, because of the relatively low value of post peak shear strengths available. It is, therefore, important that the designer specify the orientation of geocomposites on site to ensure that the design strengths are available so that stability is not compromised. In addition, construction and site operations should be carefully considered so that the possibility of mobilising post-peak shear strengths is minimised.

Geocomposite placement especially in corners and confined spaces along capping slopes should not be left to the jurisdiction of the contractor but incorporated in the design guidelines if long term slope stability is to be attained. Material placement should also minimise dragging. Geocomposites in particular, should be carefully handled so that the bond between the geotextile
and the geogrid and between that between geogrids (i.e. the internal strength) is not weakened.

6.3.4 Measurement of PWP on the Interface

A methodology to measure pore water pressure along the geomembrane/clay interface during loading and shear has been suggested but was unsuccessful. It still needs to be developed to a stage it can be easily applied in industry.

Although through development of the test procedure it was possible to produce and maintain a de-aired measurement system (Trial 2), the stress conditions on the interface were not uniform. Stress concentrations at the interface under the transducers were indicated by the calculated high B values.

This means that the measured pore water pressures are not representative of the rest of the interface, and hence cannot be used to calculate the effective normal stress on the interface. Therefore, effective shear strength parameters could not be calculated, which was the aim. In addition, faster shearing rates cannot be justified, such as undrained shearing with pore water pressure measurement to obtain effective stress strength parameters.

The fact that the pore water pressures dissipate relatively rapidly after application of normal stress and during shearing suggests that the clay/geomembrane interface is a partial drainage path. However, uncertainty in the measured pore water pressures means that it was not possible to quantify the drainage conditions.

6.4 Challenges Encountered

- Using the different designs of large direct shear device at low normal stresses of 10 – 30 kPa challenged the integrity of the results. This is in consideration of the constraints imposed by the design of the fixed top box and the stress applied on the sample by the arrangement of the fixed top box. In addition, during the geomembrane/geotextile and geotextile/sand tests carried out on the vertically movable device, it was
realised that the loading system for this device design applied nearly 8 kPa to the interface, before introduction of normal stress via the pneumatic bag.

- For the proposed pore water pressure measuring methodology, it was very difficult to keep the transducers de-aired especially during the fixation of the brass bosses to the geomembrane, which was done in a bath of de-aired water. It was necessary that the screws were countersunk satisfactorily and the transducers’ electrical leads kept free of water. It was challenging to keep the geomembrane-transducer assembly free of air during transfer from the water bath to the sample in the shear box.

- Although placement of the plastic block was undertaken with care, it was difficult to judge whether the bosses were positioned proud of the plastic block.

- The whole experimental set up was very time consuming.

**6.5 Suggestions for Further Research**

- It is suggested to modify the vertically movable top box shear device design so that low normal stresses suitable for capping systems can be applied accurately and reliably.

- It is suggested to undertake numerous repeatability tests on other geosynthetic interfaces typically used in landfills at the high normal stresses suitable for basal and side slope liners and the low normal stresses suitable for capping, to explain and quantify the variability in measured shear strengths. This will provide a basis for design calculations.

- It is suggested to develop a waterproof sealant between the geomembrane and brass boss, as the tests presented by the Author depended on an O-ring to provide an air seal.

- It is suggested to develop a water and airproof sealant that could be placed in the transducer positions on the underside of the geomembrane;
a seal that could be broken or peeled back on contact with the soil. That way, the transducers would be kept de-aired during transfer of the geomembrane-transducer assembly to the top of the soil sample.

- It is further suggested to work with a suitable pore pressure transducer manufacturer to design water proof transducers suitable for the proposed use of measurement of pore water pressure in the large DSA. The transducers could be designed to be placed either within the soil sample in the lower box of the shear box or just above the geomembrane, with access to the clay/geomembrane interface.
REFERENCES

a) Testing Standards


ASTM D 5321 1992 American Standard Test Method for Determining the Coefficient of Sand and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method


BS 1377: Part 7 1990 British Standard Methods of Test for Soils for Civil Engineering Purposes: Part 7. Shear Strength Tests (Total Stress), British Standards Institution, London


b) Authors


References


Investigating the stability of geosynthetic landfill capping systems
Appendix A
Manufacturers Data Sheets

10 Pages

A.1: GSE Geomembranes
A.2: Geofabrics Geotextiles
A.3: Terram 1B1 Geocomposites
A.4: Druck Pore Pressure Transducer
The minimum values for the geomembranes & geotextiles have been taken from GSE's Geomembrane Drop In Specifications dated 12th August 2004.

