Soil-geosynthetic interaction: Interface behaviour

Dixon, N.
Department of Civil and Building Engineering, Loughborough University, Leicestershire, LE11 3TU, UK (N.Dixon@lboro.ac.uk)

Keywords: Interface shear strength, direct shear apparatus, specification, repeatability, characteristic values

ABSTRACT: Knowledge of soil vs. geosynthetic and geosynthetic vs. geosynthetic interface shear behaviour is of fundamental importance to designers. This paper considers factors influencing measured behaviour, summarizes methods of measurement including specifications, presents data that quantifies variability, details methods for obtaining characteristic interface shear strength parameters for use in design and defines key questions to be answered by engineers. It is shown that the design of direct shear apparatus is the main reason for observed large variability of measured interface strengths from inter-laboratory comparison testing programs. The need to carry out repeat tests at each normal stress is established. The use of global databases of measured interface strengths to inform selection of strength parameters is discouraged. A recommendation is given to use the results of repeatability testing programs to support calculation of characteristic interface strength parameters. Using the example of landfill lining design, guidance is provided on selection of strength parameters in conjunction with relevant factors of safety, consequences of failure, selection of the controlling interface and minimizing interface displacements.

1 INTRODUCTION

1.1 *Scope of paper*

This paper considers the interaction between soil and geosynthetics for planar continuous sheets of material, which include geotextiles, geomembranes, geosynthetic clay liners (GCLs) and geocomposites. Specifically this type of interaction is characterized by a planar interface between the soil and the geosynthetic that can be investigated by carrying out direct shear and tilting table type element tests. Geogrid materials are discontinuous sheets allowing interlock with soils, and between soils in the openings, and this interaction is primarily characterized using pull out element tests. This paper includes geosynthetic vs. geosynthetic interaction but excludes geogrid materials. The paper uses landfill lining structures to demonstrate applications and aspects of interface behaviour but is applicable to other structures that include geosynthetics.

Geosynthetics are typically employed in conjunction with soil and other types of geosynthetics and have a number of roles such as reinforcement, separation, drainage and barrier. They are used in a range of diverse applications such as landfill barriers and construction of steep soil slopes. The use of geosynthetics in a structure introduces potential planes of weakness. For example, the stability of geosynthetic landfill lining systems are often controlled by the shear strengths mobilised at the interfaces between various soils and geosynthetics and indeed within the geosynthetics themselves. As an example, Figure 1 shows a schematic of a landfill containment system that includes a wide range of materials. The soil/geosynthetic and geosynthetic/geosynthetic interfaces control slope stability of both the capping system and the side slopes during construction and following waste placement.

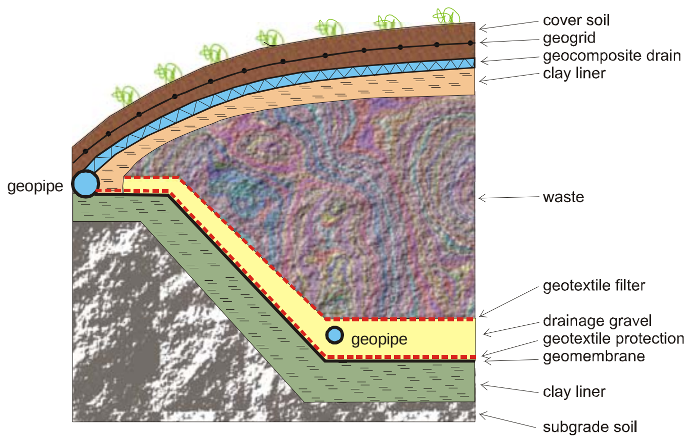

Figure 1. Schematic of a landfill containment system showing the use of geosynthetics on the capping and barrier side slopes (after Koerner 1998)

Assessment of stability requires detailed knowledge of the stress vs. strain behaviour of interfaces, as post peak shear strengths are often mobilized resulting from the strain incompatibility of soils, geosynthetics and waste materials. In addition to designing for stability (ultimate limit state), the designer must also consider long-term integrity of the system (serviceability limit state) and hence deformations in the soil/geosynthetic system must be considered.

Guidance is provided in national and international standards on the methods that can be used to measure interface shear strength but the designer must use engineering judgement to interpret the test results and evaluate values to be use in design. Many interfaces involving geosynthetic are strain softening, i.e. the shear strength reduces with displacements beyond peak. Displacements can be induced at geosynthetic interfaces by a number of mechanisms. For example, in a landfill the placement of soils onto the geosynthetic and the large settlement that the waste body undergoes can generate relative displacement between layers. Since the amount of shear strength mobilised at these interfaces is dependent upon this relative displacement, an estimate of the displacement is needed to provide values for mobilised interface shear strengths. Designers therefore must select appropriate shear strength parameters for use in design, for example peak values, residual values or indeed somewhere in between, and they should also consider variability of measured interface behaviour.

1.2 *Key questions*

This paper summarises current knowledge and practice that can be used to help engineers address the following key questions.

- What type of tests should be carried out to characterize interface interaction?
  - Performance lab tests, index tests or use of data-bases?
  - Which type of equipment?
  - What specification should be used?
- How many tests should be carried out?
- Which design parameters should be used?
  - Peak, residual or somewhere in between?
- What are the implications of variability in measured values on design?

1.3 *Common interface behaviour*

Figure 2 shows shear stress vs. displacement behaviour obtained in laboratory tests of common interfaces sheared at the same normal stress (after Marr 2001). A number of the interfaces demonstrate significant post-peak reductions in strength to large displacement, or residual, values. It should be noted that obtaining true residual shear strengths is problematic, as discussed in Section 2.2, and therefore in the large majority of cases it is large displacement shear strength that is measured and reported.

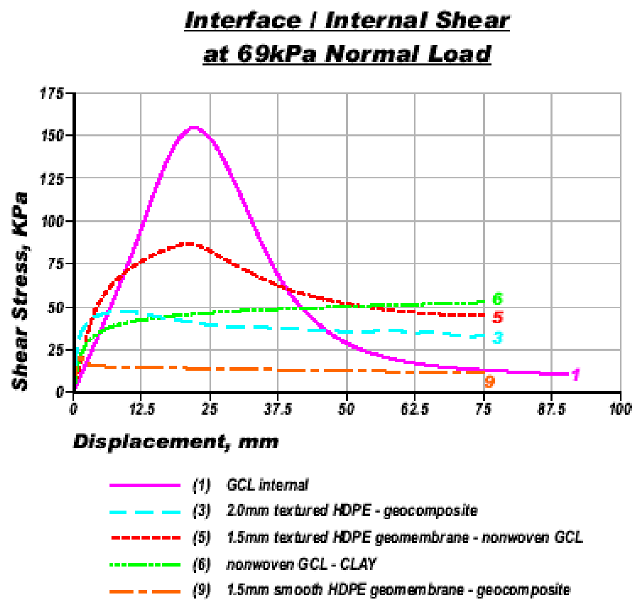

Figure 2. Direct shear tests on common geosynthetic/soil interfaces and GCL internal strength (after Marr 2001)

It is a key requirement of any interface testing that full shear stress vs. shear displacement behaviour is measured. This is to enable peak and large displacement strengths to be defined and also to quantify the shear displacements required to mobilise the peak and residual conditions. This information is required when considering whether site conditions are likely to result in mobilisation of post-peak strengths (i.e. relative displacement at the interface sufficient to mobilize post-peak values).

1.4 *Post peak reductions in shear strength*

Interface shear strength may be a combination of sliding, rolling and interlocking of soil particles and geosynthetic surfaces. Reduction in strength from peak to a residual value (i.e. strain softening behaviour) is due to physical changes in the soil and geosynthetic materials forming an interface, and results from relative displacement between the materials. The term *wear* can be used to describe the processes that lead to reduced geosynthetic/geosynthetic interface strength. This is a useful concept and describes the majority of the processes that occur at geosynthetic interfaces. However, it is perhaps less appropriate for describing changes that occur in soils involved in interface shearing (e.g. changes in moisture content, density and particle packing). A summary of the common mechanisms that can result in strain softening behaviour is provided below.

Randomly orientated fibres (e.g. non-woven geotextiles) can be realigned in the direction of shearing due to combing by adjacent rough materials (e.g. textured geomembrane). This can result in tensile failure and pull out of fibres from the matrix.

Adhesion resulting from the hook and loop effect between two geosynthetics is destroyed and hence at residual conditions shear strength is purely frictional.

Plastic materials commonly used in the manufacture of geosynthetics (e.g. HDPE geomembranes) are readily polished by relative displacement with other geosynthetics (e.g. geotextiles). The surface of the materials are made smoother by removal or reduction in the height of surface asperities. Lee et al. (1996) have reported measurements of surface roughness made following interface testing that confirm this polishing mechanism results from the shearing process. Conversely, where a hard material (e.g. quartz sand) is sheared against a softer geosynthetic (e.g. a HDPE geomembrane) roughening of the geosynthetic surface can occur through the formation of scratches and even ploughing of the geosynthetic by individual soil grains. Roughening of the interface leads to increasing shear strength with displacement (i.e. strain hardening).

Some textured geomembranes are formed through the impingement process (i.e. particles of polymer are heat bonded to the surface of the geomembrane to form the roughened surface). It has been reported (e.g. Jones and Dixon 1998 among others) that texturing can be removed during shearing at elevated normal stresses. The value of normal stress that results in significant texture removal is dependent on the properties of the adjacent material and the strength of the bond between texturing and geomembrane. Removal of texturing results in a significant reduction in interface shear strength. This potential mechanism should be investigated by conducting tests at the normal stresses to be experienced on site.

Reduction in interface strength between geomembranes and GCL during shearing can be caused by extrusion of low strength bentonite onto the interface. The potential area of the interface with a reduced strength is influenced by: hydration of the GCL under a low confining pressure; the type of confining geotextile (i.e. the openness of the weave) and the type and frequency of joints between panels.

Interfaces involving soils can have reduced strength with displacement as a result of the soil dilating during formation of the shear plane. This increase in soil volume in the shear zone leads to a reduced density and hence reduced shear strength (e.g. a dense sand shearing against a textured geomembrane). Dilatency can also occur in stiff fine grained soils sheared at an interface in a drained state (i.e. resulting in increased moisture content). Realignment of clay particles parallel to the interface can results in a further significant reduction in shear strength. The magnitude of the drop in strength due to particle realignment is a function of the clay content and clay mineralogy.

Loss of internal strength (e.g. in GCL and geocomposites) due to failure of needle punched fibres, stitching and glue connections can result in a significant loss of strength.

