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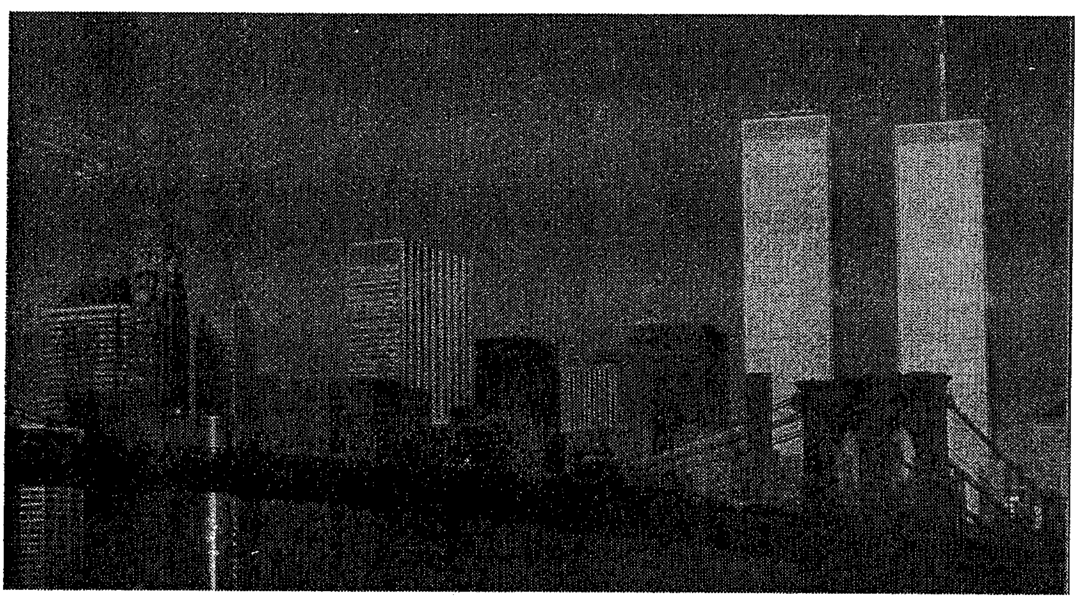
HOT TOPICS IN GEOSYNTHETICS - II

(PEAK/RESIDUAL; RECMs; INSTALLATION; CONCERNS)



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INTERFACE AND INTERNAL SHEAR TESTING PROCEDURES TO OBTAIN PEAK AND RESIDUAL VALUES

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ABSTRACT

Equipment for measuring peak and residual strength of geosynthetic materials is reviewed and discussed in the context of obtaining internal and interface strength values for design. Advantages and disadvantages of various testing devices are discussed. Recommendations are given to help establish some of the key testing variables that affect peak and residual strength. Recommendations are also given for improving specifications for laboratory testing to measure interface and internal shear strength.

SOME FUNDAMENTAL CONCEPTS

Interface and internal shear testing of geosynthetic materials is aimed at measuring the shear resistance available to hold things in place against the forces of gravity and extreme loads. Interface shear represents the shear resistance between two different materials, such as that between a textured geomembrane and a geocomposite. As illustrated in Figure 1, there are several interfaces in a typical landfill liner and cover system, any of which may provide the weakest plane for shear to occur.

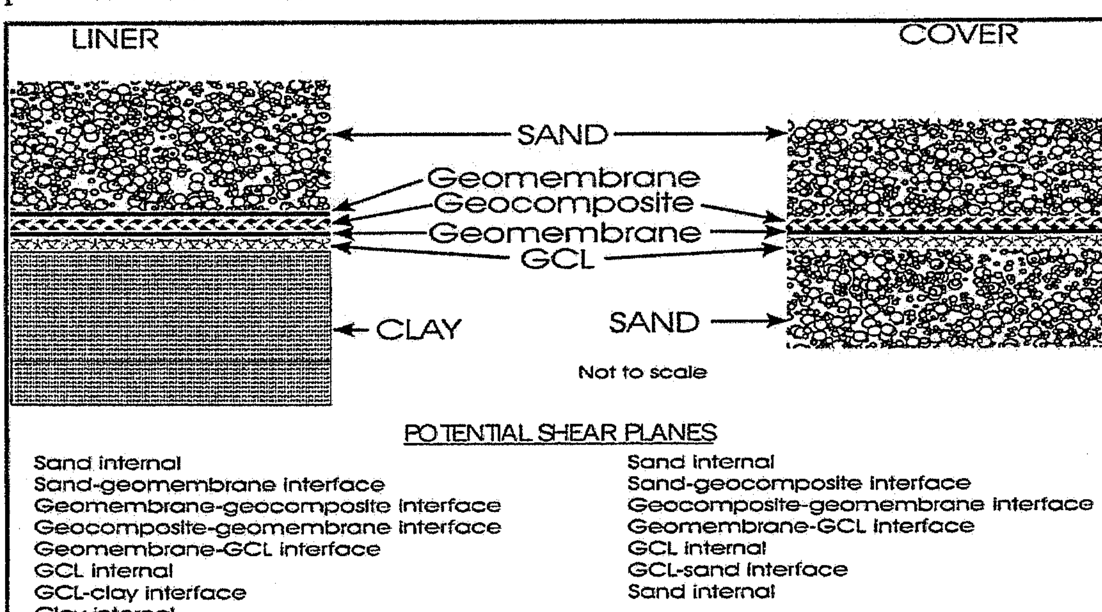

Figure 1: Typical Interfaces in Landfill Liner and Cover Systems

Internal shear represents the shear resistance internal to a particular material, such as internal to a soil. Materials that might develop internal shear failures in certain conditions include soils, GCLs, geocomposites, and potentially some thick nonwoven geotextiles. Interface and internal shear strength are a direct function of the effective normal stress on the shear plane. Effective stress is the total stress minus the pore pressure. Pore pressure may be a gas pressure, or a fluid pressure, or both. Gas and fluid develop no shear resistance for the conditions where geosynthetics are typically used. Gas and fluid pressure acts to reduce effective normal stress and thereby reduce shear resistance.

Some materials develop shear resistance at zero effective normal stress. This is called cohesion if internal to soils, or adhesion, if on an interface. Its practical effect becomes less pronounced as the effective normal stress increases. The interface between a nonwoven geotextile and a textured geomembrane has a small adhesion. The adhesion for a geotextile/textured geomembrane interface is enough to keep a geotextile placed on top of a geomembrane from sliding off a 2 horizontal to 1 vertical side slope, but is of little consequence at higher normal stresses appropriate for the design of liners and covers. The interface between a nonwoven geotextile and a smooth geomembrane has essentially zero adhesion. Cohesion or adhesion can be destroyed by concentrated shear displacements.

Shear resistance of materials is usually measured in a laboratory device designed to simulate field conditions. A shear box is generally used to measure the interface and internal shear resistance of geosynthetic materials. Figure 2 illustrates the conceptual basis of most shear boxes.