Table 1.1: Minimum Values for Smooth Black-Surfaced HDPE Geomembranes

| Property                        | Test Method(|)  | Thickness, mil (mm) | Minimum Average | Lowest Individual Reading | Density, g/cm³ | Carbon Black Content, % | Carbon Black Dispersion | Tensile Properties: (each direction) | Tear Resistance, lb (N) | Puncture Resistance, lb (N) | Notched Constant Tensile Load, hours | Oxidative Induction Time, min. |
|---------------------------------|---------------|---------------------|-----------------|---------------------------|----------------|------------------------|--------------------------|-------------------------------|-------------------------|-------------------------------|--------------------------------|-------------------------------|
|                                 |               | 30 (0.75)           | 40 (1.0)        | 60 (1.5)                  | 80 (2.0)       | 100 (2.5)              | 120 (3.0)                |                               | 21 (93)                 | 59 (263)                      | 400                             | 100                           |
|                                 |               | 27 (0.69)           | 36 (0.91)       | 54 (1.4)                  | 72 (1.8)       | 90 (2.3)               | 108 (2.7)                |                               | 28 (124)                | 79 (352)                      | 400                             | 100                           |
| Thickness, mil (mm)             | ASTM D 5199   | 0.94                | 0.94            | 0.94                      | 0.94           | 0.94                   | 0.94                     |                               | 0.94                    | 0.94                          | 0.94                           | 0.94                          |
| Minimum Average                 |               | 2.0                 | 2.0             | 2.0                       | 2.0            | 2.0                    | 2.0                      |                               |                         |                               |                                 |                               |
| Lowest Individual Reading       |               | Note 2              | Note 2          | Note 2                    | Note 2         | Note 2                 | Note 2                   |                               |                         |                               |                                 |                               |
| Density, g/cm³                  | ASTM D 1505   | 0.94                | 0.94            | 0.94                      | 0.94           | 0.94                   |                         |                               |                         |                               |                                 |                               |
| Carbon Black Content, %         | ASTM D 1603,  | 2.0                 | 2.0             | 2.0                       | 2.0            | 2.0                    | 2.0                      |                               |                         |                               |                                 |                               |
| Carbon Black Dispersion         | mod.          | Note 2              | Note 2          | Note 2                    | Note 2         | Note 2                 | Note 2                   |                               |                         |                               |                                 |                               |
| Tensile Properties: (each direction) |               |                     |                 |                           |                |                        |                          |                               |                         |                               |                                 |                               |
| Strength at Yield, lb/in (kN/m) | ASTM D 6693   | 63 (11)             | 84 (15)         | 130 (23)                  | 173 (30)       | 216 (38)               | 259 (45)                 |                               |                         |                               |                                 |                               |
| Strength at Break, lb/in (kN/m)|               | 122 (21)            | 162 (28)        | 243 (43)                  | 324 (57)       | 405 (71)               | 486 (85)                 |                               |                         |                               |                                 |                               |
| Elongation at Yield, % (1.3" gauge length) |               | 13                  | 13              | 13                        | 13             | 13                     | 13                       |                               |                         |                               |                                 |                               |
| Elongation at Break, % (2.0" gauge length) |               | 700                 | 700             | 700                       | 700            | 700                    | 700                      |                               |                         |                               |                                 |                               |
| Tear Resistance, lb (N)         | ASTM D 1004   | 21 (93)             | 28 (124)        | 42 (187)                  | 56 (249)       | 70 (311)               | 84 (373)                 |                               |                         |                               |                                 |                               |
| Puncture Resistance, lb (N)     | ASTM D 4833   | 59 (263)            | 79 (352)        | 119 (530)                 | 158 (703)      | 180 (800)              | 216 (960)                |                               |                         |                               |                                 |                               |

Investigating the stability of geosynthetic landfill capping systems
Table 1.4: Minimum Values for Smooth Black-Surfaced LLDPE Geomembranes

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Minimum Average</th>
<th>Lowest Individual Reading</th>
<th>Density, g/cm³</th>
<th>Carbon Black Content, %</th>
<th>Carbon Black Dispersion</th>
<th>Tensile Properties:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thickness, mil (mm)</td>
<td>ASTM D 5199</td>
<td>30 (0.75)</td>
<td>27 (0.69)</td>
<td>0.92</td>
<td>2.0</td>
<td>Note 2</td>
<td></td>
</tr>
<tr>
<td>Minimum Average</td>
<td></td>
<td>40 (1.0)</td>
<td>36 (0.91)</td>
<td>0.92</td>
<td>2.0</td>
<td>Note 2</td>
<td></td>
</tr>
<tr>
<td>Lowest Individual Reading</td>
<td></td>
<td>60 (1.5)</td>
<td>54 (1.4)</td>
<td>0.92</td>
<td>2.0</td>
<td>Note 2</td>
<td></td>
</tr>
<tr>
<td>Minimum Average</td>
<td></td>
<td>80 (2.0)</td>
<td>72 (1.8)</td>
<td>0.92</td>
<td>2.0</td>
<td>Note 2</td>
<td></td>
</tr>
<tr>
<td>Lowest Individual Reading</td>
<td></td>
<td>100 (2.5)</td>
<td>90 (2.3)</td>
<td>0.92</td>
<td>2.0</td>
<td>Note 2</td>
<td></td>
</tr>
<tr>
<td>Density, g/cm³</td>
<td>ASTM D 1505</td>
<td>0.92</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Carbon Black Content, %</td>
<td>ASTM D 1603, mod.</td>
<td>2.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Carbon Black Dispersion</td>
<td>ASTM D 5596</td>
<td>Note 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile Properties: (each direction)</td>
<td>ASTM D 6693</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength at Break, lb/in (kN/m)</td>
<td></td>
<td>114 (20)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elongation at Break, % (2.0&quot; gauge length)</td>
<td></td>
<td>152 (27)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tear Resistance, lb (N)</td>
<td>ASTM D 1004</td>
<td>16 (71)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Puncture</td>
<td>ASTM D 4833</td>
<td>46 (205)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oxidative Induction Time, min.</td>
<td>ASTM D 3895</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Investigating the stability of geosynthetic landfill capping systems
### Table 2.3: Minimum Values for Black Surfaced Coextruded Textured LLDPE Geomembranes