For many interfaces a combination of the above mechanisms contribute to the reduction in strength from peak to residual. All the mechanisms described above require relative shear displacement at the interface between the two materials. Changes in interface properties that occur with no relative displacement may result in changes to both the peak and residual strengths (e.g. changes in soil density and moisture content, and extrusion of bentonite).

2 TEST METHODS

Interface shear behaviour is a function of a large number of factors that can be summarized as (ASTM D 5321, 2008):

- Applied normal stress;
- Geosynthetic material characteristics;
- Soil gradation;
- Soil plasticity;
- Density;
- Moisture content;
- Size of sample;
- Drainage conditions;
- Displacement rate;
- Magnitude of displacement; and
- Other parameters!

The following sections consider a number of these key factors in detail.

2.1 *Apparatus*

It is common practice to measure interface shear strength in a direct shear apparatus (DSA), as used in soil mechanics, but with a larger shear plane. Most of the DSA currently used for shear testing geosynthetics have a top box or frame with a test area of about 300mm x 300mm. The lower part of the DSA has an equal test area or a box which is longer than the upper in the direction of shear movement so that the shear area is kept constant during the test (e.g. bottom box 300mm x 400mm). Smaller shear devices (e.g. 100mm x 100mm) can be used if it is demonstrated that for the interface tested the measured behaviour is the same as for the larger box. Smaller devices have the benefit of shorter drainage paths, which might be an advantage when testing interfaces involving fine grained soils, but measurement of large displacement shear strength is often problematic due to the limited maximum displacement available. All direct shear apparatus have limited displacements and it has been shown (Jones and Dixon 2000) that even displacements of 100mm may not mobilise the true residual interface shear strengths.

Typically for each normal stress, the shear stress increases from the origin with increasing displacement until a peak value is achieved. For some interfaces subsequent displacement results in a reduction in shear stress to a constant or residual value, e.g. Figure 3a. This example of shear stress vs. displacement behaviour of a textured geomembrane/geotextile interface is from testing carried out using a 300 mm direct shear apparatus as described by Jones & Dixon (1998).

If the peak and residual strengths are plotted against the relevant normal stresses, the resulting failure envelope can be defined (Figure 3b). A linear Coulomb-type failure envelope is usually assumed, although a curved envelope is sometimes obtained, which defines the interface shear strength in terms of the friction angle (δ) and apparent adhesion intercept (α). Shear strength (τ) is related to the effective normal stress σ_n' by :

$$\tau = \alpha' + \sigma_n'.\tan\delta' \qquad (1)$$

It should be noted that these parameters only define the failure envelope for the range of normal stresses tested and that extrapolation of both friction angle and adhesion intercept outside the range may not be representative. Parameters are defined for both the peak (denoted by subscript p) and residual (denoted by subscript r) conditions (Figure 3b).

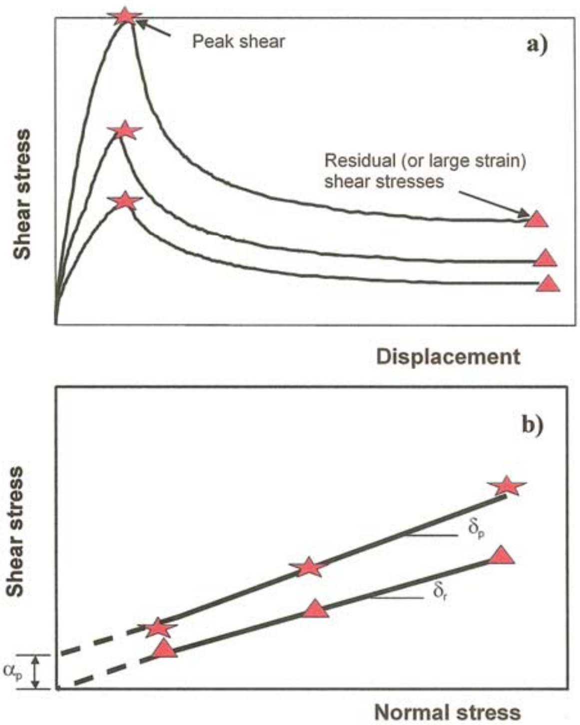

Figure 3. Direct shear test on a strain softening interface a) shear stress vs. displacement, b) shear stress vs. normal stress plot showing derived peak and residual shear strength parameters

Ring shear apparatus (RSA) can be used to investigate true residual strengths since the apparatus can produce unlimited displacements (Starke and Poeppel 1994, Jones and Dixon 2000). However, only fine soil/geosynthetic and geosynthetic/geosynthetic interfaces can be investigated using this apparatus.

The third main method for measuring interface strength is using a tilting table, which has been used predominantly in Europe. There is currently no consensus on the size of apparatus required to provide performance results and its use is limited to low normal stresses (e.g. Gourc et al. 2006, Pitanga et al. 2009). The tilting table may be more accurate in determining the behaviour of geosynthetic interfaces at low confining stress and assessing creep behaviour, but it cannot be used to obtain residual strengths.

2.2 *Measurement of large displacement shear strength*

RSA have been used to investigate non-woven vs. textured geomembrane interface large displacement behaviour (Jones and Dixon 2000). Results from the RSA are considered to be lower bound values because of the very large displacements achieved. In addition, the inability to include effects of cover soils that transfer normal stress to the interface can also lead to an underestimate of interface strength. Once the RSA has rotated through 360° the geotextile fibres have undergone realignment due to combing by the geomembrane asperities. The observed subsequent reduction in shear strength has been attributed to polishing and fibre loss effects. This mode of shearing is an extreme case and possibly does not model the field condition of a linear shear direction, where the random nature of most geomembrane texturing would result in continuing fibre tangle even after large displacements. A comparison of DSA and RSA methods for obtaining residual interface strengths has indicated significant differences in the measured values (Jones and Dixon 2000). These differences can partially be attributed to aspects of test set up such as fixity of the geosynthetics, presence of cover layers, and particle shape and grading of cover soils and the direction of shearing as discussed above (i.e. rotational/linear).

There is a degree of idealisation inherent in analysis of DSA results. In the large shear boxes commonly used a maximum displacement of 100mm is obtainable without a change in shear area. However, this does not mean that there has been relative displacement of 100mm between the materials over the whole area of the interface. As the test progresses the material on the top moves over a region of ‘virgin’ material on the bottom (i.e. material initially outside the footprint of the top box) and hence the bottom material experiences displacements ranging from 100mm to a few mm. Therefore, it is impossible for the DSA to produce true residual shear strength conditions over the entire shear area and the DSA results will always be higher than those from the RSA. The degree of difference will depend upon the properties of the materials tested.

Despite the fact that 'true' residual interface shear strengths can only be measured using an RSA, the design engineer must decide whether these values should be used in stability analysis. Problems in reproducing site specific conditions in RSA tests often result in measured values that are over-conservative (i.e. on the low side). In addition, ring shear testing should not be used to measure peak interface shear strengths due to non-uniform strains across the shear surface (Dixon & Jones, 1995). It is proposed that values of 'residual strength' obtained in large DSA tests are acceptable for use in the majority of stability assessments. However, due to the practical problems of shearing an interface sufficient distance to mobilize the 'residual strength', it is often more correct to describe the results from DSA tests as 'large displacement strengths' in order to indicate that true residual strengths may not have been obtained.

2.3 *Index vs. performance testing*

Site specific materials and boundary conditions should be used in tests to simulate field conditions (e.g. direction of shearing in relation to manufacturing process - i.e. roll direction, using site specific cover soils and appropriate moisture conditions). Many authors have demonstrated significant differences in measured interface behaviour that arise from the test boundary conditions (e.g. Jones and Dixon 1998, Gallagher et al. 2003).

Four standards are summarized in this paper to provide guidance on testing procedures using direct shear apparatus to measure geosynthetic interface shear behaviour (i.e. BS 6906:1991, ASTM D5321-08, a German recommendation for landfill design GDA E 3-8 (1997) and European standard BS EN ISO 12957-1). In addition, a significant number of research papers have been published on this topic in the past 25 years. It would appear therefore that there is adequate information and guidance to ensure high quality testing is carried out. However this is not the case. There is growing evidence that tests specified to obtain parameters for design, and those reported in the literature, often lack sufficient control on the key factors affecting the measured values. This is resulting in uncertainty regarding the likely variability of measured shear strengths, and in some instances is leading to the use of un-conservative (i.e. high) interface strengths in design.

2.4 *Design of direct shear apparatus*

Detailed evaluation of the results from inter-laboratory comparison test programs and discussions with the participants indicates that one of the main reasons for the scatter of measured strengths is the different DSA devices used for test performance. Bluemel and Stoewahse (1998) have investigated some effects of DSA device design on test results and this work has been extended by Stoewahse (2001) who used four types of DSA with different top box supports and with different loading and load controlling systems as shown in Figure 4.

The floating top box (designed by Casagrande) is supported only in one point and it is able to rotate around this support (Figure 4a). The design of the top box is well known from soil testing and is in accordance with BS 1377: Part 7 and ASTM D 3080. However, there are only a few devices of this type with sufficiently large test areas to be used for interface friction testing.

The type of DSA mainly used for geosynthetic interface testing is constructed in a manner such that the top box is fixed (Figure 4c). This device was designed specifically to measure interface shear behaviour. The fixed top box was introduced on the premise that a pre-formed shear plane (i.e. the interface) is located between the bottom and top boxes, and hence formation of a shear plane with associated volumetric straining of material in the top box does not occur. However, this assumption is wrong for many combinations of materials that can be tested and hence there are concerns regarding the magnitude and time dependent variation of the vertical stress on the interface during shearing. The vertical stress is usually applied by air or water pressure via a membrane, and hence is known only on the top of the sample. Friction between the test material and internal walls of the top box both during application of normal stress and shearing will alter the actual vertical stress acting in the shear plane by an unknown amount. There are large numbers of these devices in use around the world including in the UK, USA and Germany.

Stoewhase (2001) overcame the problem of unknown vertical stress in the shear plane with the fixed-box-DSA device. The modification allows the average vertical stress acting on the interface to be determined by measuring the vertical support forces to the top box. The pressure applied to the top of the sample is then regulated in order to keep the resulting vertical force on the interface at a constant value. The device is sketched in Figure 4d.