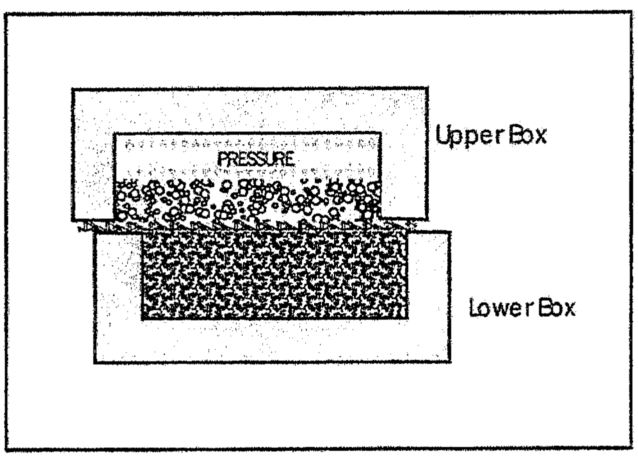

Figure 2: Basic Elements of Shear Box

The box consists of two parts, which will be referred to as the lower box and the upper box. One half is kept fixed while the other is moved horizontally. The horizontal displacement and the force required to cause this movement are measured.

The specimen is placed in the box so that the intended plane of shear is aligned with the plane separating the two halves of the shear box. A known normal stress can be applied to the specimen perpendicular to the intended plane of shear.

Figure 3 shows a typical shear box, in this case one that can hold a 12 inch by 12 inch (0.3 by 0.3 m) square specimen. This is the standard box size specified in ASTM D-5321 and D-6423. Shear boxes have been used to test specimens from as small as 1 square inch to larger than 1 square meter.

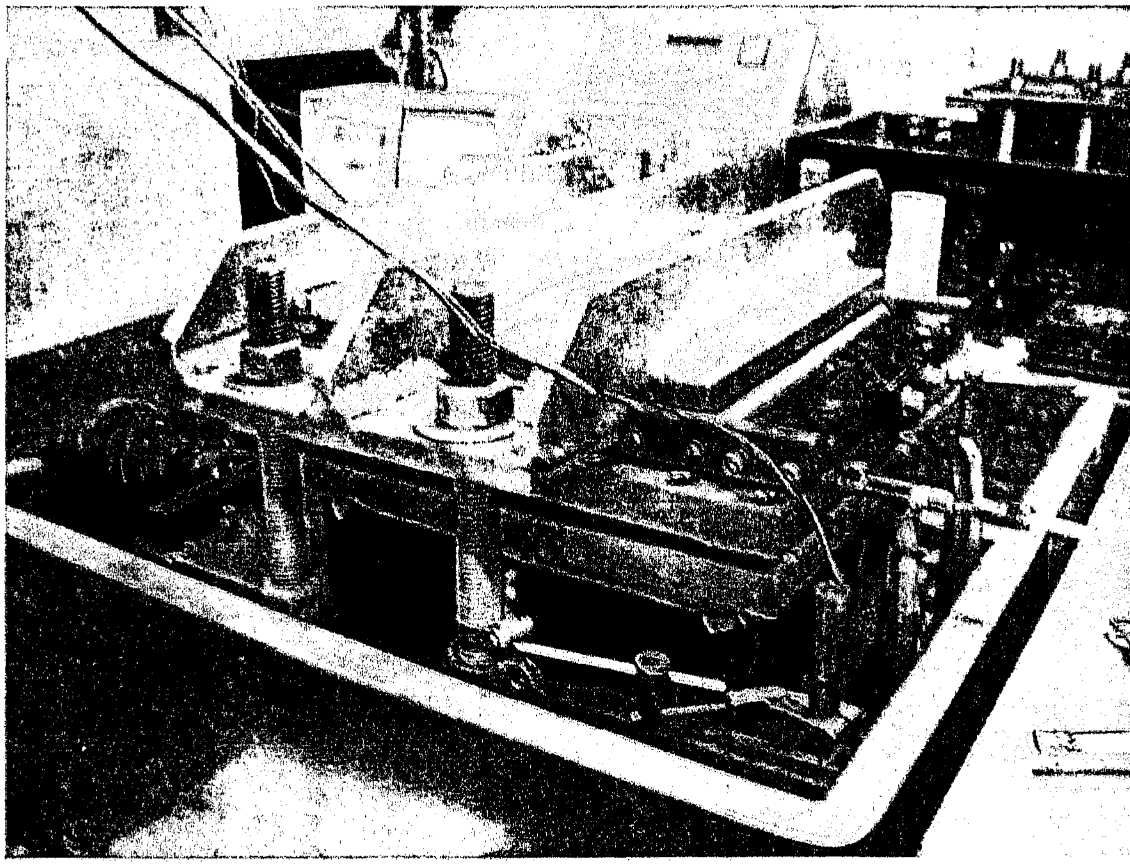

Figure 3: Shear Box for 12 Inch Specimens

Figure 4 shows a set of typical data obtained with a shear box on the interface between a textured geomembrane and the geotextile component of a geocomposite. The data are shear resistance (horizontal force divided by area of the sample) versus horizontal displacement of one half of the box relative to the other half. The different curves are for specimens subjected to different normal stresses. Each curve shows that the shear resistance reaches a peak value and then decreases with continued horizontal shear to some constant value. The peak value indicates the peak strength of the interface. The constant value at large displacement is called the "post-peak" strength. It may also be called the residual strength, if the post-peak resistance reaches a constant value.

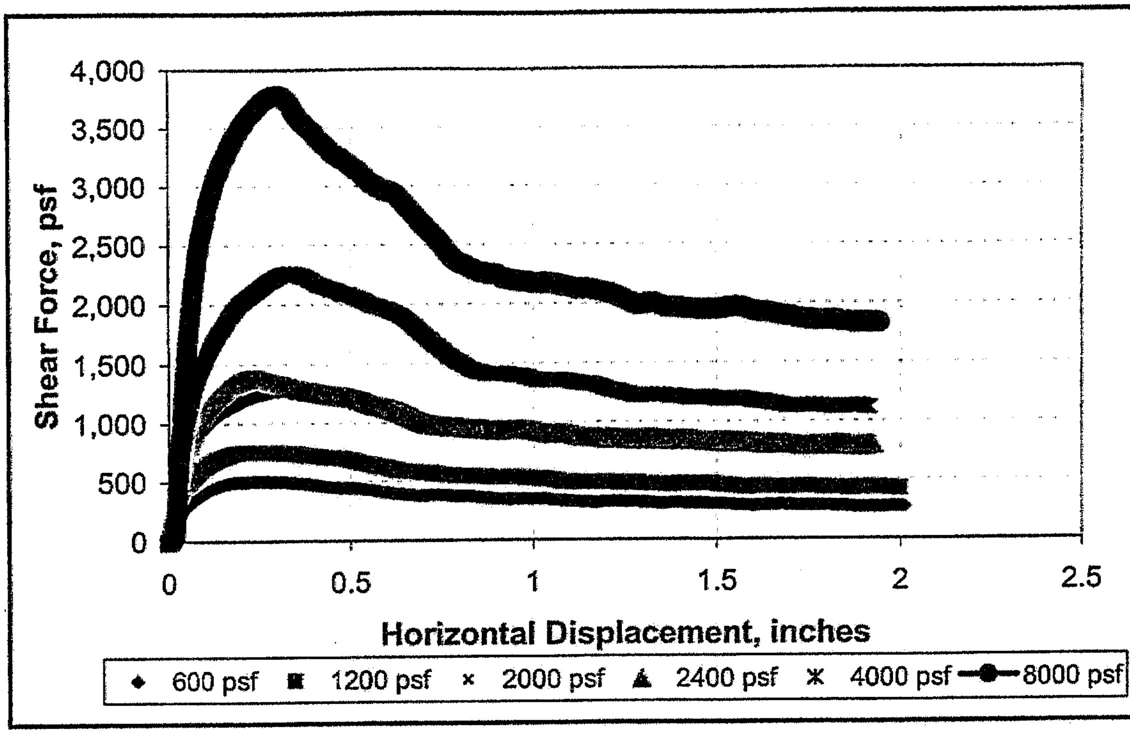

Figure 4: Direct Shear Test Results for Geomembrane-Geocomposite Interface

These data are usually summarized in a plot like Figure 5 that shows strength versus normal stress. Figure 5 shows peak strength data and the strength at a displacement of approximately 2 inches (50 mm).