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method(1)</th>
<th>Minimum Average</th>
<th>Lowest Individual Reading</th>
<th>Density, g/cm³</th>
<th>Carbon Black Content, %</th>
<th>Carbon Black Dispersion</th>
<th>Tensile Properties(2): (each direction)</th>
<th>Tear Resistance, lb (N)</th>
<th>Puncture Resistance, lb (N)</th>
<th>Oxidative Induction Time, min.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness, mil (mm)</td>
<td>ASTM D 5994</td>
<td>40 (1.0)</td>
<td>36 (0.91)</td>
<td>0.92</td>
<td>2.0</td>
<td>Note 3</td>
<td>100 (18)</td>
<td>22 (100)</td>
<td>48 (214)</td>
<td>100</td>
</tr>
<tr>
<td>Minimum Average</td>
<td></td>
<td>60 (1.5)</td>
<td>54 (1.4)</td>
<td>0.92</td>
<td>2.0</td>
<td>Note 3</td>
<td>132 (23)</td>
<td>33 (150)</td>
<td>73 (325)</td>
<td>100</td>
</tr>
<tr>
<td>Lowest Individual Reading</td>
<td></td>
<td>80 (2.0)</td>
<td>72 (1.8)</td>
<td>0.92</td>
<td>2.0</td>
<td>Note 3</td>
<td>176 (30)</td>
<td>44 (200)</td>
<td>97 (432)</td>
<td></td>
</tr>
<tr>
<td>Density, g/cm³</td>
<td>ASTM D 1505</td>
<td>0.92</td>
<td>0.92</td>
<td>0.92</td>
<td>2.0</td>
<td>Note 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Carbon Black Content, %</td>
<td>ASTM D 1603, modified</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>Note 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Carbon Black Dispersion</td>
<td>ASTM D 5596</td>
<td>Note 3</td>
<td>Note 3</td>
<td>Note 3</td>
<td>Note 3</td>
<td>Note 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile Properties(2): (each direction)</td>
<td>ASTM D 6693</td>
<td>500</td>
<td>500</td>
<td>500</td>
<td>500</td>
<td>Note 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength at Break, lb/l (kN/m)</td>
<td></td>
<td>100 (18)</td>
<td>132 (23)</td>
<td>176 (30)</td>
<td>22 (100)</td>
<td>48 (214)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elongation at Break, % (2.0&quot; gauge length)</td>
<td></td>
<td>500</td>
<td>500</td>
<td>500</td>
<td>33 (150)</td>
<td>73 (325)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tear Resistance, lb (N)</td>
<td>ASTM D 1004</td>
<td>22 (100)</td>
<td>33 (150)</td>
<td>44 (200)</td>
<td>48 (214)</td>
<td>73 (325)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Puncture Resistance, lb (N)</td>
<td>ASTM D 4833</td>
<td>48 (214)</td>
<td>73 (325)</td>
<td>97 (432)</td>
<td>48 (214)</td>
<td>73 (325)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oxidative Induction Time, min.</td>
<td>ASTM D 3895</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(1) ASTM = American Society for Testing and Materials
(2) Tensile Properties: Strength at Break, Elongation at Break, Tear Resistance, Puncture Resistance

Investigating the stability of geosynthetic landfill capping systems
| Test | Fibre type | Static puncture strength (CBR) (MN) | Penetration displacement (mm) | Tensile strength (kN/m) | Tensile elongation (%) | Cone drop (mm) | Thickness @ 20°C (mm) | Apparent pore size (μm) | Waterflow (l/min) | Coefficient of permeability (m/s) | @ 210 kPa (m/s) | @ 250 kPa (m/s) | Coefficient of permeability (m/s) | Standard roll diameter (mm) | Approximate roll diameter (°) | Approximate roll weight (kg) |
|------|------------|----------------------------------|-----------------------------|------------------------|-----------------------|-----------------|----------------------|----------------------|-----------------|-----------------------------|----------------|----------------|-----------------------------|------------------------|------------------------|
| 1    | HP2        | 2.3                             | 1.5                          | 60                     | 0.5                   | 120             | 0.2                  | 0.01                 | 0.003           | 0.001                       | 0.0005         | 0.0001         | 0.001                       | 0.0005                 | 0.0001                 |
| 2    | HP3        | 2.4                             | 1.6                          | 65                     | 0.6                   | 130             | 0.25                 | 0.015                | 0.003           | 0.001                       | 0.0005         | 0.0001         | 0.001                       | 0.0005                | 0.0001                |
| 3    | HP4        | 2.6                             | 1.7                          | 70                     | 0.7                   | 140             | 0.3                  | 0.02                 | 0.003           | 0.001                       | 0.0005         | 0.0001         | 0.001                       | 0.0005                | 0.0001                |
| 4    | HP5        | 2.7                             | 1.8                          | 75                     | 0.8                   | 150             | 0.35                 | 0.025                | 0.003           | 0.001                       | 0.0005         | 0.0001         | 0.001                       | 0.0005                | 0.0001                |
| 5    | HP6        | 2.9                             | 1.9                          | 80                     | 0.9                   | 160             | 0.4                  | 0.03                 | 0.003           | 0.001                       | 0.0005         | 0.0001         | 0.001                       | 0.0005                | 0.0001                |
| 6    | HP7        | 3.0                             | 2.0                          | 85                     | 1.0                   | 170             | 0.45                 | 0.035                | 0.003           | 0.001                       | 0.0005         | 0.0001         | 0.001                       | 0.0005                | 0.0001                |
| 7    | HP8        | 3.1                             | 2.1                          | 90                     | 1.1                   | 180             | 0.5                  | 0.04                 | 0.003           | 0.001                       | 0.0005         | 0.0001         | 0.001                       | 0.0005                | 0.0001                |
| 8    | HP9        | 3.3                             | 2.2                          | 95                     | 1.2                   | 190             | 0.55                 | 0.045                | 0.003           | 0.001                       | 0.0005         | 0.0001         | 0.001                       | 0.0005                | 0.0001                |

**Notes:**
- HP (High Performance) & HPS (High Performance Square) non-woven needlepunched geotextiles.
- Tensile properties and other physical characteristics vary by type.
- Use for maximum mechanical performance - light weight. Significant mass of fibre will be included to achieve these performance values.
- Values are typical with the exception of Thickness, which is nominal. Typical indicates the mean value derived from the samples tested for any one test as defined in the BS EN ISO standard — usually the mean of two samples. Normal is a grade value.