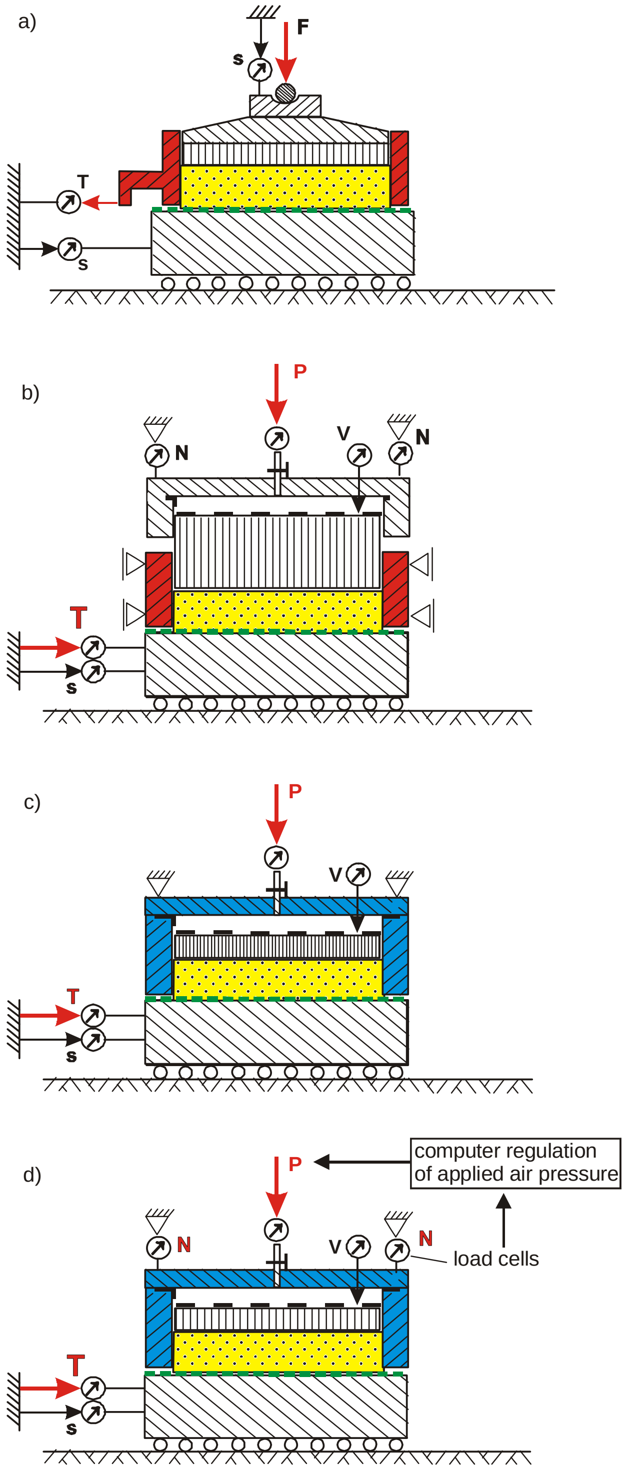

Figure 4. Direct shear apparatus (DSA) with different top box supports: F - applied forces; P - air pressure; N - vertical support force; T - shear force; s – displacement; and V - dilation/compression measurement by volume control (after Stoewahse et al. 2002)

Stoewhase (2001) made another improvement to the DSA by separating the loading system from the upper box and allowing it to move vertically as shown in Figure 4b. The vertically moveable top box together with a control system ensures that the vertical stress applied to the interface remains constant during the testing process. This construction was selected as the standard DSA design incorporated in the German DIN 18 137-3. Stoewahse et al. (2002) present test results (Figure 5) that demonstrate the influence on measured interface shear behaviour of the top box support systems.

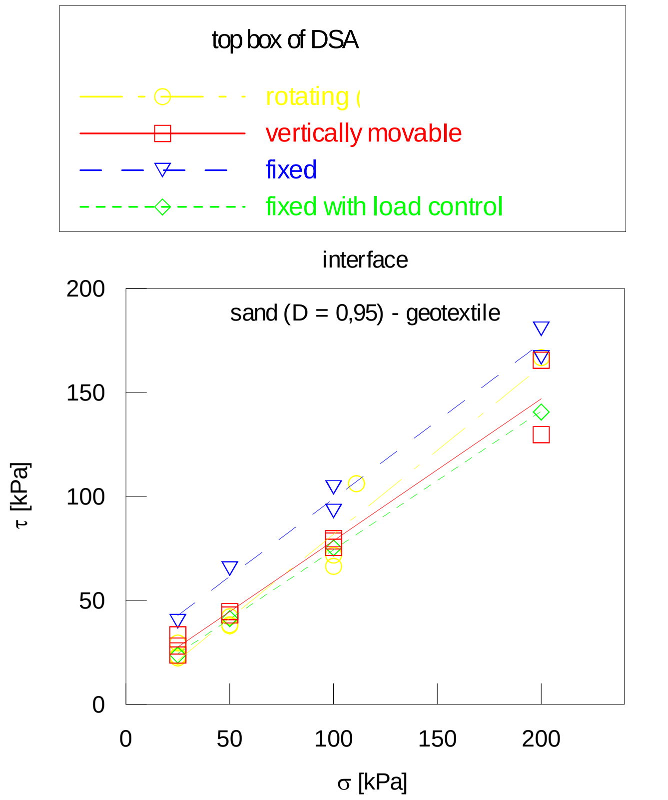

Figure 5. Peak shear stress τ measured with different types of DSA vs. normal stress σ (after Stoewhase et al. 2002)

2.5 *Direct shear apparatus specifications*

The four standards listed in Section 2.3 are available to provide guidance on testing procedures and evaluation of measured data. These standards provide useful guidance for both the specifier, test operator and user of the measured values (i.e. designer). The ASTM gives guidance for performance testing of soil vs. geosynthetic and geosynthetic vs. geosynthetic interfaces. BS6906 Part 8 essentially covers only index tests on these two types of interface, although limited guidance on performance testing is provided in Appendix A. The EN ISO standard is restricted to index tests on standard sand vs. geosynthetic interfaces. BS6906 is 18 years old and therefore does not include recent developments, however the ASTM was updated in 2008 and currently provides the most up to date information. The EN ISO document is of limited use for designers as it only covers index testing. GDA E3-8 (Gartung and Neff, 1998) is specifically devoted to landfill design and gives detailed recommendations for performance testing of a range of interfaces for liner systems and covers, although it is understood that it is not available in English. A summary of the test standards and the key factors controlling measured interface strength is give in Table 1, which is an updated version of the summary presented by Stoewahse et al. (2002).

Table 1. Key elements of DSA test standards

Standard	BS6906:1991	BS EN ISO 12957-1:2005	ASTM D5321.08	GDA E 3-8
Scope	Index tests + some guidance on performance testing	Index tests only	Performance tests	Performance tests
Test Apparatus	DSA 'about 300mm square'.	DSA minimum shear area 300mm square. Grids, dimension of device at least 2 longitudinal and 3 transverse ribs.	DSA minimum shear area 300mm square and >15x d_{85} of soil 5x maximum opening size of geosyn.	DSA minimum shear area 300mm square, for geosynthetics without surface structure and fine grained soil 100 mm.
Specific requirements of DSA	σ_n applied through rigid load plate. Measure vertical deformations. Design of box not specified.	Design should allow for sand dilation, $\sigma_n \pm$ 2% Fluid filled membrane systems to apply σ_n uniformly. Measure vertical movement of loading plate at end of test.	σ_n applied by device that maintains a constant uniform σ_n for duration of test $\pm$ 2% Design should allow for soil deformation during shearing.	Design of DSA not specified Measurement of normal and friction stresses and of vertical movement Calibration measurements recommended to determine the stress acting in the friction plane.
Number of Tests conducted	9 tests in total, σ_n = 50, 100 and 200, kPa (3 tests at each σ_n) Conduct tests in different directions and sides.	4 tests in total, σ_n = 50, 2 x 100 and 150 kPa.	Minimum of 3 σ_n, user defined. Test different directions and sides.	3 tests with 3 different normal stresses and 2 repeating tests with the mean value, which should match the expected normal stress in situ.
Material Conditioning	Sand and geosyn. 20° ± 5°C	Sand and geosyn. 20° ± 2°C Humidity 65% ± 2% if applicable.	Soil and geosyn. 21°C + 2°C	Soil mechanical laboratory conditions
Method of fixing geosynthetics	Clamp or glued to rigid sub-stratum	Fix geosyn. to rigid support to prevent relative displacement between specimen and support (e.g. glue, friction support or clamped outside area).	Clamping outside shear area, use of a textured surface or gluing to rigid sub-stratum. Keep geosyn. Flay during testing.	Recommendations about support and fixation of geosynthetics depending on the individual test case.
Soil Properties	Complying with fraction B (1.18mm to 600 μm) BS 4550 Compacted dry ρ_d = 1.65 $\rightarrow$ 1.7Mg/m^3 Performance tests, compact soil w_{nat} to 92 ± 2% ρ_{dmax}.	Natural siliceous sand, rounded particles (1.6mm to 0.08mm) Compacted w of <2% to ρ_d=1.75Mg/m^3	User defined Take care not to damage geosyn. during placement. Measure ρ and w after test.	Cohesive soils with not more than 95% $\rho_{Proctor}$ 'on the wet side' or as proposed by the landfill designer. Not less than 24 hr preconsdolidation time under normal stress equal to the test. Noncohesive soils compacted to medium density or proposed by designer.
Maximum particle size and Gap size (top/bottom base)	Sand vs. geosyn. (index) not specified. Soil vs. geosyn. (performance) gap is d_{85}/2 or 1mm for fine grained soils. Maximum particle size < 1/8th box depth.	Maximum particle not applicable. Gap size = 0.5mm.	Maximum particle size < 1/6th box depth. Gap large enough to prevent friction between parts of box but small enough to prevent soil entering the gap.	Maximum partice size d_{85}< 1/15th of box length. Gap size is depending on test materials and has to be chosen so that there cannot develop additional normal forces by the frame and secondary friction planes; chosen gap size has to be reported.
Location of materials in DSA	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.	Sand vs. geosyn., rigid sub-stratum in bottom box and sand in top box. Depth of sand layer = 50mm.	Geosyn. vs. geosyn. rigid sub-stratum (i.e. no soil). Soil vs. geosyn., geosyn. supported by rigid sub-stratum. Soil either in top or bottom box. Depth of sand layer not specified.	Geosyn. vs. geosyn. rigid sub-stratum (i.e. normally no soil). Soil vs. geosyn., geosyn. supported by rigid sub-stratum. Soil either in top or bottom box. Depth of soil layer not specified.
Shearing rate	Geosyn. vs. geosyn. and sand vs. geosyn. (index) 2mm/min. Soil vs. geosyn., variable rate depending on drainage.	Sand vs. geosyn. 1mm/min.	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.	Geosyn. vs. geosyn and non cohesive soil vs. geosyn., 0.167 to 1 mm/min. Geotextile vs. cohesive soil 0.167 mm/min. Geosyn. liner vs. cohesive soil 0.005 mm/min.
Derivation of shear strength parameters	Obtain δ_p, δ_r from best fit straight line through all 9 points. Disregard any apparent adhesion (α) values.	Best fit straight lines through all points (peak and residual) to obtain, δ_p, δ_r, α_p and α_r	Correct for reducing shear area if top and bottom boxes are the same size. Failure envelopes defined by best fit straight lines to obtain strength parameters δ_p, δ_r and c_a adhesion intercept.	Tests should be performed independendly by a second institution. Best fit straight lines through all points (peak and residual) to obtain test values of δ_p, δ_r, α_p and α_r. Derivation characteristic values. Disregard any apparent adhesion (α) values for non-cohesive soils and for cohesive soils in special cases.
Specific reporting requirements	All plots and calculations. Describe failure mode. Report φ' of sand.	'For comparison of index test results, all graphs and data have to be submitted to judgement of an engineer.' Description of 'post peak be-haviour observed in each test'.	Test conditions All plots and calculations	Detailed report about the test equipment, procedures and observations during testing, about the measured data and the further evaluation.