Typically, strength increases as effective normal stress increases. Many materials show a linear increase of strength as effective normal stress increases. This permits one to use a linear equation to define strength as follows:

$$\vartheta = c' + \Phi'_n * \tan(N') \qquad (1)$$

where ϑ is the shear strength, c' is called the cohesion intercept, Φ'_n is the effective normal stress, and N' is the friction angle in terms of effective stress. (The prime superscript is used to indicate that these parameters are determined using effective stresses.) Equation 1 is called the peak strength envelope. c' is the value of shear strength at zero normal stress. N' is the angle of the line relative to horizontal. Conceptually, N' represents the steepest slope on which the material will remain stable, if cohesion and pore pressure are zero. For interface strength, it is common practice to replace "c" in Eq. 1 with "a" for adhesion and N with *. As shown in Figure 5, a linear relationship may apply over a specific range of normal stresses, but over a large range, the strength envelope may curve downward.

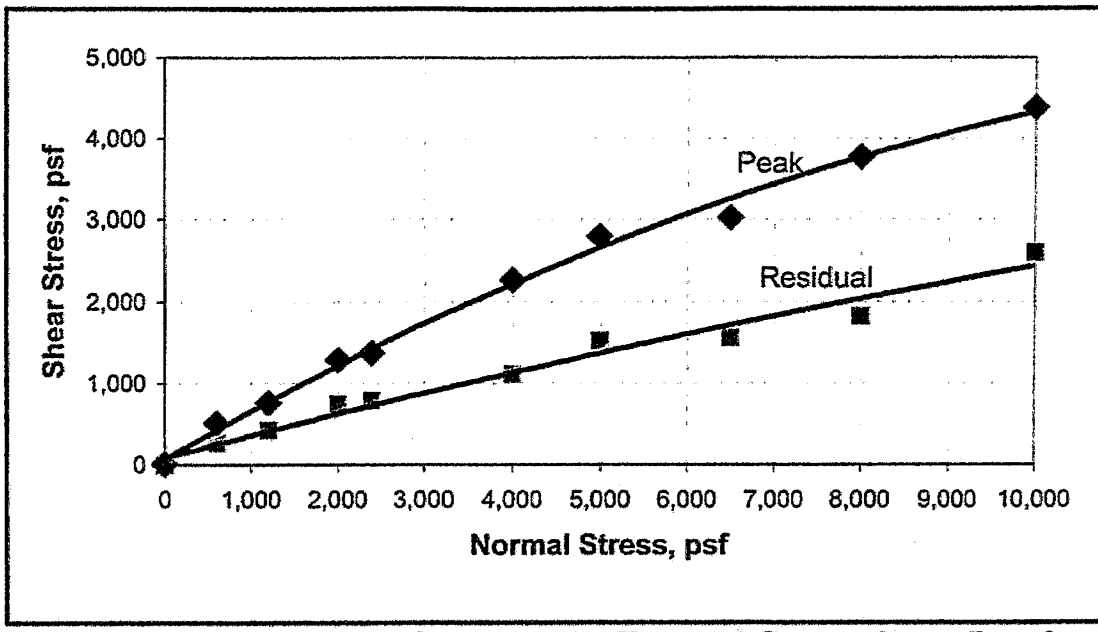

Figure 5: Strength for Geocomposite-Textured Geomembrane Interface

One more concept is essential to understanding interface and internal shear strength issues. That is the role of pore pressure on strength. Its role is indicated by the effective stress equation, which states that the effective stress equals the total stress minus the pore pressure, or:

$$\sigma_n' = \sigma_n - u \qquad (2)$$

External forces such as gravity produce total stress. Internal fluid or gas pressure produces pore pressure. Effective stress controls strength behavior. Equation 1 can be written in terms of total stress as:

$$\vartheta = c' + (\Phi_n - u) * \tan(N') \qquad (3)$$

Equation 3 shows that positive pore pressure acts to reduce strength by reducing the effective normal stress. A child's air hockey game illustrates this concept. If the game's air blower is turned off, the hockey puck's movement over the playing table is quickly slowed by friction. With the air blower on, air pressure between the table and the puck supports the weight of the puck. This reduces the effective stress between the puck and the table, which reduces the frictional resistance. The puck glides back and forth over the playing surface. Similarly, gas or fluid pressure can support some of the weight of overlying materials in a geotechnical setting. This reduces the effective stress and thereby reduces the shear resistance within soils and geosynthetic materials.

Pore pressure results from steady state conditions and from variant conditions. Static groundwater causes pore water pressure. A foot of fluid in the leachate collection layer creates a pore pressure of about 63 psf along the interface of the drainage material and the liner. Atmospheric pressure causes pore gas pressure. Gas pressure beneath a cap may build up to the equivalent of several inches of water column, even with a gas collection system in place. Adding or removing load may cause pore pressure to increase or decrease. Changing groundwater conditions can change pore fluid pressure. Waste decomposition can change pore gas pressure. Shearing of a material can cause pore pressure to increase or decrease. Flow of fluid or gas into or out of a material to adjust to external boundary conditions can increase or decrease pore pressure. As shown by Equation 3, interface and internal shear strength depend on how pore pressure changes with changes in site conditions and with time.

Pore pressure different than steady state conditions causes flow. The flow of fluid depends on the permeability of the material. Some materials have high permeability. Gas and fluid can flow into and out of these materials faster than we can load them or shear them. For practical purposes, pore pressure in these materials remains at static values. For dry materials, there is no fluid to create pore fluid pressure. Gas can flow through most dry soils relatively quickly. Hence, for dry materials, effective stress equals total stress and there is no concern with pore pressure. (However waste decomposition building up gas pressure beneath an unvented lined cap, or saturated gas causing a dry silty sand to become saturated and lowering its gas permeability to essentially zero can produce positive pore pressure.)

Some materials have low permeability and require years for small quantities of fluid to flow short distances. For example, a molecule of water flows through bentonite at the rate of about 1 inch per 100 years under a gradient of 1. It takes a long time for excess pore pressure to dissipate in bentonite. Understanding the internal and interface strength behavior of geosynthetic and soil materials requires careful consideration of the applicable pore pressure conditions.

Figure 6 shows typical stress-displacement curves for common geo-materials at one value of effective normal stress. These curves show some of the following characteristics:

- Some materials, such as Material 6 in Figure 6, require a relatively large displacement to mobilize their peak strength.
- Some materials, such as Material 1 in Figure 6, experience substantial loss of strength with continued displacement beyond their peak strength.
- Materials develop their peak strength at different values of displacement. Material 2 and Material 6 in Figure 6 show typical extremes.