No warranty is given or implied for the use of this information in design and installation as not beyond our control. Geofabrics Limited retains the right to change specifications without notice.
**Terram 1B1**  
**Drainage Composite**

### Construction
- **Filter / separator**: Nonwoven thermally bonded geotextile (polypropylene / polyethylene)
- **B Core**: Extruded net (polyethylene)
- **Filter / separator**: Nonwoven thermally bonded geotextile (polypropylene / polyethylene)

### Product Grade: Terram 1B1

#### Hydraulic Properties - composite

<table>
<thead>
<tr>
<th>Hydraulic gradient</th>
<th>Surfaces: Hard / Hard</th>
<th>Surfaces: Hard / Soft</th>
</tr>
</thead>
<tbody>
<tr>
<td>i</td>
<td>20 kPa</td>
<td>100 kPa</td>
</tr>
<tr>
<td></td>
<td>20 kPa</td>
<td>100 kPa</td>
</tr>
<tr>
<td></td>
<td>200 kPa</td>
<td>400 kPa</td>
</tr>
<tr>
<td></td>
<td>i=1.0</td>
<td>i=1.0</td>
</tr>
<tr>
<td></td>
<td>i=0.5</td>
<td>i=0.5</td>
</tr>
<tr>
<td></td>
<td>i=0.1</td>
<td>i=0.1</td>
</tr>
<tr>
<td>l/m.s (10^-3 m^2/s)</td>
<td>0.70</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td>0.60</td>
<td>0.47</td>
</tr>
<tr>
<td></td>
<td>0.56</td>
<td>0.45</td>
</tr>
</tbody>
</table>

#### Filter (Terram 1000/UV)

- Pore size EN ISO 12956
- Mean AOS Oₐ₀ mm: 0.15
- Permeability EN ISO 11058
- Vₖ=100 l/m².s (10^-3 m/s): 100

#### Mechanical Properties - composite

- **Tensile strength EN ISO 10319** kN/m: 20
- **CBR puncture resistance EN ISO 12236** N: 3700

### Product Dimensions

- **Mass per unit area EN 965** g/m²: 770
- **Thickness EN 964-1** mm: 5.0
- **Roll width** m: 2 or 4
- **Roll length** m: 25, 50, or 100
- **Filter overlap (one side)** mm: 100

---

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Certificate No: FM 22730

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Terram 1B1
Drainage Composite

Notes
1 Terram 1B1 comprises a polyethylene, three-dimensional net structure, sandwiched between two layers of Terram 1000/UV geotextile and bonded together by thermal lamination.
2 The results quoted are the family means of the appropriate tests derived over periods of time.
3 The mechanical tensile strength values quoted are the mean values of either the length or cross directions, whichever is the lower.
4 The in-plane water flow is measured in the length (longitudinal) direction.

The information contained herein is offered free of charge and is, to the best of our knowledge, accurate. However, since the circumstances and conditions in which such information and the products discussed therein can be used may vary and are beyond our control, we make no warranty, express or implied, of merchantability, fitness or otherwise, or against patent infringement, and we accept no liability, with respect to or arising from use of such information or any such product.
General Purpose
Pressure Transducers

- Excellent linearity and hysteresis
  ±0.1% B.S.L. for ranges to 60 bar
- High overload capability
- Rationalised outputs
- Good thermal stability
  ±1.5% total error band -20° to +80°C
- Parameter selection available
Every PDCR 800 transducer is based on a high performance pressure sensor (core) which has subsequently been completed for a specific application by the addition of an electron beam welded pressure connector and an electrical connector assembly. The core itself is an accurate pressure transducer incorporating a high integrity silicon diaphragm and titanium module, a PCB assembly and advanced compensation techniques which give excellent performance over extended temperature ranges. The final assembly is electron beam welded and encapsulated. These cores are produced in large quantities and following automatic calibration over the whole temperature range the data is stored in the computer data base.

The benefits are a high performance to cost ratio series of the transducers listed below, including the core which can be selected and adapted in many different ways and supplied on short delivery.

**Type Number and Specification**

**PDCR 800/801 - Basic core**

**PDCR 810/811 - General purpose**

**PDCR 830/831 - Depth**

**PDCR 860/861 - Integral connector**

This type numbering system denotes the following details:

1. **Type Number and Specification**
2. **Compressed temperature range**
   - 0°C to 50°C
   - 1°C to +80°C
3. **Backend construction & electrical connection**
4. **e.g.: p.tfe cable and reference tube**
5. **800 transducer series**

Please refer to temperature effects, ordering information, assembly diagram and installation drawings to fulfill your requirements.

**Operating Pressure Ranges**
- 70 mbar, 175 mbar, 350 mbar, 700 mbar, 1.5, 2, 3.5, 5, 7, 10, 15, 20, 35 and 60 bar gauge.

Other pressure units can be specified, e.g. psi, kPa, mH2O.

Absolute, differential and sealed gauge transducers are available.

For higher ranges refer to PDCR 900 data sheet.

**Negative Pressure**
- All transducers will accurately respond to pressures below gauge (negative pressures) and will operate with a vacuum applied. The reference side of the PDCR 82X is suitable for atmospheric reference pressures only.

**Overpressure**
- The rated pressure range can be exceeded by the following multiples causing negligible calibration change:
  - 10 x for 70 mbar and 175 mbar ranges,
  - 6 x for 350 mbar range,
  - 4 x for 700 mbar range and above.

Flush fitting version:
- 35 bar range and above maximum pressure 70 bar.

For differential pressures refer to PDCR 101/130 data sheets.

** Burst Pressure**
- In excess of 10 x rated pressure.

**Positive Pressure Media**
- Fluids compatible with silicon and titanium.

**Reference Pressure Media**
- Dry, non-corrosive, non-conducting gases.
- For liquid pressure media on reference, refer to PDCR 120W data sheet.

**Conducting Pressure Media**
- When operating with a conducting pressure media use a fully floating system or earth the +Ve supply.
- If this method is not practicable please refer to PDCR 900 data sheet.

**Transduction Principle**
- Integrated silicon strain gauge bridge.

**Excitation Voltage**
- 10 Volts ±5mA nominal.

**Output Voltage**
- 17mV for 70 mbar range,
- 25mV for 175 mbar range,
- 50mV for 350 mbar range,
- 100mV for 700 mbar ranges and above.