None of the guidelines specify the construction of the testing device although detailed specification of the DSA exists in all the standards for direct shear tests on soils. As there are presently many DSA with a fixed top box or similar design in use, it is important that a full description of the testing equipment is provided together with the test results. The investigating laboratory should comment on the key question of how the effective normal stress on the interface is calculated or measured during shearing (Bluemel and Stoewahse, 1998, and Bluemel et al., 2000).

In fixed top box DSA the gap between the top and bottom boxes must be set prior to shearing. Advice from the test standards is both ambiguous and in some cases outdated. The gap size must be so small that no soil particles can migrate out of the box but it must also be large enough so that no constraints are induced. This is nearly impossible to achieve in a fixed box DSA. Bemben and Schulze (1998) demonstrated that the gap size has a significant effect on the measured strength. ASTM D5321 does not specify the dimension of the gap but provides the criteria to be met, and the criteria in BS6906 can lead to significant errors if additional considerations on the materials to be tested are not made. Practical experience indicates that the accuracy with which the gap can be adjusted is not less than ±0.5mm in a 300mm square DSA.

GCLs are not included in this testing summary. Readers are directed to Marr (2001) and McCartney et al. (2004) for discussions of factors controlling testing and behaviour of interfaces involving GCL.

2.6 *Reporting of results*

The evaluation of geosynthetic/soil system failures has shown that a contributing factor is often a lack of communication between the participating parties. It is important to inform the testing engineer about the project details and the site-specific conditions in order to ensure correct testing conditions are applied and thus appropriate results obtained. In addition, the test method and results must be documented in a manner such that design engineers, who might not be knowledgeable about the testing practice, are able to interpret the results and use them in stability calculations in order to produce a rigorous design.

From inter-laboratory comparison test programs (Section 3.1) it has been found that even apparently minor changes in testing conditions can affect the results significantly. Therefore, a detailed test report is necessary and must include information on the following:

- **Description of test device**; including support of the top box, load application, etc.
- **Test set up and boundary conditions**; shearing rate, samples tested dry or submerged, consolidation time, method of fixing the geosynthetics to resist stretching, exact location of interface in relation to top and bottom boxes, gap between top and bottom boxes (if relevant), placement method of soils (e.g. compaction effort and layer thickness) and density and water content before and after the test.
- **Full material descriptions**; *geosynthetics:* manufacturer, mass per area, thickness, polymer, description of the structure, etc.; *soils:* origin, soil mechanical classification, other mechanical parameters.
- **Description of sampling methods employed**; *geosynthetics:* sample preparation, e.g. presoaking; *soils:* any form of pretreatment like crushing of aggregates, drying, adding of water.
- **Test results**; 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 beginning and end of test, shear strength parameters δ and α and the method of derivation (e.g. linear regression).
- **State of the materials after the test**; stretching of the geosynthetics, abrasion of geomembrane textures, orientiation of geotextile fibers, post shearing damage such as tearing of stitch bonding or welding points, development of additional shear zones in geocomposites and also in soils, changes of water content, etc.

This list is necessarily incomplete. With the development of new geosynthetics other aspects might become important. For example, creep aspects are not usually considered in short-term friction tests but can occur under combined compression and shear stress (e.g. geocomposite drains).

2.7 *Summary of key issues controlling measured behaviour*

There are many factors that influence measured geosynthetic interface behaviour. A brief summary of the main factors is provided below:

- Design of the direct shear device - i.e. fixity of top box, method of applying normal stress.
- Test set up (e.g. method of clamping and restraining the geosynthetics, gap size between the top and bottom boxes, dry or submerged conditions, type of material used in the top box to transmit the normal stress to the interface, shearing rate, temperature and normal stress range).
- Material variability (i.e. direction of shearing, number of tests required to obtain representative values).

- Soil mechanics principles (density of soil, maximum particle size, consolidation properties, drained or undrained shearing, value of pore water pressures, volume changes).

While the concept of testing is straight forward there are many issues that must be considered by the person specifying the test. These must also be understood, followed, controlled and reported by the person carrying out the testing. Anecdotal evidence suggests that at present in many instances the specification of testing is inadequate and that site specific ‘performance’ interface testing is considered a luxury rather than a necessity by a significant number of design engineers. Selecting peak or residual strength for use in design in conjunction with an appropriate factor of safety, will not guarantee a safe design unless relevant site specific testing has been carried out and interpreted in accordance with best practice.

3 REPEATABILITY OF MEASURED BEHAVIOUR

Factors influencing measured values of interface shear strength can be categorized as either due to inherent material variability or measurement error. It is seldom possible to separate the relative contribution of the two factors. However, Phoon and Kulhawy (1999) report comparative studies of errors in laboratory strength tests on soil. Statistical analysis of results from a number of test programs indicates that measurement errors for most laboratory strength tests, expressed in terms of coefficient of variation, which is defined as standard deviation/mean, are in the range of 5 to 15%. Inherent material variability results in coefficients of variation also of between 5 and 15%, and the combined influence of measurement error and inherent variability is expressed by coefficient of variation of measured strengths between 7 and 21%.

There are three categories of factors that lead to variability of measured interface shear strength:

- Test apparatus design;
- Operator/test procedure; and
- Variability of both geosynthetic and soil materials.

A number of research studies have been conducted to quantify the likely variability of test results and to identify the key factors that control measured strengths. These studies can be divided into three categories:

- Inter-laboratory;
- Repeatability; and
- Data bases

3.1 *Variability of measured interface strengths: Inter-laboratory*

Inter-laboratory comparison tests were conducted in support of the development of the European geosynthetic test standard (BS EN ISO 12957-1: 2005), in an effort to quantify the likely scatter in measured strengths resulting from the use of different operators and test equipment (Gourc & Lalarakotoson, 1997). Tests were carried out in seven commercial and research laboratories (two each in France, Germany and UK and one in Italy) using geosynthetic materials supplied by the co-ordinator and obtained from one source. The interface shear strengths between a range of geosynthetic materials and standard sand were measured. The study produced a wide range of measured strengths, including measurement of widely varying shear stress vs. displacement behaviour.

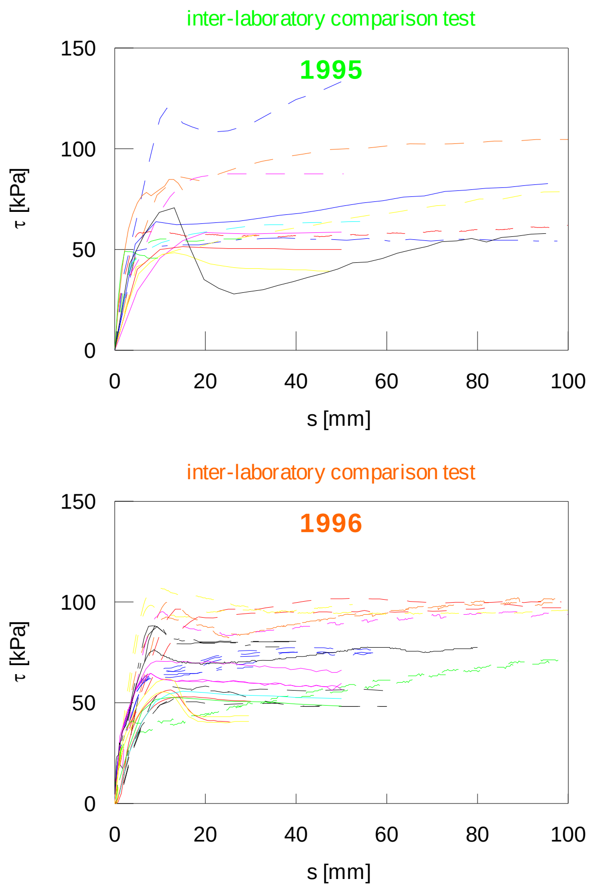

Figure 6. Variability of the shear stress τ vs. displacement s curves on a sand-geotextile interface (each colour and line style indicates one laboratory) (after Stoewahse et al. 2002)

Two similar, and complementary, inter-laboratory comparison test programs were conducted by a working group of the German Society for Geotechnical Engineering in 1995 and 1996, as part of their response to development of the European standard (Bluemel and Stoewahse, 1998). The latter program incorporated a more detailed specification of the testing procedure. These programs, each involving approximately twenty laboratories, produced a range of measured strengths that is similar to the European study. Figures 6 and 7 show test results from the German studies for a non-woven geotextile vs. sand interface. The significant variability of the shear stress vs. displacement curves in Figure 6 is typical. The different laboratories produced a range of peak and large displacement shear strengths, and widely varying stress vs. displacement relationships. Figure 7 shows the distribution of peak failure envelopes obtained by the laboratories. In addition to the large variation of results, of particular concern is that some laboratories produced high, and hence unsafe, shear strengths. Inspection of the data in Figure 6 shows clearly that some of the results are significantly in error (i.e. indicated by the shape of the shear stress vs. displacement curves).