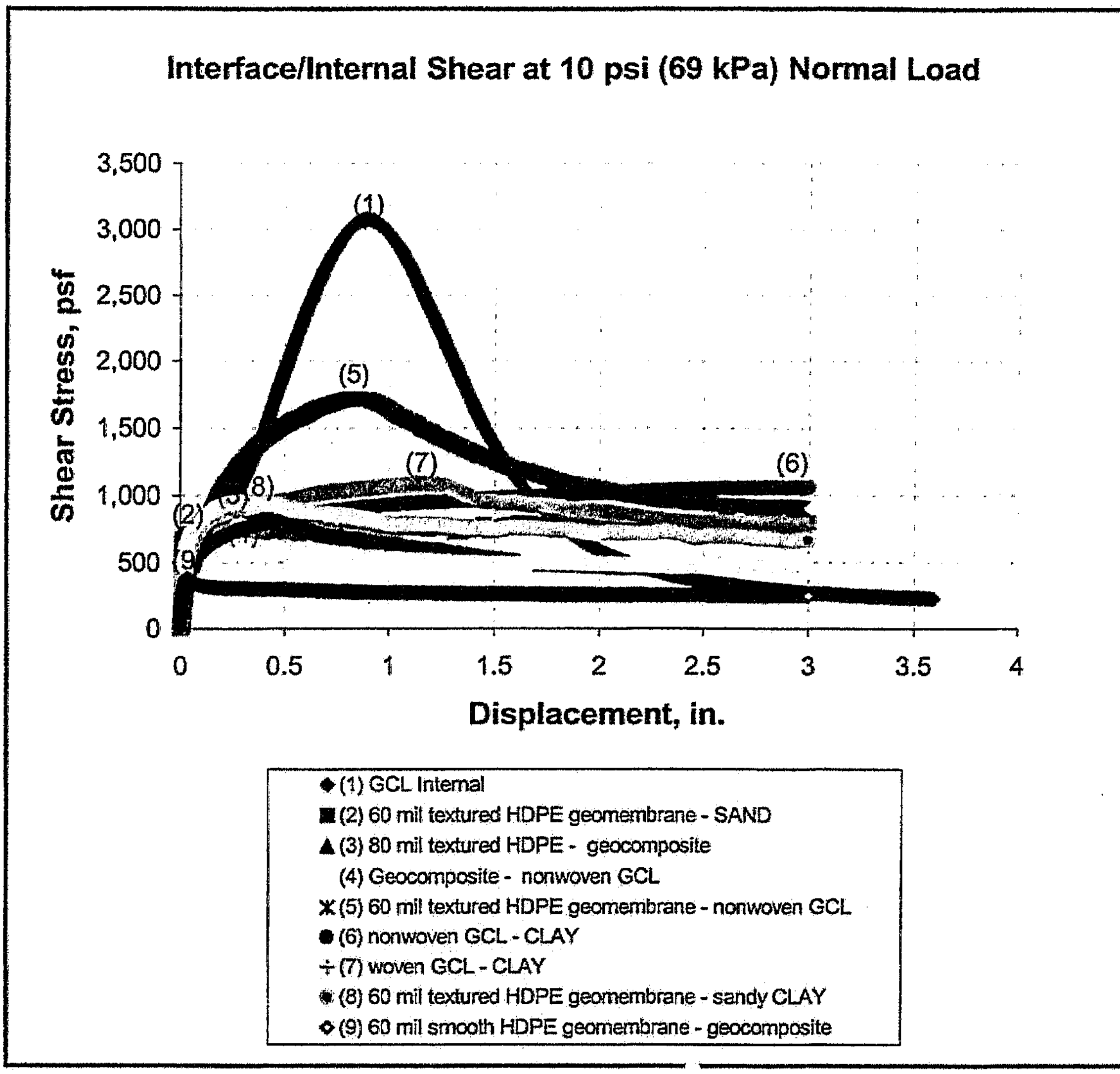

Figure 6: Interface and Internal Shear for Various Materials

The drop off in shear resistance for displacement past the peak strength depends on the materials involved. For clays it is primarily due to the gradual reorientation of clay particles into parallel, face-to-face arrangements (Lambe and Whitman, 1969). For dense soils, it may be due to an increase in volume of the material within the shear zone. For a textured geomembrane, it may be due to a loss of roughness as shearing occurs, or an increase in volume of the sheared material adjacent to the geomembrane. For a reinforced GCL, it is due to rupture of the needle punched fibers and a reorientation of the bentonite clay particles. Figure 7 shows peak and post-peak internal strength for a typical needle-punched GCL. The peak strength is represented by a best-fit straight line with the parameters c =