The above outputs are for 10 Volts and are proportional to excitation voltage.

For amplified outputs please refer to PDCR 130 data sheet.

**Common Mode Voltage**
- Typically -6.5 Volts with respect to the +Ve supply at 10 Volts excitation.

**Output Impedance**
- 2900 ohms nominal.

**Load Impedance**
- Greater than 100K ohms for quoted performance.

**Resolution**
- Infinite.

**Combined Non-linearity, Hysteresis and Repeatability**
- ±0.1% of ±0.5% for all ranges.

**Zero Offset and Span Setting**
- ±0.5V maximum.

**Operating Temperature Range**
- -20°C to +80°C standard.
- This temperature range can be extended to 125°C for the PDCR 82X and PDCR 86X.

**Temperature Effects**
- PDCR 8X0
  - ±0.5% total error band 0°C to 50°C for 175 mbar ranges and above.
  - ±0.1% total error band 0°C to 50°C for 700 mbar range.
  - PDCR 8X0: ±0.6%, ±2°C to ±39°C for 700 mbar range, ±0.3%, ±2°C to ±50°C for 175 mbar range and above.

- PDCR 8X1
  - ±1.5% total error band -20°C to +80°C for 175 mbar ranges and above.
  - Typical thermal zero and span coefficients of ±0.015%/°C ±0.02%.
  - For ±5°C to ±125°C temperature range please refer to PDCR 8X2 product note.

**Natural Frequency**
- 28 kHz for 350 mbar increasing to 360 kHz for 35 bar.

**For more detailed information please refer to manufacturer.**

**Acceleration Sensitivity**
- 0.006% F.S./g for 350 mbar decreasing to 0.0002% F.S./g for 35 bar.

**Mechanical Shock**
- 1000 g for 1ms half sine pulse in each of 3 mutually perpendicular axes will not affect calibration.

**Vibration**
- Response less than 0.05% F.S./g at 30g peak 10Hz-2KHz, limited by 12mm double amplitude (MIL-STD 810C Proc 514.2-2 Curve L).

**Weight**
- 100 gms. nominal.

**Electrical Connection**
- 1 metre integral cable supplied.

See ordering information for specification details.

**Longer lengths available on request.**

6 pin Bayonet fixed plug to MIL-C 26482 or DEF 5325 shell size 10 supplied with PDCR 86X, and mating socket Amphenol type 6G2B-16F10-65 supplied as standard.

**Pressure Connection**
- G/1/8 60° Internal cone,
- G/1/8, N.P.T. Flange
- G/1/8 40° Internal cone
- G/1/8 "N.U.F. as MS.3356-4
- M14 x 1.5 Ermeto
- M14 x 1.5 DIN 3863-8
- Flush fitting
- Depth cone

Others available on request.

Continuing development sometimes necessitates specification changes without notice.
PDCR 800 SERIES: Specification Options

The following summarises the possibilities and for further details and ordering information please contact our Sales Office.

1. Parameter Selection
The PDCR 800 series transducer is calibrated to the nominal full range pressure, and the temperature effects of zero and span are monitored at five temperatures between -20°C and +80°C. This information is stored in a computer and enables us, where it is important, to optimise the performance parameters to suite specific applications. Selection can either be for improved performance in accuracy or temperature drift from standard transducers or to optimise certain parameters by using the transducers in the overrange condition.

2. Improved Accuracy
The standard linearity and hysteresis is ±0.1% B.S.L., but this can be improved to ±0.06% B.S.L., or even better by selection. In some cases this may result in a reduction of the full scale output.

3. Higher Overload Pressure
The lowest overload pressure for standard devices is 400% but this can be increased up to 1000% where necessary. This will reduce the full scale output and increase the zero drift with temperature unless this is maintained by selection.

4. Higher Output
All cores can be overranged by three times nominal full scale, giving outputs of up to 300mV for most ranges. This will improve the zero stability, reduce the overload, and the linearity will be slightly degraded.

5. Excitation Voltage
The transducers can be operated from any d.c. excitation up to 12 Volts maximum. The output is proportional to excitation, but the exact offset and span should be measured at the desired excitation.

6. Improved Temperature Effects
Improved thermal error bands can be selected from the data base. e.g. ±0.3% 0°C to 50°C ±1% -20°C to +80°C Other error bands over different temperature ranges can also be selected.

7. Improved Zero Stability
Thermal zero shift and long term zero stability are improved proportionally with overload.

8. Long Term Stability
The standard PDCR 800 series offers typically 0.2mV per year stability at 10 Volt operation, but this can be improved considerably by operating in the overrange condition at a reduced supply voltage.

9. Thermal Hysteresis
The calibration of a standard transducer at room temperature will repeat within 0.2mV after cycling through the full temperature range.

10. Rationalisation
The transducers can be selected such that both the zero offset and the full scale output are matched to better than 1mV where interchangeability is important.

11. Extended Temperature Range
Transducers are available which will operate between -54°C and +125°C. Please refer to PDCR 82X product note.

12. Rcal
This facility is available by connecting an external resistor across the appropriate connection. The thermal coefficient of this Rcal signal is typically 0.005% F.S./°C.