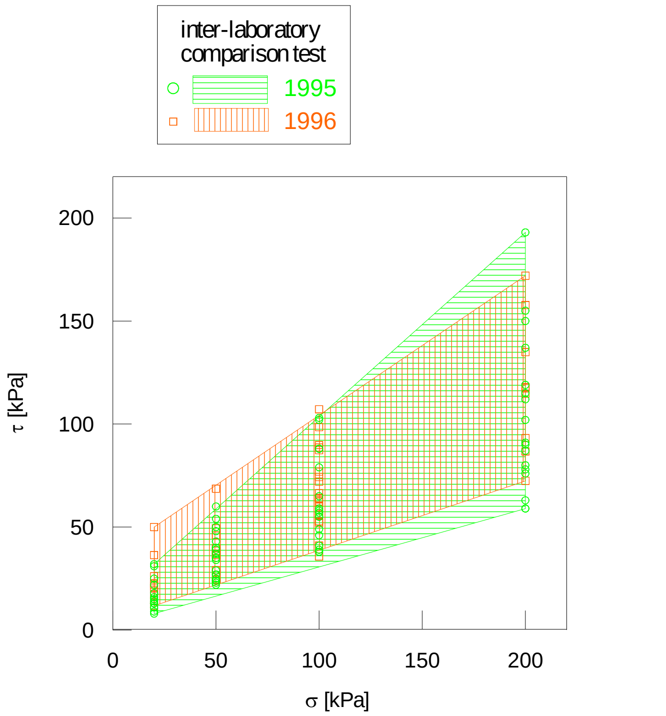

Figure 7. Scattering of data delivered by different laboratories from friction tests on a sand-geotextile interface (peak shear stress τ vs. applied normal stress σ) (after Stoewhase et al. 2002)

An experienced engineer would not use these results and would require repeat tests to be carried out. By removing this spurious data the variability could be reduced significantly. However, it is worth noting that all tests were conducted by laboratories experienced in measuring geosynthetic interface shear strength, and that the laboratories knew their results would be compared with those from a large number of other laboratories (i.e. their competitors). It can only be assumed that those who submitted the spurious test results must have considered them to be correct. This indicates that at the time of the comparison, some laboratories lacked the experience to interpret the results they obtained.

Both the European and German test programs involved the use of a clearly defined common test standard and samples from a common source, but involved different operators and a range of different DSA designs. Unfortunately, rather then provide confidence in our ability to undertake reproducible tests, the results of the inter-laboratory comparison test programs cast doubt on the applicability of aspects of current test procedures.

3.2 *Variability of measured interface strengths: Repeatability*

Repeatability can be improved by using one design of DSA and one operator, although the results may have a consistent error. Test programs have been carried out under these conditions at Hanover University (Bluemel et al., 1996), Loughborough University (Dixon et al., 2000; Sia and Dixon, 2007) and are reported in the literature (e.g. Criley and Saint John, 1997). Scatter of results from these tests "under conditions of repeatability" would be primarily due to variation in the geosynthetic and soil test material. Some results of these studies are shown in Figure 8**,** together with the results of inter-laboratory tests.

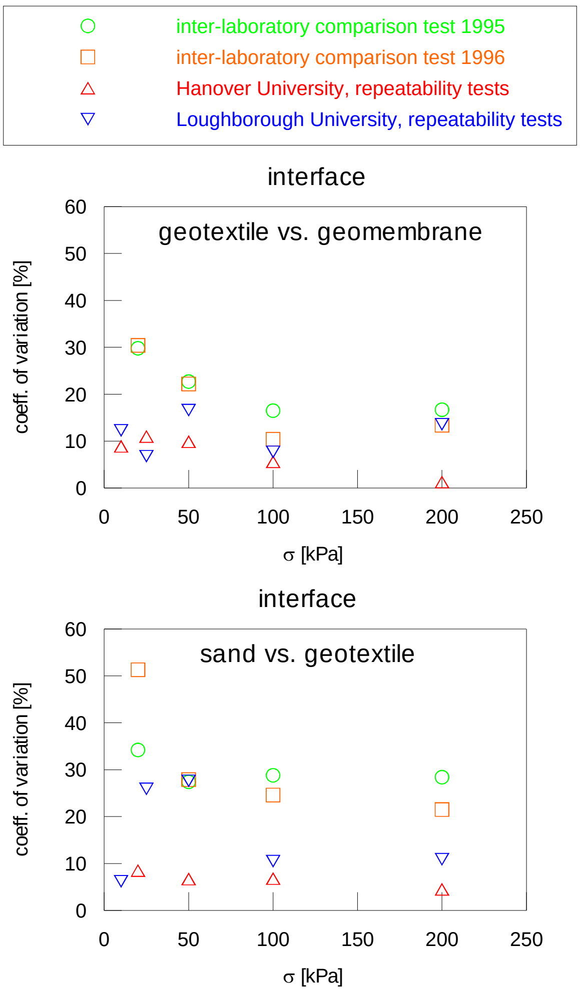

Figure 8. Coefficient of variation of peak shear strength vs. normal stress σ from repeatability and inter-laboratory comparison testing programs.

Results from the test programs are presented as coefficient of variation vs. normal stress for interfaces between sand and a geotextile as well as between a geotextile and a geomembrane. Each point represents a number of tests on materials from the same source conducted at the same normal stress. The two important trends that can be observed are, reduced scatter of data is obtained if tests are carried out in one laboratory (not surprisingly), and the coefficient of variation increases with decreasing normal stress for all repeatability testing. The latter trend is of practical importance to the design of shallow structures, such as landfill cover systems, due to the increased uncertainty in measured interface strengths at low normal stresses.

3.3 *Variability of measured interface strengths: Data bases*

Dixon et al. (2006) present a summary of measured strengths and an assessment of variability for common generic interfaces. This combines values from the international literature, an internal database and results of repeatability testing programs. Interfaces involving GCL are excluded from the summary as these have been considered in detail by McCartney et al. (2004) using a similar approach. The data presented by Dixon et al. (2006) were obtained from 76 sources including journal papers, conference proceedings and internal shear testing reports. Shear strength data for seven interfaces commonly found in landfills are reported, including both geosynthetic/soil and geosynthetic/geosynthetic interfaces. The combined database consists of over 2500 shear strength values, each representing either a peak or a large displacement value. Dixon et al. (2006) present summary plots of measured peak and large displacement shear strengths for each interface, with associated best fit trend lines. Also reported are plots of coefficient of variability against normal stress and standard deviation against normal stress, with associated trends, for each interface.

The data sets presented by Dixon et al. (2006) show a large variability in the number of tests and their distribution across the range of normal stresses. This is to be expected as the data sets are, in the main, compilations of tests conducted for a range of specific purposed. However, despite this there is sufficient data to demonstrate general trends. It was anticipated that data sets would show ranges of variability dependent upon the number of variables involved in testing (e.g. test equipment, personnel, test specification and material). For example, literature data sets would be expected to show greater variability than inter-laboratory data sets obtained using material from only one source. However they do not show this trend. Apart from the repeatability results, the data sets for a given interface (both peak and residual) define similar ranges of shear strength with respect to normal stress. This is surprising because it indicates that differences in measured strengths resulting from material variability (e.g. from type of texturing, type of soil/conditions etc) represented in the literature and internal data bases are of the same order as that resulting from tests on the same materials but at different laboratories. Therefore, it could be concluded that for a given generic type of interface, the range of measured shear strengths is primarily controlled by test conditions rather than differences in the materials.

Best fit lines for combined data sets can be used to define the relationship between standard deviation and normal stress for each interface type. This information can then be used to calculate shear strength parameters for use in stability analyses (i.e. using equation 2, Section 4). The summary standard deviations are conservative values because they include different materials, test equipment and test specifications and hence would be expected to give upper bound values. The small number of repeatability test data sets, for example the Criley & Saint John (1997) data, gave smaller variability and these values of variability are more likely to be representative of those that would be achieved in site specific repeatability tests. Of note, is that coefficient of variation values are not constant but in general decrease with normal stress. This is consistent with coefficient values for soil φ′ summarized by Phoon & Kulhawy (1999).

Sia and Dixon (2007) present a comprehensive repeatability testing program for three generic interfaces: non-woven needle-punched geotextile against coarse grained soil (NWGT-coarse), textured high polyethylene geomembrane against non-woven needle-punched geotextile (TGM-NWGT) and textured high polyethylene geomembrane against fine grained soil (TGM-Fines). To investigate the possible variability and uncertainty in generic interfaces, two interface shear strength databases compiled by Dixon et al. (2006) and Koerner and Narejo (2005) were combined to form a global database containing interface shear strengths obtained from an extensive literature review, internal databases, and inter-laboratory datasets. The repeatability testing program was conducted to investigate the minimum variability that can be expected by using single operator, equipment, procedures, and materials from single sources. The repeatability testing program consisted of 15 tests carried out at each of four normal stresses, for each of the three interfaces.

Example plots comparing the global and repeatability data for the non-woven geotextile vs. coarse soil interface are shown in Figure 9. For all the interfaces, Sia and Dixon (2007) demonstrate that the global datasets display 3-5 times greater variability and uncertainty than that of repeatability tests. Figure 10 shows the variation and uncertainty in the measured non-woven geotextile vs. coarse soil shear strengths corresponding to each normal stress in terms of coefficient of variation (COV). The power function is fitted to obtained an empirical relationship between the variation and applied normal stresses. In general, the global and inter-laboratory datasets contain higher variability and uncertainty in comparison to the repeatability datasets, which is consistent with the findings of Dixon et al. (2006).

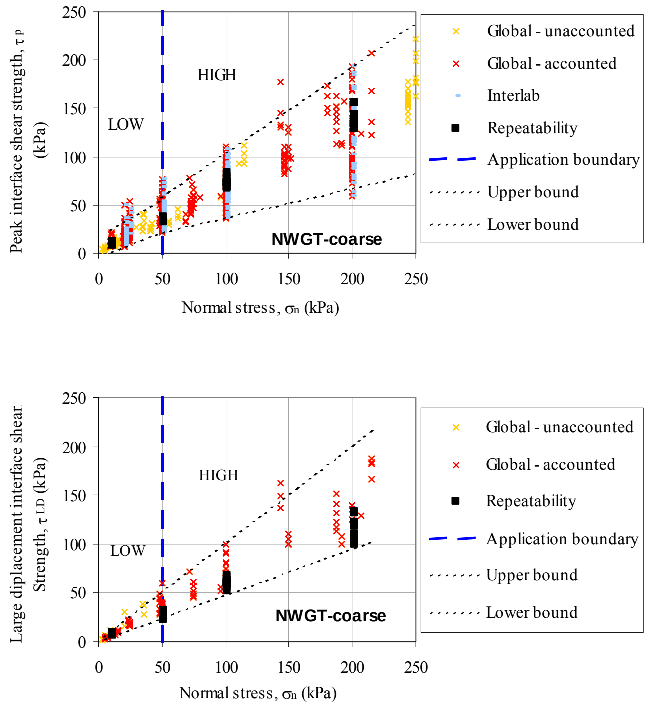

Figure 9. Non-woven geotextile vs. coarse interface: upper - peak, lower - large displacement interface shear strengths (after Sia and Dixon 2007)