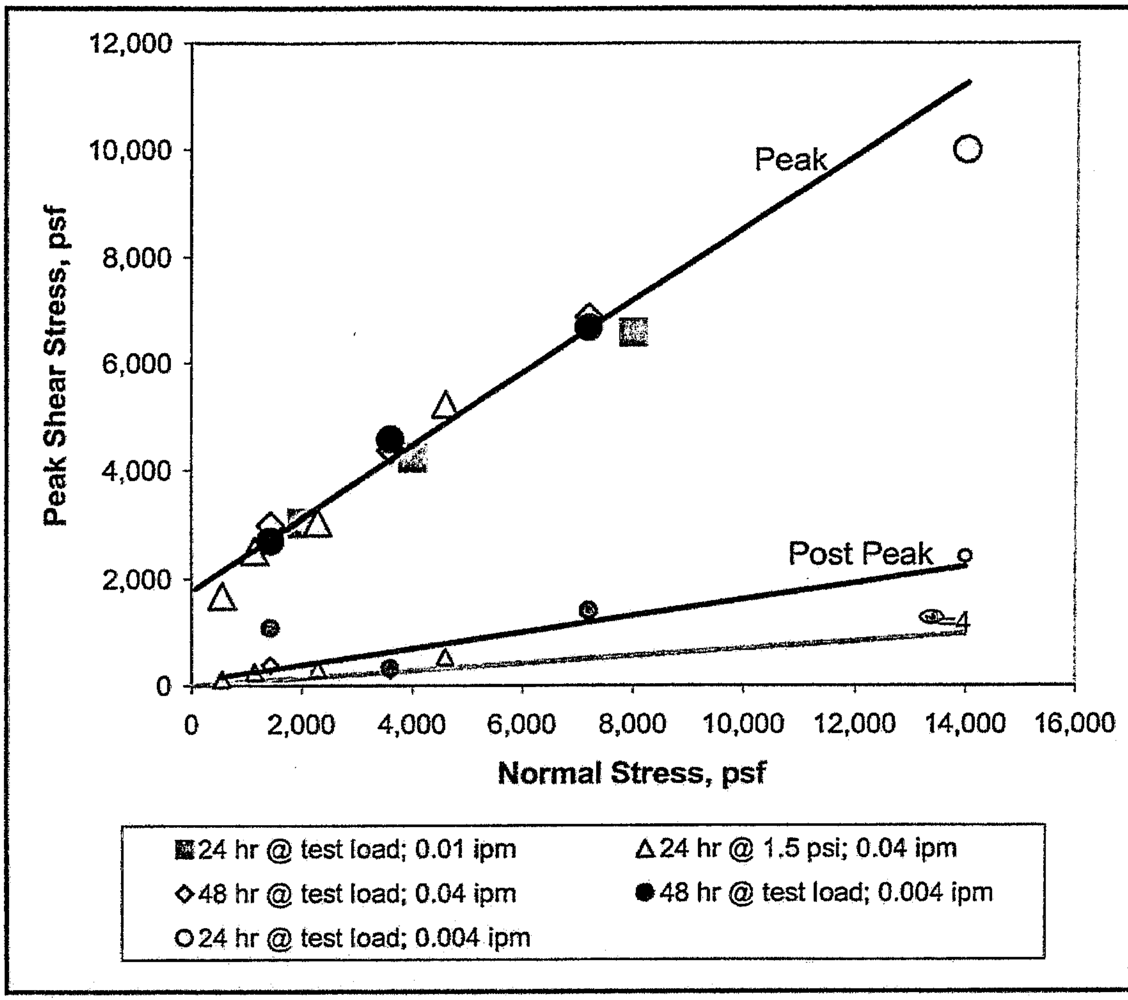

Figure 7: Internal Strength of GCL

1,760 psf (84 kPa) and N = 34°. A curved line would provide a better fit to the peak strength in Figure 7, suggesting that the slope of the peak strength envelope decreases at higher normal stresses for this GCL. The residual strength is represented by a best-fit straight line with c = 0 and N = 9. This is higher than the residual strength of bentonite. Values of residual N as low as 3-4 degrees have been measured for bentonite (Lambe and Whitman, 1969 referencing work by Herrman and Wolfskill, 1966), Mesri and Olson, 1970. Muller-Vonmoos and Loken, 1988. The data in Figure 7 are for specimens sheared at 0.004 to 0.04 inches per minute (0.1 to 1 mm/min). As will be shown later, it is possible that these displacement rates are too high to produced drained strength parameters for internal shear of the GCL. Shearing too fast may be causing negative excess pore pressure, which gives an apparent higher strength envelope (as can be seen from Eq. 3). However, residual strength of GCL being higher than that for bentonite may also be due to the presence of the plastic fibers within the GCL. Although they are completely broken or have pulled out of their opposing surfaces at a displacement of 3 inches (76 mm), they may be providing additional resistance within the bentonite to resist shear. Another explanation is that 3 inches (76 mm) of relative displacement is not sufficient to achieve residual strength in the bentonite. Relative displacements along the shear plane on the order of 1 m may be required to reach residual strength in highly plastic clays (Skempton, 1964). Stark and Poeppel, 1994 indicated that 40-60 cm might be required to obtain residual shear strength in GCL materials.

The literature includes a number of publications that summarize results of shear testing on various interfaces. Richards and Scott (1985) looked at data for geotextile-soil interfaces. Williams and Houlihan (1987) looked at various geosynthetic-soil interfaces. Lydick and Zagorski (1991) examined geonet-soil interfaces. Leisher (1992), Gilbert et al (1997) and Fox et al (1998) examined internal and interface strength for GCLs. These studies and compilations provide useful data and insight into the strength performance of various geosynthetics and geosynthetic-soil interfaces. However, experience has shown that a number of material factors, specific site conditions, and details of the testing can substantially influence the interface and internal shear strength of these materials. Internal and interface strength should be measured for each application with the specific materials and conditions applicable to the site's unique conditions.

TEST EQUIPMENT OPTIONS

Equipment to test interface and internal shear of geosynthetics is primarily an adaptation, modification or extension of equipment used to measure the strength of soils. The first measurements of interface shear strength of geosynthetic materials were performed in a small shear box using the procedures given in ASTM D-3080. Boundary effects on the shearing surface and increases in the size of the geometric patterns of the geosynthetics to more than 10 mm (e.g. geonets and geogrids) made it necessary to look for equipment that could test larger specimens. Standard testing practice limits the size of a geometric element being tested to less than about 10% of the size of the test specimen. With the introduction of stitch bonded GCLs and the concern for residual shear strength of these materials, the need arose for a device that could produce larger horizontal displacements within the shear plane. These needs lead to larger versions of the basic shear box equipment and eventual standardization around a shear box with a specimen size of 12 by 12 inches (0.3 by 0.3 m) that could produce displacements along the shear plane of 3 to 4 inches (75 to 100 mm). Large shear box testing of geosynthetic interfaces and geosynthetic-soil interfaces is addressed in standard ASTM D-5321. Large shear box testing of GCLs to measure internal strength is addressed in standard ASTM D-6423. Even the 3 to 4 inch displacement possible in a 12 inch box may not produce sufficient movement to reach residual shear strength in highly plastic clays.

Some laboratories have built tilt tables that can test specimens several feet in size. The geosynthetic is fastened to a base plate. The second material, a geosynthetic or soil is placed into a box positioned over the base. The entire apparatus is raised on one side to induce a shear force along the interface. Devices are used to measure the angle of tilt and the displacement of the top box. The angle at which the top box slides off indicates the peak internal coefficient of friction along the slip plane. Once peak strength is reached, failure occurs rapidly in the tilt box. While the displacement along the shear plane in a tilt table test can be sufficient to reach residual strength, it is not easy to measure the residual strength because the top slides off suddenly and completely after peak strength is reached.

Some have adopted the rotational, torsional or ring shear device to test geosynthetic interfaces. This device uses a donut shaped specimen with the upper half fixed to provide and measure resisting torque and the lower half rotated about the center to reach high values of displacement. Stark and Poeppel (1994) concluded peak strength from ring shear is similar to that from a large direct shear box. They also found similar results for residual strength if the material develops residual strength within displacement limits of a shear box. Vaid and Rinne (1995) concluded that a more confident measurement of residual strength is obtained in ring shear than in conventional direct shear. Jones and Dixon (2000) provide data from torsional shear tests that displaced by as much as 12 m with results that are only marginally lower than obtained from 12 inch shear box tests displaced 90 mm.

A recent device by Moss (1999) folds the test specimen into a cylinder and rotates the inside of the cylinder relative to the outside. This device can potentially test large specimens and produce large displacements along the shear plane.

Other geotechnical shear testing equipment can be helpful in studying the strength behavior of geosynthetic materials. These include the triaxial test and the direct simple shear test. These devices are primarily used to measure the internal strength of soil materials used in conjunction with geosynthetics. The triaxial equipment consists of a cylindrical specimen encased in a flexible watertight membrane and surrounded by a chamber. Pressure is applied inside the chamber to equal the stresses that develop in the specimen in the ground. Force is applied along the axis of the cylindrical specimen to cause it to shear. Pore pressure within the test specimen and volume change of the specimen can be measured as well. Triaxial cells with specimens up to 12 inches (0.3 m) in diameter have been used; however the more common sample sizes are diameters of 1.4 to 3 inches (35 to 76 mm). Triaxial testing permits us to more closely duplicate the stress and drainage conditions that develop in the field. Triaxial testing is the only practical way we have to measure the pore pressure behavior of specimens under undrained conditions. However the shear plane in a triaxial test is inclined at an unknown angle typically between 30 and 60 degrees from vertical. It is very difficult to test a geosynthetic interface in this equipment. It is not possible to reach large displacements along the shear plane with available equipment and conventional test procedures.

The direct simple shear test is similar to the shear box described above except that the rigid sides of the box are replaced with some mechanism to allow the sides to slip in the horizontal plane without losing confinement of the test specimen. A common approach is to stack thin rigid rings together to create a test chamber. Each ring can slide independently of its neighbor. The faces of the rings are treated to minimize friction between rings. Direct simple shear provides a more uniform shear strain across the height of the specimen and allows the shear plane to develop at any position within the specimen. Conceptually a direct simple shear box would be an ideal way to measure the drained strength of an entire liner system consisting of multiple interfaces in one setup. The equipment is considerable in size and expense however.