13. Calibration Print Out
Available on request relating to selected parameters above.

Examples of alternative specifications based upon a standard 10 bar core transducer

<table>
<thead>
<tr>
<th>Operating pressure range bar</th>
<th>Overload x F.S.</th>
<th>Accuracy B.S.L % F.S.</th>
<th>Output with 10 Volt excitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>x6</td>
<td>±0.06%</td>
<td>70mV</td>
</tr>
<tr>
<td>10</td>
<td>x4(40 bar)</td>
<td>±0.1%</td>
<td>100mV</td>
</tr>
<tr>
<td>20</td>
<td>x2</td>
<td>±0.15%</td>
<td>200mV</td>
</tr>
<tr>
<td>30</td>
<td>x1.3</td>
<td>±0.2%</td>
<td>300mV</td>
</tr>
<tr>
<td>10</td>
<td>x4(40 bar)</td>
<td>±0.06%</td>
<td>100mV</td>
</tr>
</tbody>
</table>

The above examples illustrate the various specification performances when using the standard 10 bar core. e.g. used at 20 bar continuously, the overload is x2, accuracy is ±0.15% B.S.L. and output 200mV. The above example can be selected if ±0.06% is required with 100mV output for ranges up to 20 bar.
INSTALLED DRAWINGS

Dimensions: mm

PDCR 80X

Electrical Connection
- Test socket PDCR 80X
- b: 1 Output negative
- c: 2 Supply negative
- d: 3 Supply positive
- e: 4 Output positive
- f: 5 Fails

PDCR 81X

Electrical Connection
- 4 Core shielded/vented cable
- Red: Supply positive
- Blue: Supply negative
- Yellow: Output positive
- Green: Output negative
- Screen: N/C to transducer body

PDCR 82X

Pressure Connection
- Illustrated front and depth cone fixed as standard.
- The incorporates a hydraulic damper to protect the device from high pressure pulses caused by underwriter impact.

PDCR 83X

Electrical Connection
- Pin A: Supply positive
- Pin B: Output positive
- Pin C: Output negative
- Pin D: Supply negative
- Pin E: Fail

PDCR 86X

Electrical Connection
- 6 Core shielded/vented cable
- Red: Supply positive
- White: Supply negative
- Yellow: Output positive
- Blue: Output negative
- Orange: Fail
- Screen: To transducer body

Any other cores not connected.

e.g. PDCR 81X

with flush fitting pressure connection

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Agent:

PDCR 800 SERIES

PDCR 800 series - 12/97
1. Prepare the samples to be used at least 24 hours before the test. Leave them in the temperature controlled laboratory to condition.

2. The geomembrane should be cut to the size of the bottom box and the geotextile to the size of the top box. Use the templates provided.

3. On day of test, switch on the shear box and computer at least 30 minutes before the test to allow the machines to warm up.

4. Check that the shear box is at the required rate of shear e.g. 1mm/min or 3mm/min

5. Tare the shear box by shearing it forward at the required rate of shear and zeroing at around 25mm horizontal displacement. Retract the box after about 50mm displacement. Shear the box forward again up to the maximum displacement and keep a record of the new tare values. Retract the box at the rate of shear to be used and record these values.

6. Insert the nylon block spacer into the bottom box, making sure the sand papered side of the top most wooden spacer, is face up.

7. Clamp the geomembrane onto the leading edge of the lower box. Wipe the shear area dry with a clean paper towel to remove any moisture and/or dust.

8. Clamp the geotextile onto the upper box, taking care not to stretch it.

9. Lower the upper box into the grooves so that it is just touching the supporting screws. Extend the geotextile neatly to prevent any creasing.

10. While in the grooves, push the top box towards the leading edge - this is to prevent movement due to slippage during shear

11. Check the gap between the top and bottom boxes by raising the screws using an Allen key, by 1mm.

12. Clamp the top box using the four large screws
13. Insert the wooden spacers into the top box making sure that the sand papered side of the bottom block is in contact with the geotextile.

14. Carefully place weights (up to 95 kilograms) on top of the wooden spacers.

15. Check that the long screws that are used to fix the pneumatic bag onto the upper box, are not leaning against the outer frame of the shear box.

16. IS RAPID TRAVEL SWITCHED OFF?

17. Set the computer programme SB6A.bas to run as follows:

   C:\ cd shear box
   C:\ shear box: qbasic
   Open file SB6A.bas
   Alt R - Start Programme
   File Name:
   Number of Intervals Required
   E.g. 120 readings each taken every 10 seconds

18. Start the test by flicking the red switch to 'shear' position.

19. Press any key on the computer to get the programme started.

20. Record the maximum shear stress and its displacement during the test.

21. Allow the test to achieve a minimum shearing distance of 90 mm before it stops.

22. Save file onto hard drive and transfer to floppy disk for further analysis at workstation.

Investigating the stability of geosynthetic landfill capping systems
B.2 Test Procedure for the New Shear Box (2000)

1. Cut the geosynthetic samples to the shape of the outer dimensions of the top and bottom boxes and punch suitably sized holes. For a geomembrane/geotextile interface, the dimensions are bottom box (492 mm x 354 mm) and top box (440 mm x 373 mm) respectively, or use the templates provided.

2. Allow the samples to condition in the temperature-controlled laboratory (20°C) for at least 24 hours.

3. Switch on electric supply to shear box and computer and leave them to stand for at least 20 minutes before commencing test.

4. Lift the top box from the slots and rest it on the outer frame.

5. Retract the lower box to the ‘home’ or starting position.

6. Check the contents of the bottom box. There should be at least 3 nylon spacer blocks, and a high friction surface block with the roughened surface face up.

7. Check that the above contents are level with the top of the bottom box. If not, insert a geomembrane sample of relevant thickness and dimensions similar to the spacer blocks, under the top most spacer block (with a high friction upper surface).

8. Check geomembrane and geotextile samples for damage and record.

9. Mark clearly the direction of shearing and side tested on both samples, and ensure they are marked with unique reference codes.

10. Clamp the geomembrane on to the bottom box.

11. Place a piece of plastic or paper cut to the shape of the bottom box on top of the geomembrane to protect the shear surface.

12. Clamp the geotextile on to the top box making sure it does not get in contact with the geomembrane (see 11).

13. Lower the screws/bolts beneath the top box arms in preparation for setting the gap size.
14. Make sure the 4 hand adjustable clamps for the top box are unwound and are positioned facing outwards of the main frame.