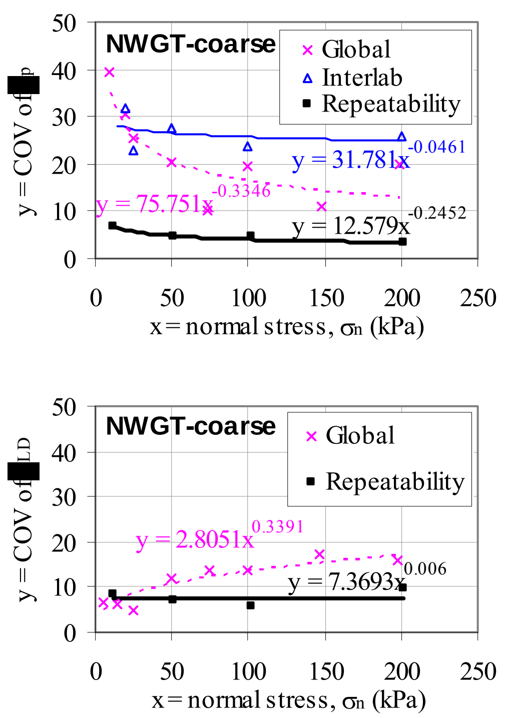

Figure 10. Variability of τ_p and τ_{LD} for NWGT-coarse in terms of coefficient of variation (COVs) (after Sia and Dixon 2007)

4 CHARACTERISTIC SHEAR STRENGTH PARAMETERS

Information on variability of interface shear strength parameters is required for both traditional limit equilibrium and reliability based stability calculations. It is standard practice to obtain characteristic values of governing parameters (i.e. conservative values) for use in design. In Eurocode 7 (1997), the characteristic value of a soil property is defined as: '*A cautious estimate of the value affecting the occurrence of the limit state*'. Hence, the characteristic value should be a cautious estimate of the mean value over the governing zone of soil (Orr & Farrell, 1999). Assessment of an interface between a geosynthetic and soil requires characteristic values of the shear strength parameters that produce a cautious calculated mean shear strength over the entire area of the interface involved in the potential failure.

Selection of characteristic values of soil and geosynthetic properties must take account of:

- Inherent variability of soil;
- Inherent variability of manufactured geosynthetic materials;
- Measurement errors; and
- Extent of zone governing behaviour of limit state being considered

Measurement errors are a significant factor and are caused by equipment, procedural, operator and random test effects (Section 3).

Eurocode 7 advises that: '*If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of a limiting state is not greater than 5%*'. Schneider (1997) has proposed a statistical approach for determining the characteristic value (X_k) using the mean value of the test results (X_m) and the standard deviation of the test results (σ_m):

$$X_k = X_m - 0.5\sigma_m \qquad (2)$$

The approach aims to ensure in the order of 95% confidence that the real statistical mean of the interface strength is superior to the selected X_k. This equation has been in use in Switzerland for many years and has been proven to produce values that are in close agreement with values estimated by experienced geotechnical engineers (Schneider, 1997).

The process of obtaining design parameters is typically: Selection of *representative samples* → *Measured values* (e.g. results of laboratory direct shear tests - peak and residual shear strengths at specific normal stress levels) → Calculated *derived values* based on theory, empirical relationship or correlations (e.g. obtaining α_m and δ_m values that describe the best fit straight line through the measured strengths) → Calculated *characteristic values* α_k and δ_k (a cautious estimate of α_m and δ_m as discussed above) → Calculated *design values* α_d and δ_d obtained by applying partial factors to α_k and δ_k.

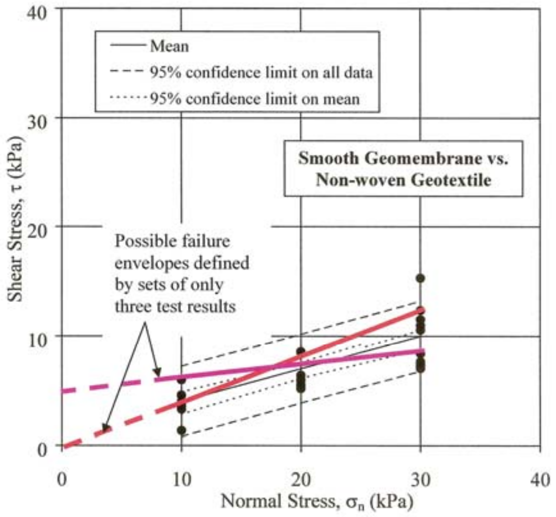

Figure 11. Variability of test data and possible failure envelopes defined by sets of only three tests

4.1 *Derivation of interface shear strength parameters*

Interface shear strength parameters are obtained by plotting peak and residual shear strengths measured in direct shear apparatus on a shear stress vs. normal stress graph (Figure 3). It is rare for duplicate tests to be carried out at each normal stress, and hence failure envelopes are typically taken as the best-fit straight line through one point at each of three or four normal stresses. Given the inevitable scatter of measured interface strengths, this approach provides insufficient information to enable characteristic strength parameters to be selected. If only one or two tests are conducted at each normal stress, it is not known whether the measured shear strengths are high, low or in between values and the potential scatter of measured strengths is also unknown. Depending upon the position of the measured strengths within the possible range at each normal stress, the best-fit line can have a variety of positions, and hence a wide range of shear strength parameters could be obtained. Figure 11 demonstrates possible strength envelopes that can be obtained if a limited number of tests are conducted. The results are from a series of drained repeatability tests conducted on a smooth geomembrane vs. non-woven needle punched geotextile at low normal stresses.

Although shear strength envelopes are defined by pairs of apparent adhesion (α) and slope angle (δ) parameters, it is common practice in soil mechanics to ignore apparent adhesion values in design. However, it is advised that this approach is not used automatically for geosynthetic interfaces. Apparent adhesion values can be taken into consideration in design of structures incorporating interfaces when they are:

- a measure of true strength at zero normal stress (e.g. the hook and loop affect between non-woven needle punched geotextile and textured geomembranes and internal strength of a laminated geocomposite);
- used to define a failure envelope over a range of normal stresses (i.e. assuming a linear failure envelope) when the full envelope curves towards the origin at lower normal stresses; and
- used to define a best-fit straight line through limited variable test data (see Figure 11).

In these cases it would be over conservative to assume $\alpha = 0$, especially for design cases with low normal stresses (e.g. design of cover systems). Therefore, as the quantification of interface shear strength requires two parameters (α and δ) it is not appropriate to obtain characteristic values for the shear strength parameters derived directly from the best-fit straight line through the measured values. A methodology has been proposed by Dixon et al. (2002) where by characteristic *shear strengths* are calculated for each normal stress and then these 'corrected' strengths are used to derive characteristic shear strength parameters α_k and δ_k.

The analyses by Dixon et al. (2002) show that unconservative high shear strengths can be obtained if limited interface shear strength data sets are used. This has important implications for selection of characteristic values (α_k, δ_k), as these must provide a *cautious estimate* of interface shear strength. It is clear that the present common practice of requesting one test at each normal stress is insufficient to calculate a mean value or to assess the variability of measured shear strengths. Hence current practice is inadequate to enable characteristic interface shear strength parameters to be obtained and guidance on selection of characteristic values is required.

4.2 *Guidance on selection of characteristic shear strength parameters*

Three approaches for obtaining characteristic shear strength parameters from laboratory test data are summarized below (after Dixon et al. 2002). They are listed in order of preference.

4.2.1 *Generation of site-specific statistical data*

Selection of characteristic values using a site-specific statistical analysis of test data is the most rigorous approach. It requires multiple performance tests to be conducted at each normal stress to enable the mean (X_m) and standard deviation (σ_m) of measured strengths to be calculated for each stress level. The characteristic shear strengths (X_k) can then be calculated from equation (2). The process is demonstrated in Figure 12.

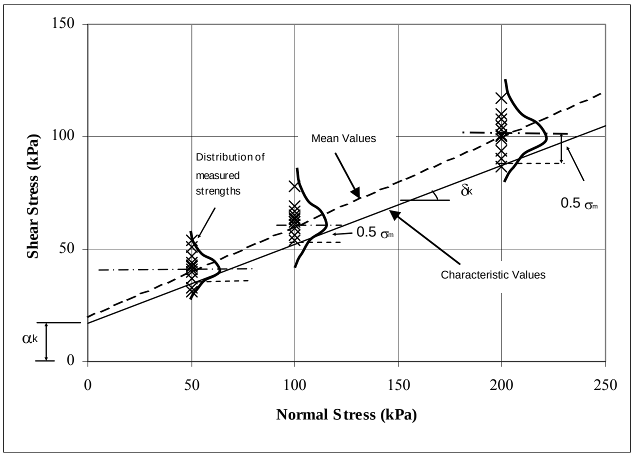

Figure 12. Derivation of interface shear strength parameters from measured shear strengths

Characteristic shear strength parameters (α_k and δ_k) are obtained from the best-fit straight line through the characteristic shear strengths. This approach is based on assessing the variability of measured shear strengths and not the derived shear strength parameters. A sufficient number of tests should be carried out to allow a valid statistical analysis. It is proposed that a minimum of four tests should be conducted at each of three normal stresses (i.e. a minimum of 12 tests in total). However, the number of tests required is also dependent upon the level of existing information relating to the shear strength of the interface being tested. Although, it should be noted that variability of geosynthetics and soils could result in significant differences in shear strength for what appear to be similar interfaces. The level of experience of the engineer interpreting the test results should also be taken into consideration. This approach may appear an expensive option due to the large number of tests required, however the experience of the Author indicates that significant errors can result from carrying out an inadequate number of tests.

4.2.2 *Lower bound of limited repeatability test data*

Recommendations provided by the Germany Geotechnical Society related to the design of waterfront structures involving soils is for three tests to be conducted at each of three normal stresses (EAU 1990). The failure line defining the characteristic shear strength parameters is taken as the best fit straight line through the lowest measured strength at each normal stress (i.e. a lower bound to the test data). The selection of three tests is consistent with the guidance in Eurocode 7 (1999) Part 2, Table A.9.2, which suggests carrying out three tests in cases where the results exhibit significant scatter and there exists a medium level of comparable experience. Where there is limited test data available, an alternative approach is to calculate standard deviation using the three-sigma rule, which uses the fact that 99.73% of all values of a normally distributed parameter fall within three standard deviations of the average (Duncan 2000). The three-sigma rule has been used by Sabatini et al. (2002) to quantify the variability of geosynthetic/soil interface strength. While in this approach a smaller number of tests can be carried out, it can lead to over conservative (i.e. low) strength parameters being calculated.