Table 1 summarizes some of the advantages and limitations of the different test devices used to measure interface and internal shear strength of geosynthetic materials. Despite its limitations, the large shear box test is the most commonly used equipment to measure interface and internal strength of geosynthetic materials for the following reasons:

- Controls the shear plane to follow the interface between materials
- Can achieve shear displacements large enough to approach residual strength
- Can test large enough sample to minimize geometry and edge effects
- Existing base of equipment and personnel to respond to the needs of the industry
- Large and successful experience using the results from the test

ISSUES WITH SHEAR BOX TESTING

A number of factors affect the strength results obtained with shear boxes. Manufacturers of the testing equipment and those using the equipment take different approaches to deal with these factors. As a result, strength data from multiple laboratories on the same materials may differ considerably. This has been demonstrated a number of times in inter-laboratory round robin testing and with problems on specific projects. Some of these factors are discussed in this section. The discussion is meant to heighten awareness of the issues but not to necessarily resolve them.

Table 1: Devices for Measuring Interface and Internal Strength

Test Type	Advantages	Limitations
Standard Direct Shear ASTM D-3080	• Equipment readily available and not expensive • Specimen preparation and setup is relatively easy • Can test undisturbed soil samples	• Small size limits horizontal displacement to about 0.3 inches (7.5 mm) maximum • Cannot measure strength of materials with high cohesion at low normal stresses. • Limited to materials with geometrical features (particle sizes, reinforcement spacing, texturing) less than 1/10 of box size (0.25 to 0.4 inches, 6 to 10 mm) • Failure plane is fixed so measured strength may be higher than for field conditions. • Cannot control drainage so must run slowly • Cannot test undrained conditions
Large Direct Shear (0.3 m) ASTM D-5321 and D-6423	• Can test materials with geometric features up to 1 inch (25 mm) • Can induce shear displacements of 3-4 inches (75-100 mm) • Boundary effects are reduced • Equipment readily available • Can test undisturbed soil samples	• Shear displacement may not be sufficient to reach residual • Friction may be relatively large at low normal stresses • Applied normal load may not get to the failure plane. • Failure plane is fixed so measured strength may be higher than for field conditions. • Significant area change at large displacements changes the vertical and shear stresses. • Cannot measure strength of materials with high cohesion at low normal stresses. • Cannot control drainage so must run slowly • Cannot test undrained conditions • More expensive than standard direct shear test.
Tilt Table	• Large displacements possible • Failure plane is not forced • Can test multiple interfaces at the same time • Can test a constant shear stress over long time easily	• Limited to normal stresses less than about 500 psf (25 kN/m^2). • Measures only peak strength • Limited availability • Considerable effort to use equipment • Can't test hydrated or undrained condition.
Rotational Shear	• Unlimited continuous shear displacement • Area of shear plane remains constant	• Not widely available • Complex specimen preparation procedures • Small specimen size • Forced failure plane location • Difficult to prepare specimens to field conditions • Nonuniform shear displacement within sample • Direction of shear constantly changes • Cannot test undrained condition
Cylindrical Shear	• Unlimited continuous shear displacement • Constant direction of shear displacement • Larger sample size • Reduced edge effects	• Limited experience for dry conditions only • Equipment not widely available
Triaxial	• Equipment readily available • Can match stress conditions to field • Can measure undrained strength and pore pressure • Useful to measure internal strength of particulate components like bentonite	• Limited specimen size • Can't measure interface strength for geosynthetic material
Direct Simple Shear	• Uniform shear stress in sample models actual conditions more closely • Can simulate undrained conditions by running as a constant volume test	• Expensive equipment • Limited availability • Not typically used to test geosynthetics

Top and bottom conditions – Different materials are used above and below the geosynthetic samples in a shear box. Metal or wooden plates, rubber sheets and soil are common. Metal or wooden plates are convenient because they can be placed and removed quickly and do not dirty the equipment. However, they may present uneven distributions of normal and shear stress within the tested materials that may affect the measurement of strength. Rubber sheets are sometimes used to overcome this problem. Any of these materials appear to be acceptable for testing interface strength of geosynthetics. Both metal and wooden plates and rubber sheets may impede the flow of water causing incomplete hydration and consolidation, even after several days for GCL materials. Solid materials must have closely spaced holes or other porous materials must be added between these plates and the specimen to allow unimpeded flow of water into and out of the GCL.

Clamping – Some configurations can allow the geosynthetic material to slip at a location different than the interface that is being tested. For example, the coefficient of friction between a metal cover plate and a geotextile is less than that between a geotextile and a textured geomembrane. Consequently, slip will occur between the geotextile and the metal rather than between the geotextile and the geomembrane. This problem is overcome by clamping the materials to force failure through the desired interface. The edges of the geosynthetic materials may be clamped to one half of the shear box to force it to move with the box. Clamping of the edges may not be sufficient. Insufficient friction between the geosynthetic and its backing material may cause the geosynthetic to neck or rupture in tension. It becomes necessary to increase the friction between the geosynthetic and its backing material. Roughening the backing plate to increase the friction, or adding serrated metal files, or gluing the geosynthetic to the backing plate can do this. However, drainage cannot be assured.

Normal stresses – Different approaches are used to apply normal stress to the test specimens, including mechanical, pneumatic and hydraulic systems. The apparent normal stress may not all get to the shear plane, however. For example, in systems that use air pressure inside a strong rubber bladder, only 70 to 80% of the force from the air pressure gets to the shear plane. This value can change with shear even though the applied air pressure remains constant. The measured strength will be less because the normal stress on the shear plane is less than assumed. Friction within the seals of hydraulic pistons can produce the same problem where the force computed from the hydraulic pressure in the cylinder is considerably more than the force delivered to the shear plane. Inserting a load cell between the loading mechanism and the shear plane (or between the shear plane and the bottom of the box) to get a direct measurement of normal force can reduce this problem. The distribution of this force over the shear plane is not known, but the average value of normal stress can be determined by dividing the measured normal force by the area of the sample.

Gap – A gap between the upper box and the lower box is used to remove friction caused by one box sliding on the other. The size of the gap may affect the measured shear resistance. A gap that is too small can allow friction to develop between the two boxes as they displace which increases the apparent strength. A small gap with large particles, such as gravel, can also increase the apparent strength. A gap that is too large permits soil to displace into the gap and create friction between the boxes. This is particularly a problem at larger displacements. Bemben and Schultze (1993) examined the importance of gap spacing in considerable detail. The ASTM standards give no guidance on the size of the gap. The laboratory must apply experience and judgment to select a gap appropriate to the materials being tested.

Area of sample – As the top half of a shear box displaces relative to the bottom half, the area of the sample is reduced by the amount of displacement times the width of the box. For tests in a 12-inch box that are run to 3 or 4 inches (75 to 100 mm) of displacement, this results in an area reduction of 25 to 33%. Since the normal force is maintained at a constant value during shear, the normal stress actually increases by 33 to 50%. Shear stress is also increased by the same ratio, since it is determined by dividing the horizontal force necessary to cause shear by the area of the sample. The data for a test can be easily corrected for the area change, but doing so presents confusion. It results in a normal stress at the post peak condition that is much higher than at the start of the test. This inevitably leads to questions about the laboratory's inability to control the normal stress to the prescribed value. The effect on stresses at peak strength is relatively small, since peak strength usually occurs at less than 1 inch of displacement and the correction would be less than about 10%. Also the effect on the strength parameters computed for a test series is relatively small as all test results are computed in the same way. In fact for a material with no cohesion or an interface with no adhesion, the effect of the area correction is zero on the measured friction angle. Consequently, most laboratories do not make the area correction. This seems like a reasonable thing to do as long as the test report indicates that no area correction was made. Another approach to this problem is to make one half of the box 3 to 4 inches longer than the other half. Then the area of shearing is always constant. This introduces a new problem for materials that lose strength after reaching a peak. The leading edge of the shear plane is engaging virgin material that still has peak strength while the trailing edge is at the post-peak value. The measured shear force is somewhat higher than the actual post-peak value. This is primarily an issue when using this type of box to measure the internal post-peak strength of a GCL.