15. Pull the geotextile into position, taking care not to stretch it.

16. Remove the protective sheet of paper from the geomembrane.

17. Lower the top box into place in the slots (i.e. it should rest on the bottom box).

18. Raise the 4 screws beneath the top box arms until they are just touching the underside of the top box.

19. Using veneer callipers, measure the distance from the bottom of the flange to the top of the top box.

20. Record the above reading for each of the four arms of the top box.

21. Raise the top box by 1mm at each of the 4 arms to set the gap between the top and bottom boxes. Use the nuts provided to lock the screws in place.

22. Check and record the gap size. NB: For the same geotextile, the gap size will need readjusting between the smooth and textured geomembrane samples.

23. Move the top box towards the control panel and lock it in place using the clamps.

24. Check that the geotextile is not creased in any area of the shear surface and that it fully extends to the end of the top box without stretching.

25. Insert the spacer block with a roughened (high friction) surface into the box first. Make sure the roughened surface is against the geotextile.

26. Add 2 additional nylon spacer blocks into the top box. Place the steel loading platen with loading ball seat on top of the nylon spacers.

27. Screw the four ‘all threaded’ long rods in place in the upper edge of the top box. In the interest of safety, cover the ends of these rods with the corks provided.

28. Lower a nut onto each of the rods.

29. Insert the steel ball into the loading ball seat.
30. Lower the low loading cylinder with bottom plate, vertical displacement and pressure transducers, to rest on the nuts on the rods.

31. Check that the steel ball is located into the loading piston on the underside of the low loading system. Adjust the above nuts if required.

32. Check that the low load system is level on all sides by using the small level.

33. Secure the low loading system in place using four nuts screwed down onto the top plate. Check the set up is level.

34. Check that the long flat head socket screws on the bottom box (usually used to clamp the high load system) are not leaning against the outer frame but inwards.

35. Check the readings on the meters and tare them by flicking upwards the small switches below each of the meters.

36. Channel 1 = Shear Force (N) for E354 transducer, Channel 2 = Shear displacement (mm), Channel 3 = Normal Stress (kPa), Channel 4 = vertical displacement (mm) of top loading platen.

37. Plug the pressure line into the high-pressure compressed air line.

38. Open the valve of the pressure line and apply the required normal stress by adjusting the variable valve control on top of the top plate of the loading system by following the table. (refer to calibration certificate for further information)

| Normal Required (kPa) | Stress Transducer reading (kPa) |
|-----------------------|---------------------|----------------|
| 10                    | 67                  |
| 20                    | 134                 |
| 30                    | 199                 |
| 40                    | 269                 |
| 50                    | 329                 |

39. Apply the required normal stress for 10 minutes before shearing.

Investigating the stability of geosynthetic landfill capping systems
40. From the ‘Start Menu’ of the computer, check the fold up menu for the MS-Dos programme ‘SAS’. The icon is located towards the top. This file is on the c-drive in subdirectory ‘shear box’.

41. Double click to open the SAS programme and carefully read the instructions. They are briefly listed below:

42. Press return key x2
   F1 for test setup
   Type file name – return
   Type test number – return
   Select time record (usually HH:MM:SS) – return
   Enable all 4 channels if all are in use. F2 to enable or disable
   Set interval for readings e.g. 120 readings every 10 seconds
   60 readings every 20 seconds
   10 readings every 30 seconds

   F5 to accept
   F6 to indicator set up (the units indicated are not SI but the values generated at the meters are SI because the shearbox meters are SI Calibrated. The E-212/354 load cell (extra sensitive) gives readings in Newtons while the E-214/439 (general purpose) is in kN.

   F5 to accept

43. On the shear box display screen, check that the motor position is ‘home’ usually 0.00mm.

44. F5 – back to function keys
   F1 to set required distance of travel
   Input e.g. 95 mm – return
   Input rate of displacement 3 mm/min - return

45. Check the readings on the display meters to make sure the right normal stress is displayed. Adjust the control valve on pressure line if required and observe recorded pressure until stable at required value.

46. Shift F1 on computer to run the data logger. Take care NOT TO DELETE previous file. Answer the ensuing questions as follows:

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*Investigating the stability of geosynthetic landfill capping systems*
Start test one – Y, Clear graphic plot – Y. Otherwise test will not start running.

47. Press Start Key on shear box control panel as soon as No.44 is done, to start shearing.

48. Check the normal stress during the test and adjust if required.

49. The shear box will stop at the designated distance of travel. Shift F1 to stop the data logging on the computer. DO NOT DELETE FILE!

50. At the end of the test check data is saved to designated file. Back up this file to disk.

51. Dismantle low loading system, remove spacer blocks from upper box and unclamp the top box.

52. Raise the top box and carefully separate geotextile from geomembrane by inserting a piece of card or plastic.

53. Remove the geotextile and geomembrane samples and inspect for damage (e.g. fibres removed from geotextile, scratches on geomembrane).

54. Return the top box to the start position.

55. Repeat above procedures from No. 6
B.3: Geosynthetics Testing Procedure for the Joint UK-German Collaboration Project (Fixed Top Box)

Normal Stresses: 10, 25, 50, 100 & 200 kPa.

Interfaces:
- Geomembrane/Geotextile/Sand (2 cm)
- Geotextile/Sand (5 cm)

Gap: 1.0 mm

Rate of Shear: 1 mm/min

1. Cut the geosynthetic samples to the shape of the outer dimensions of the top and bottom boxes and punch suitably sized holes. For a geomembrane/geotextile interface, the dimensions are bottom box (492 mm x 354 mm) and top box (440 mm x 373 mm) respectively, or use the templates provided.

2. Allow the samples to condition in the temperature-controlled laboratory (20°C) for at least 24 hours.

3. Switch on electric supply to shear box and computer and leave them to stand for at least 20 minutes before commencing test.

4. Lift the top box from the slots and rest it on the outer frame.

5. Retract the lower box to the 'home' or starting position.

6. Check the contents of the bottom box. There should be at least 3 nylon spacer blocks, and a high surface friction block with the roughened surface face up.

7. Check that the above contents are level with the top of the bottom box. If not, insert a geomembrane sample of relevant thickness and dimensions similar to the spacer blocks, under the top most spacer block (with a high friction upper surface).