4.2.3 *Method using statistical data from databases*

An alternative approach is to carry out a limited number of tests to obtain site specific strength values and to use information from published databases (e.g. Section 3.3) on the possible variability for that specific type of interface. Note that design based wholly on literature values should not be attempted. A limitation of this approach is that there is an inherent assumption that the measured strengths at each normal stress are approximately representing mean values. As shown in Figure 11, this may not be the case and they could be significantly higher or lower than the mean. Therefore, the engineer must decide whether the measured strengths approximate to mean values. Schneider (1997) proposed a possible approach for estimating the mean values by using the relationship:

$$X_m \approx (a + 4.b + c) / 6 \qquad (3)$$

Where: a = estimated minimum value
b = most likely value
c = estimated maximum value

The estimation of the values a, b and c can be based on the engineers experience and personal judgment, backed by published data (e.g. Section 3.3). If in comparison to the estimated mean values the measured strengths are considered to be high or there is limited experience of testing the interface, and hence difficulty defining minimum, maximum and most likely values, then further tests should be conducted and the characteristic values obtained using the approach described in Section 4.2.1. Blindly using a best-fit line through only 3 tests to define mean shear strengths could result in significant over estimation of the characteristic values and hence to an unsafe design.

Dixon et al. (2002) provide an example based on analysis of variability of measured interface shear strengths from a published database. The database is used to provide statistical information on the magnitude of scatter of measured shear strengths for non-woven needle punched geotextile vs. sand and geomembrane vs. geotextile interfaces, and hence to obtain characteristic values of the shear strength parameters.

4.3 *Distribution of shear strength parameters*

The simple statistical methods outlined above (e.g. equations 2 and 3) are based on an assumption that the distribution of the measured shear strength values are normally distributed. Normal distributions are also used as models for factor of safety (Duncan, 2000; Sabatini et al., 2002; Koerner, 2002; Dixon et al., 2006). Sia and Dixon (2007) use the results of repeatability tests (e.g. Figure 9) to investigate the assumption of normality, and they conclude that a normal distribution can be assigned to interface shear strengths and derived parameters. This was demonstrated through distribution fitting involving probability plots and histograms to indicate visual fit of the global and repeatability datasets to the empirical distribution, comparing the departures from normality using skewness and kurtosis of the datasets, and computing the goodness-of-fit statistics for acceptance. They note that a standardized distribution type for interface shear strengths is of significant importance in practice especially when comparing decisions on material selection between competing companies for a project. The normality assumption is recommended for interface shear strengths because (1) measured variability and uncertainty (e.g. equipment, operators, test procedures, materials) are believed to be additive, (2) most statistical inferences are based on the normality assumption, and (3) practicing geotechnical engineers are more familiar with normal distributions.

5 SELECTION OF PARAMETERS FOR DESIGN

This section contains comments on selection of appropriate target factors of safety and interface shear strength parameters using landfill lining systems as the design example.

5.1 *Consequences of failure*

The mobilisation of post-peak shear strengths could result in a loss of integrity of a lining system, for example large displacements along a side slope interface could lead to the loss of liner protection materials (both soils and geosynthetics). A common approach is to ensure that the weakest interface is above the primary liner so that any deformations do not result in movement within the liner itself (e.g. Gallagher et al. 2003). However, adequate protection must be ensured when large displacements are anticipated above the primary liner and this is particularly an issue when considering steep wall landfill lining systems since the relative displacement at the interfaces can be in the order of several metres.

The consequence of failure must be reflected in the selection of the strength parameters and factors of safety. For example, failure of a basal lining system would be costly and difficult to repair, whereas veneer stability failure, although highly undesirable, could be repaired with lower disruption and cost. For high risk design cases such as failure of a basal liner, both Gilbert (2001) and Thiel (2001) proposed an approach based on ensuring that the factor of safety using the residual strength controlling lining system stability is > 1.0 (if only just so), with a higher factor of safety obtained using the peak strength (say 1.5). Essentially in this approach the consequence of failure is being taken into consideration, all be it in a simplistic way.

5.2 *Selection of controlling interface*

An important consideration when selecting whether to use peak or residual shear strengths is 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). This approach confirms the importance of carrying out site specific interface tests for all combinations of materials to be used in the lining system, and the need to obtain the full shear strength/displacement relationship for each interface. It is only when armed with this information that the designer can identify the interface(s) controlling stability and then apply appropriate factors of safety. It should be noted that at locations along a lining system it is possible for the controlling interface to be different. This can occur due to the normal stress dependency of interface shear strength.

5.3 *Minimizing interface displacements*

Displacements can be induced at geosynthetic interfaces in a landfill by a number of mechanisms including the placement of soils onto the geosynthetic and the large settlement that the waste body undergoes. Therefore, since the amount of shear strength mobilised at these interfaces is dependent upon the displacement at the interfaces, an estimate of the displacement is needed to provide values for mobilised interface shear strengths.

A number of mechanisms associated with the construction and operation of landfill facilities can result in relative displacements occurring at geosynthetic/geosynthetic and geosynthetic/soil interfaces and hence in mobilization of post-peak shear strengths. Those related to construction activities are:

- Dragging geosynthetic materials over one another to position correctly;
- Construction plant loads (including acceleration and braking forces) from trafficking interfaces with inadequate cover. Particular attention should be given to placement of veneer soil layers on slopes (Koerner and Daniel 1997, Jones et al. 2000);
- Compaction of fine grained soils above geosynthetic layers. This should be particularly discouraged on slopes; and
- Improper storage and handling of geosynthetics leading to loss of internal strength (e.g. breaking of glued connections in geocomposites).

Activities associated with landfill operations are:

- Placement of waste against side slopes (i.e. similar issues as placement of veneer soil layers);
- Settlement of waste adjacent to the interface. This includes both short-term compression and long-term creep/degradation related settlements; and
- Differential settlement of the sub-grade beneath a basal liner or of waste beneath a cap.

The designer must consider all possible mechanism that could potentially result in the mobilisation of post-peak, and even residual, strength conditions for the interfaces under consideration. This assessment should be used to justify the selection of strength parameters and factors of safety used in the design.

If there is a mechanism to generate post-peak shear strengths then it is suggested that residual shear strengths are used in design. A factor of safety that would be deemed acceptable in this instance would be less than the factor of safety against failure using peak shear strengths. If the design allows for the development of post-peak shear stresses, then it follows that displacements will occur at the interfaces. If such displacements are not desired, then it is suggested that peak shear strengths are used in the analysis with a suitable factor of safety that reflects the consequence of failure.

6 INCLUDING VARIABILITY OF INTERFACE STRENGTH IN DESIGN

Although the use of characteristic parameters allows consideration of interface strength variability in the design process, the traditional practice of using a factor of safety to ensure adequate design has major limitations. A factor of safety (FS) does not explicitly consider uncertainties inherit in the model and design parameters. By adopting a single common safety factor for a typical system such as FS of 1.5 to ensure slope stability, the approach does not consider objectively any additional information, lessons learned, technology gained or advances that have been made to improve and optimize design. These limitations have led researchers to propose the use of reliability based design approaches. However, before design engineers can use reliability based stability analysis, guidance is required on quantifying variability of interface shear strength and on use of outputs from such analyses, in conjunction with traditional factors of safety, in the decision making process to ensure design of stable slopes. The use of reliability assessment in landfill stability is demonstrated by Dixon et al. (2006) through consideration of two common landfill design cases: Veneer and waste slope stability. Veneer stability has also been considered by Koerner & Koerner (2001) and McCartney et al. (2004) and waste slope stability by Sabatini et al. (2002), to demonstrate the sensitivity of landfill design to interface variability.

The reliability analyses carried out by Dixon et al. (2006) show that relatively high probabilities of failure are obtained when using interface strength variability values obtained from the global database even when factors of safety ≥ 1.5. The use of repeatability datasets produced lower probabilities for typically used factors of safety, although they are still higher than commonly accepted target probability of failure values.

Designing based on combined criteria for factor of safety and probability of failure would allow uncertainty in measurement of interface shear strength to be considered fully. However, appropriate and attainable target factor of safety and probability of failure values need to be selected if this methodology is to be implemented into general practice. It is clearly unacceptable to rely on low values of factor of safety when using interface data with a large standard deviation. Conversely, when repeatability tests have been carried out to derive interface shear strength, requiring a factor of safety in excess of 1.5 to achieve an acceptable probability of failure will in many cases be considered over conservative by designers, and this will inhibit use of the method. Repeatability data sets have been shown to produce lower variability and hence more realistic information, and therefore it is recommended that repeatability data be used for design in place of the combined global datasets. Unfortunately, to date there is only a small number of such studies reported in the literature (see Section 3.3). Additional repeatability studies on common interfaces are required.

Probability of failure analysis is an appropriate technique to apply to landfill design. The simple method used in studies by Koerner & Koerner (2001), Sabatini et al. (2002), McCartney et al. (2004) and Dixon et al. (2006) require the same input information on shear strength variability as traditional stability analyses using characteristic values.

7 CONCLUSIONS

This paper provides a summary of current knowledge and practice regarding the measurement of geosynthetic vs. soil and geosynthetic vs. geosynthetic interface shear behaviour. It specifically excludes geogrids. Key questions that have to be considered by engineers specifying and interpreting results from laboratory tests were highlighted and responses are provided below based on the information presented in the paper.

What type of tests should be carried out to characterize interface interaction? Performance tests should be carried out using site specific materials (i.e. geosynthetics and soils) and relevant boundary conditions. Database information on generic interface shear strength should never be used as the sole source of information. These databases can help inform the range of expected strengths when used in conjunction with engineering judgment, but they can not replace the need for performance testing. The most common type of laboratory test world wide uses the direct shear apparatus. Considerable experience exists in both the specification of tests and interpretation of results using these devices. However, all laboratory intercomparison test series published to date have produced a large degree of variability in measured behaviour. It is believed that this is primarily due to differences in design of the direct shear apparatus used. The specifier of tests is recommended to obtain information on the design and operation of the device used, specifically including the degrees of freedom of the top box. This will facilitate some interpretation of measured interface shear behaviour if unusual results are obtained. Common test standards have been reviewed and it is concluded that ASTM D 5321 (2008) is the most detailed and accessible up to date reference document currently in use.

How many tests should be carried out? In order to quantify the variability of measured interface shears strength, and hence enable derivation of characteristic values, it has been demonstrated that multiple tests at each normal stress are required. This is not current practice and it is likely that many engineers will not be able to persuade their client that the cost of this is warranted, especially given the current poor practice regarding the number and quality of performance tests conducted for many projects. Therefore, it is unrealistic to expect that large numbers of test would be conducted for each interface to enable production of meaningful statistics on variability. However, a small number of repeatability tests at a normal stress, when used in conjunction with measured variability obtained from published repeatability test programs, could be used to obtain meaningful characteristic values for the shear strength parameters. The cost of providing site specific data, which allows calculation of mean and standard deviation of measured shear strengths, is likely to be significantly less than the cost of repairing even a veneer slope failure.