Friction – Friction between the upper half of the specimen and the sides of the shear box may decrease the normal stress delivered to the shear plane. There is no known way to measure how large this friction may be. The best course is to take every reasonable action to keep it as low as possible. Friction also develops in the horizontal direction that adds to the apparent shear resistance of the specimen. The test standards require horizontal friction to be measured and to be removed from the measured horizontal force. However this is difficult to do with any reliability over the range of normal stresses used in the equipment. The value of friction may also differ for actual test conditions compared to those in the calibration mode. In some equipment at low normal stresses, the friction correction may be a substantial part of the measured peak horizontal force. If the friction correction is more than 10% of the measured horizontal force, the results from the equipment may be suspect.

Calibration – Most shear boxes in use today are outfitted with load cells and displacement transducers to measure load and displacement. These devices require calibration to transform their readings to engineering quantities. Common practice is to use linear calibration factors for these devices. Most electronic sensors have some degree of nonlinearity in their response, particularly at the ends of the calibrated range. Strength measurements should be within 10-90% of the calibrated range of the load cell.

MATERIAL SPECIFIC ISSUES

Shear strength of geosynthetics, soils and their interfaces is very dependent on specific details of these materials. Seemingly small changes in the texturing on a geomembrane, or the needle punching of a GCL, or the density of the soil can have a significant effect on the shear strength. Consequently, published values of strength or values taken from other projects should not be substituted for testing of site-specific materials where stability is possible mechanism of failure.

Site-specific materials – To the extent possible, it is recommended that site-specific materials be used when testing geosynthetic - soil interfaces and geosynthetic – geosynthetic interfaces. If soil is a part of the interface being tested, its should be preconditioned to the highest value of moisture content and placed at the lowest dry density that will be allowed during construction. Soil with particle sizes greater than 0.25 inches (6 mm) should be removed if they constitute less than 10 percent of the sample. If more that 10% are large particles, special arrangements should be made with the laboratory to perform the test using material with the large particles present. In some situations the internal strength of the soil itself may be the weakest plane. This may be the case for plastic clays, placed in a wet condition in the field, and loaded within a few months so that shear induced pore pressures cannot completely drain from the clays during the shearing. In this situation, it is better to perform triaxial strength testing on the soil with consolidation and drainage conditions controlled to match those in the field. Wetter soil conditions, lower soil density, less texturing of a geomembrane, lower peel strength of a geocomposite or GCL, and lower tensile strength of a geotextile are examples of conditions where the failure may be internal or along the interface. It may be useful to perform index tests on the materials to provide a baseline for determining whether the materials are changing. Possible index tests for soils that may relate to strength are grain size, plasticity and optimum moisture content. Possible index tests for geotextiles are thickness, mass per unit area and tensile strength. Possible index tests for geocomposites are peel strengths. Possible index tests for internal GCL strength is peel strength. No unique relationships exist between index test results and strengths of these materials; consequently, index testing is no substitute for strength testing on the site-specific materials.

<u>Specimen Selection</u> – Soils used for internal and interface shear testing should be representative of those used in the field. A minimum sample size to fill a five-gallon bucket should be provided to the laboratory. The sample should be taken from the interior of a stockpile or from a mixed composite sample that represents the material, as it will be placed. For geosynthetics, typical practice is to take a sample encompassing the entire width of a roll, then cutting specimens from locations across the width of the roll. However this approach may produce scatter in the strength envelope due to differences in the material across the roll. For a geomembrane, the specimens should be cut from any portion of the roll that appears to have the least texturing, as this will give the lowest interface strength. For GCLs, the strength of the reinforcement may be less at the sides than in the middle. Peel tests can be performed across the roll to identify which location across the roll has the lowest peel strength. This location should be used for samples for internal strength testing.

<u>Sample Conditioning</u> – The primary issue with sample conditioning prior to shearing is getting the materials to a moisture condition that represents worst-case conditions for the field situation. This is usually not a significant issue with geosynthetic to geosynthetic interfaces since the presence of moisture has no known effect on the frictional resistance of plastic materials, except as it influences static pore pressure. The same results should be obtained for geosynthetic interfaces tested dry or fully wet. (However, some materials may have surface finishes or lubricants present from the manufacturing process that can produce different frictional resistance for wet and dry conditions.) Moisture conditioning is a significant issue with tests involving soils and those involving GCLs. In both cases, the materials should be fully hydrated with the water surface standing at least ½ inch (10 mm) above the shear plane so that all parts of the shear plane has ready access to water during shear. Conditions which deny access

to water during shear produce the opportunity for negative pore pressures to develop during shear which may give an apparent increase in shear strength. Such negative pore pressures may not develop for field conditions, so the test results will not match reality. Hydration should be done with fluid that is representative of that present in the field for hydration. Strength, particularly residual strength of a GCL, may be affected by the chemistry of the hydrating fluid. Where possible, hydration should be done with drainage access to the top and bottom of a GCL. Otherwise the time required to achieve complete hydration can be greatly increased. Hydration should occur under a normal stress representative of that which occurs during the first month after deployment of the materials. GCLs hydrate in the field by pulling water from moist adjacent materials. Hydration should last sufficiently long for the materials to reach moisture equilibrium. The hydration time can be determined by monitoring the change in height of the materials with time, running separate swell tests in a consolidation cell, or using past experience to establish the required hydration time. Some laboratories use a separate setup to hydrate the materials to avoid tying up the shear box. The fully hydrated materials are then transferred to the shear box for consolidation and shearing. This practice appears to work as long as the transfer to the shear box occurs quickly, the shear surface does not become contaminated, and the operation does not damage the softened materials. The transfer should occur and the seating load reestablished within 30 minutes. Table 2 gives some recommended hydration times. Hydration times for some materials may be very long and depend on the seating load used during hydration. Figure 8 shows measurements of change in height of stich bonded GCL specimens for different seating loads. Data in Figure 8 were obtained on 2.5-inch (64 mm) diameter specimens placed in a consolidometer. For higher seating loads, this GCL requires about 20 hours to come to height equilibrium after the application of water. Even then it is not clear that the specimens are at moisture equilibrium, since the height continues to change for as long as 100 to 200 hours after the application of water. For low seating loads, it is clear that it takes this material about 200 hours to reach height equilibrium after the application of water. It is quite possible that for some seating loads, one can be fooled into thinking that the sample has reached equilibrium because the compression has stopped when in fact the sample has reversed direction and started to swell. See the curve for a seating load of 1,152 or 2,304 psf (55 or 110 kPa) in Figure 8. The data in Figure 8 show that the behavior of GCLs during hydration is far from simple. It seems clear that for this particular sample, one should hydrate for at least 200 hours to be certain that the GCL has reached an equilibrium moisture condition before continuing with the rest of an internal shear test (assuming two sided drainage over the entire surface of the GCL).