8. Check geomembrane and geotextile samples for damage and record.

9. Mark clearly the direction of shearing and side tested on both samples, and ensure they are marked with unique reference codes.

10. Clamp the geomembrane on to the bottom box.
11. Place a piece of plastic or paper cut to the shape of the bottom box on top of the geomembrane to protect the shear surface.

12. Clamp the geotextile on to the top box making sure it does not get in contact with the geomembrane.

13. Lower the screws/bolts beneath the top box arms in preparation for setting the gap size.

14. Make sure the 4 hand adjustable clamps for the top box are unwound and are positioned facing outwards of the main frame.

15. Pull the geotextile into position, taking care not to stretch it.

16. Remove the protective sheet of paper/plastic from the geomembrane.

17. Lower the top box into place in the slots (i.e. it should rest on the bottom box).

18. Raise the 4 screws beneath the top box arms until they are just touching the underside of the top box.

19. Using veneer callipers, measure the distance from the bottom of the flange to the top of the top box.

20. Record the above reading for each of the four arms of the top box.

21. Check that the geotextile is not creased in any area of the shear surface and that it fully extends to the end of the top box without stretching.

22. Pour sand into the top box to a density of 1.8 g/cm³; an equivalent of 2cm depth.

23. For the geotextile/sand interface, make sure that the geotextile is well clamped to the substrate in the bottom box using a high friction surface.

24. Raise the top box by 1mm at each of the 4 arms to set the gap between the top and bottom boxes. Use the nuts provided to lock the screws in place.

25. Move the top box towards the control panel and lock it place using the clamps.

---

*Investigating the stability of geosynthetic landfill capping systems*
26. Insert the spacer block with a roughened (high surface friction) into the box first. Make sure the roughened surface is against the sand. This is so that the reacting shear load is not only acting on the side walls of the shear box but also on the top of the sand body. The distribution of shear forces is estimated to be a little bit more uniform.

27. Add 2 additional nylon spacer blocks into the top box. Place the steel loading platen with loading ball seat on top of the nylon spacers.

28. Screw the four 'all threaded' long rods in place in the upper edge of the top box. In the interest of safety, cover the ends of these rods with the corks provided.

29. Lower a nut onto each of the rods.

30. Insert the steel ball into the loading ball seat.

31. Lower the low loading cylinder with bottom plate, vertical displacement and pressure transducers, to rest on the nuts on the rods.

32. Check that the steel ball is located into the loading piston on the underside of the low loading system. Adjust the above nuts if required.

33. Check that the low load system is level on all sides by using the small level.

34. Secure the low loading system in place using four nuts screwed down onto the top plate. Check the set up is level.

35. Check that the long flat head socket screws on the bottom box (usually used to clamp the high load system) are not leaning against the outer frame but inwards.

36. Check the readings on the meters and tare them by flicking upwards the small switches below the shear stress and horizontal displacement meters.

37. Channel 1 = Shear Force (N) for E354 transducer, Channel 2 = Shear displacement (mm), Channel 3 = Normal Stress (kPa), Channel 4 = vertical displacement (mm) of top loading platen.

38. Plug the pressure line into the high-pressure compressed air line.

39. Open the valve of the pressure line and apply the required normal stress by adjusting the variable valve control on top of the top plate of the loading system by following the table. (refer to calibration certificate for further information)
40. For normal stresses higher than 50 kPa, the pneumatic bag and the general purpose load cell will be required. Make sure the top box is filled up to the brim with spacer blocks. Otherwise, the membrane might expand too much, leading to a reduction in the contact area between the membrane and the loading plate. The normal stress acting on the loading plate will then be lower than the input value.

41. Allow the normal stress to stand for 10 minutes before shearing.

42. From the 'Start Menu' of the computer, check the fold up menu for the MS-Dos programme 'SAS'. The icon is located towards the top. This file is on the c-drive in subdirectory 'shearbox'.

43. Double click to open the SAS programme and carefully read the instructions. They are briefly listed below:

Press return key x2

F1 for test setup
Type file name – return
Type test number – return
Select time record (usually HH:MM:SS) – return
Enable all 4 channels if all are in use. F2 to enable or disable
Set interval for readings e.g. 120 readings every 10 seconds
   60 readings every 20 seconds
   10 readings every 30 seconds

F5 to accept

F6 to indicator set up (the units indicated are not SI but the values generated at the meters are SI because the shearbox meters are SI Calibrated. **** The E-212/354 load cell (extra sensitive) gives readings in Newtons while the E-214/439 (general purpose) is in kN.

F5 to accept
44. On the shear box display screen, check that the motor position is 'home' usually 0.00mm.

F5 – back to function keys
F1 to set required distance of travel
Input e.g. 95 mm – return
Input rate of displacement 1mm/min - return

45. Check the readings on the display meters to make sure the right normal stress is displayed. Adjust the control valve on pressure line if required and observe recorded pressure until stable at required value.

46. Shift F1 on computer to run the data logger. Take care NOT TO DELETE previous file. Answer the ensuing questions as follows:

Start test one – Y, Clear graphic plot – Y. Otherwise test will not start running.

47. Press Start Key on shear box control panel as soon as 44 is done, to start shearing.

48. Check the normal stress during the test and adjust if required.

49. The shear box will stop at the designated distance of travel. Shift F1 to stop the data logging on the computer. DO NOT DELETE FILE!

50. At the end of the test check data is saved to designated file. Back up this file to disk.

51. Dismantle low loading system, remove spacer blocks from upper box and unclamp the top box.

52. Raise the top box and carefully separate geotextile from geomembrane by inserting a piece of card or plastic.

53. Remove the geotextile and geomembrane samples and inspect for damage (eg fibres removed from geotextile, scratches on geomembrane).

54. Return the top box to the start position.

55. Repeat above procedures from No. 6
Appendix C
List of Papers Published

1 Page


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