Which design parameters should be used? Consideration is given in the paper to factors and processes that can lead to mobilization of post-peak interface shear strengths. These can only be examples as there are numerous mechanism by which this could occur. It is important that an assessment is made for each project and application, including construction and operational conditions, to identify the controlling interface and to assess the likelihood of post-peak strengths being mobilized. The designer can then justify the selection of appropriate interface strength parameters (e.g. peak, residual or somewhere in between) and associated factors of safety.

What are the implications of variability in measured values on design? This question has not been addressed specifically in the paper. However, it should be noted that without quantification of the variability of measured interface shear strength it will not be possible to understand the impact on design. There is a growing realization that using reliability based assessments in conjunction with the traditional lumped sum factor of safety approach has many benefits. By adopting a single common safety factor for a typical geosynthetic/soil system such as FS of 1.5 to ensure slope stability, the approach does not consider objectively any additional information, lessons learned, technology gained or advances that have been made to improve and optimize design. Designing based on combined criteria for factor of safety and probability of failure would allow uncertainty in measurement of interface shear strength to be considered fully. However, reliability based assessment of designs will not develop further unless variability of controlling factors such as interface shear strength are quantified on a regular basis. Regulators, operators and designers need to agree acceptable design requirements in relation to the probability of failure. This could lead to justification of the cost of obtaining the required quality of input parameters in relation to the consequences of failure.

REFERENCES

ASTM D 3080 1990. Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions, *American Society for Testing and Materials*, Philadelphia

ASTM D 5321 2008. Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method, *American Society for Testing and Materials*, Philadelphia

Bemben, S.M. & Schulze, D.A. 1198. The inflence of equipment style and setup dimensions on sand/ geomembrane direct shear test measurements. *Proceedings 6th Internatonal Conference on Geosynthetics*, Atlanta, 453-458.

Blümel, W., Brummermann, K., Lottmann, A. & Stoewahse, C. 1996. Reibung zwischen Erdstoffen und Geokunststoffen im direkten Scherversuch, Braunschweiger Deponieseminar 1996 'Konstruktion, Bemessung und Qualitätssicherung bei Abdichtungssystemen', *Institute for Foundation Engineering and Soil Mechanics*, TU-Braunschweig, Vol 51.

Blümel, W. and Stoewahse, C. 1998. Geosynthetic Interface Friction Testing in Germany - Effect of Test Setups, *Sixth International Conference on Geosynthetics*, Atlanta, 1, 447-452

Blümel, W., Stoewahse, C., Dixon, N., Kamugisha, P. & , Jones, D. R. V. 2000. British-German Cooperative Research on Geosynthetic Friction Testing Methods, *2nd European Geosynthetics Conference EuroGeo 2000*, Bologna

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.*

BS 6906: Part 8 1991. Methods of test for geotextiles. Determination of sand-geotextile frictional behaviour by direct shear. *British Standards Institution*.

BS EN ISO 12957-1 2005. Geosynthetics – Determination of friction characteristics. Part 1 – Direct Shear tests. *British Standards Institution*.

Criley, K.R. & Saint John, D. 1997. Variability analysis of soil vs. Geosynthetic interface friction characteristics by multiple direct shear testing. In: *Proceedings of Geosynthetics 1997*, Long Beach, California, pp. 885-897.

DIN 18137-3 1997. Baugrund, Untersuchung von Bodenproben - Bestimmung der Scherfestigkeit, Teil 3: Direkter Scherversuch, *Deutsches Institut für Normung*, Berlin

Dixon, N. & Jones D.R.V. 1995. Discussion on "Landfill Liner Interface Strengths from Torsional Ring Shear Tests". *Journal of Geotechnical Engineering*, Vol. 121, No. 6, ASCE. pp. 509-510.

Dixon, N., Blumel, W., Stoewahse, C., Kamugisha, P. & Jones, D.R.V. 2002. Geosynthetic Interface Shear Behaviour: Part 2 Characteristic Values for use in design. *Ground Engineering*, March, 49-53.

Dixon, N., Jones, D.R.V. & Fowmes, G.J. 2006. Interface shear strength and its use in reliability-based landfill stability analysis. *Geosynthetics International*, 13 (1), 1-14.

Duncan, J.M. 2000. Factors of safety and reliability in geotechnical engineering. *Journal of Geotechnical and Geoenvironmental Engineering*, 126 (4), 307-316.

EAU 1990. Recommendations of the Committee for Waterfront Structures Harbours and Waterways. Sixth English Edition, *Wilhelm Ernst & Sohn*.

Eurocode 7 1994. Geotechnical Design - Part 1: General rules. *ENV 1997-1:1994*.

Gallagher, E.M.G., Needham, A.D. & Smith, D.M. 2003. Use of geosynthetics in landfill steepwall lining systems. *Proceedings 1st IGS UK National Geosynthetics Symposium, Geosynthetics: Protecting the Environment*, Thomas Telford Publishing.

Gartung, E. & Neff, H.K. 1998. Empfehlungen des Arbeitskreises 'Geotechnik der Deponiebauwerke' der Deutschen Gesellschaft für Geotechnik e. V. (DGGT), *Bautechnik 74*, H. 9, pp 564 ff

GDA E 3-8 1997. Empfehlungen des Arbeitskreises 'Geotechnik der Deponiebauwerke' der Deutschen Gesellschaft für Geotechnik e. V. (DGGT)", *Bautechnik 74*, H. 9: 564 ff.

Gilbert, R.B. 2001. Peak versus residual strength for waste containment systems. *Proceedings of 15th Geosynthetic Research Institute Conference on Hot Topics in Geosynthetics – II*, GRI Publishers, 29-39.

Gourc, J.P. & Lalarokotson, S. 1997. Research and Intercomparison Tests for the Harmonization of Standards on Geotextiles, EC Measurement and Testing Programme *Project 0169 Task 3.2 Friction*, Report No. 3.

Gourc, J.P., Pitanga, H.N., Reyes-Ramı´rez, R. & Jarousseau, C. 2006. Questions raised regarding interpretation of inclined plane results for geosynthetics interfaces. In: *Proceedings of the Eighth International Conference on Geosynthetics*, Yokohama, Japan, pp. 259-264.

Jones, D.R.V. & Dixon, N. 1998. Shear strength properties of geomembrane/geotextile interfaces. *Geotextiles and Geomembranes*, 16, 45-71.

Jones, D.R.V. & Dixon, N. 2000. A comparison of geomembrane/geotextile interface shear strength by direct shear and ring shear. *Proceedings of 2nd European Geosynthetics Conference*, Bologna, Italy, October, Eds. Cancelli, A., Cazzuffi, D. & Soccodato, C., 2, 929-932.

Jones, D.R.V., Dixon, N. & Connell, A. 2000. Effect of Landfill Construction Activities on Mobilised Geosynthetic Interface Shear Strength, *Proc. EuroGeo 2, Second European Geosynthetics Conference*, Bologna, Eds. Cancelli, A., Cazzuffi, D. & Soccodato, C., pp 581-586.

Koerner, R.M. 1998. Designing with geosynthetics. 4th Edition, *Prentice Hall*.

Koerner, R. M. 2002. Beyond factor of safety: The probability of failure. In: *Proceedings of the GRI-16 Conference on "Hot Topics in Geosynthetics - III*", Philadelphia, Pennsylvania, pp. 1-18.

Koerner, R.M. & Daniel, D.E. 1997. Final covers for solid waste landfill and abandoned dumps, *ASCE Press*, Reston, USA.

Koerner, R.M. & Koerner, G.R. 2001. Geosynthetics design beyond factor of safety; risk assessment using probability of failure analysis. *Proc. GRI-15, Hot Topics in Geosynthetics-II*, December 2001, Houston, 235-253.

Koerner, G.R. & Narejo, D. 2005. Direct shear database of geosynthetic-to-geosynthetic and geosynthetic-to-soil interfaces. *GRI Report #30*, Geosynthetic Research Institute, USA.

Lee, S.W., Frost, J.D. & Righter, G.K. 1996. The influence of geomembrane surface roughness on geomembrane-geotextile interface strength. *Proceedings 6th International Conference on Geosynthetics*, Atlanta, Editor K. Rowe, 1, 433-442.

McCartney, J.S., Zornberg, J.G., Swan Jr, R.H. & Gilbert, R.B. 2004. Reliability-based stability analysis considering GCL shear strength variability. *Geosynthetics International*, 11 (3), 212 - 232.

Marr, W.A. 2001. Interface and interface shear testing procedures to obtain peak and residual values. *Proceedings of*

15th Geosynthetic Research Institute Conference on Hot Topics in Geosynthetics – II, GRI Publishers, 1-28.
Orr, T.L.L. & Farrell, E.R. 1999. Geotechnical Design to Eurocode 7. *Springer*.
Phoon, K.K. & Kulhawy, F.H. 1999. Characterization of Geotechnical Variability. *Canadian Geotechnical Journal*, 36, 612-624.
Pitanga, H.N., Gourc, J.P. & Vilar, O.M. 2009. Interface shear strength of geosynthetics : Evaluation and analysis of inclined plane tests. *Geotextiles and Geomembranes*, 27, 435-446.
Sabatini, P.J., Griffin, L.M.,Bonaparte, R., Espinoza, R.D. & Giroud, J.P. 2002. Reliability of state of practice for selection of shear strength parameters for waste containment system stability analyses. *Geotextiles and Geomembranes*, 20 (4), 241-262.
Schneider, H.R. 1997. Definition and determination of characteristic soil properties. *Proc. 14th International Conference on Soil Mechanics and Geotechnical Engineering*, Hamburg, 4, 2271-2274.
Sia, A.H.I. & Dixon, N. 2007. Distribution and variability of interface shear strength and derived parameters. *Geotextiles and Geomembranes*, 25, 139-154.
Stark, T.D. and Poeppel, A.R. 1994. Landfill liner interface strengths from torsional ring shear tests, *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 120, 3, 597-615.
Stoewahse, C. 2001. Ermittlung des Reibungsverhaltens von Geokunststoffen und Erdstoffen im Rahmenschergerät, *Publications Institute for Soil Mechanics and Foundation Engineering*, Hanover University, Vol 56.
Stoewahse, C., Dixon, N., Jones, D.R.V., Blümel, W. & Kamugisha, P. 2002. Geosynthetic Interface Shear Behaviour: Part 1 Test Methods. *Ground Engineering*, February, 35-41.
Thiel, R. 2001. Peak vs. residual shear strength for landfill bottom liner stability analysis. *Proceedings of 15th Geosynthetic Research Institute Conference on Hot Topics in Geosynthetics – II*, GRI Publishers, 40-70.