Table 2: Recommended Minimum Hydration Times

Condition	Recommended Minimum Hydration Time
Geotextile-Geotextile Interface	Hydration not required
Geotextile-Geomembrane Interface	Hydration not required
Geotextile-Sand Interface	15 minutes
Geotextile-Clay Interface	1 hour
Geomembrane-Sand Interface	15 minutes
Geomembrane-Clay Interface	Measure vertical displacement to get end of consolidation/swelling
Geotextile-GCL interface	24 hours
Geomembrane-GCL interface	Measure vertical displacement to get end of consolidation/swelling
GCL internal	Measure vertical displacement to get end of consolidation/swelling

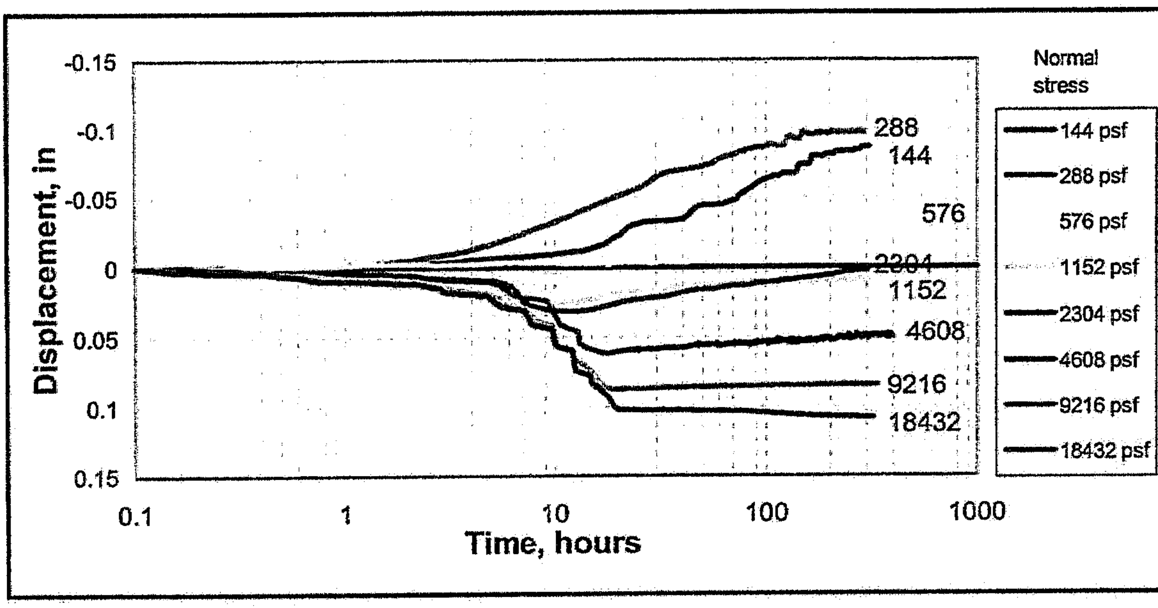

Figure 8: Hydration Times for GCL

Seating Load during Hydration - The seating load during hydration may greatly affect what happens to the specimen during hydration, as Figure 8 demonstrates. Should the seating load be a low value to reflect installation conditions or a high value to reflect completed conditions? The test can be completed sooner if it is hydrated under the load for which shearing will be performed. However this may not represent field conditions. An interface hydrated under full load may not give the same strength behavior as one hydrated under low stress and subsequently consolidated to full load. Hydration may occur relatively quickly in the field. Several GCLs have been examined two to three weeks after they were installed and found to already be soft and pliable. The top surface of a clay liner became wet and greasy from the condensation of moisture underneath a geomembrane a few weeks after installation. It is recommended that GCLs and clays be fully hydrated under the normal stresses representative of field conditions during the first month after installation.

Normal stress and time for consolidation - The normal stress for shearing is generally higher than the seating load used for hydration. For tests involving materials that take more than a few minutes to hydrate and consolidate, the normal stress should be applied in stages by doubling the existing stress in the next step. As an example if the seating load used for hydration is 144 psf (7 kPa), the first consolidation stress should be 288 psf (14 kPa). The next would be 576 psf (28 kPa) and so on until the desired normal stress is reached. It is not necessary that the intermediate load steps be maintained for complete consolidation in each step. Maintaining the load for a time equal to t_{50} allows enough strength gain for the material to support the next load step without squeeze out. For most cases the final load step should be maintained long enough for the materials to completely consolidate before the shearing phase is started. Most field situations involve loading slow enough for the materials to fully consolidate under the applied load. However, cases involving an internal GCL failure, a clay-geomembrane interface, or a nonwoven GCL-geomembrane interface may not be fully consolidated under the applied load during shear. Table 3 gives some recommendations for consolidation times for different materials.

Some engineers specify that a GCL should be fully hydrated under 1 psi (7 kPa) seating load, then loaded in one step to the final normal load, then sheared immediately at a rate of 0.04 in/min (1 mm/min). This is a very severe condition that can never be achieved in the field. Applying the normal load in one step causes significant squeeze of the bentonite out of the GCL. Significant bentonite on the woven side of a GCL and some bentonite on the nonwoven side of a GCL have been observed when subjected to loading of this type. Applying the normal load rapidly in one step to a hydrated sample also induces significant excess pore pressures that require days to dissipate. Initiating shear at a fast rate immediately after applying the normal stress almost certainly results in significant reduction in the measured strength for that normal stress. At normal stresses above about 3,000 psf (150 kPa), this may cause the internal GCL strength to become less than the interface strength between the GCL and a textured geomembrane. The intended interface strength tests actually shears as an internal GCL shear failure. Results from a series of such tests are misleading. They tend to be reported with a "c" and "N". The value of N may be quite low. This can lead to much consternation for situations where a minimum interface friction angle is specified that the test results don't meet, although the measured strength may be higher than the required value due to a high measured cohesion. In fact, the specified test conditions do not match the assumptions in the engineering analyses, and they do not match reality.

Shear rates - Establishing an appropriate shear rate is difficult when testing any material that has a low permeability. A slow shear rate requires a longer testing time, which increases the cost of the test and increases the time to complete the work. The potential for polymer creep may affect the interface strength between geosynthetic materials for the conditions we typically face. (It is interesting to speculate on the interface strength between two geosynthetics during a rapid shear failure where the shear rate might be inches per second. Since the interface would have to be strained beyond peak to establish this condition and getting to peak is normally relatively slow, this rapid strain rate is only of interest when analyzing failure conditions.) The original D5321 standard established the shear rate for geosynthetic interfaces as 0.2 in/min. This is about as fast as we can run the test and collect relevant data without undue complexity. D5321 also established 0.04 in/min for geosynthetic-soil interfaces. This rate is acceptable for geosynthetic-sand interfaces. It is too high for geosynthetic-clay interfaces where a drained strength is required. Rates this high can generate shear induced pore pressures on the failure plane. Table 4 gives some representative values for illustrative purposes. These rates have been deduced from the recommendations given in

Table 3: Recommended Consolidation Time for Normal Stress at Shear

Condition	Recommended Minimum Hydration Time
Geotextile-Geotextile Interface	15 minutes if creep is not significant
Geotextile-Geomembrane Interface	15 minutes if creep is not significant
Geotextile-Sand Interface	15 minutes
Geotextile-Clay Interface	1 hour
Geomembrane-Sand Interface	15 minutes
Geomembrane-Clay Interface	Measure vertical displacement to get end of consolidation/swelling
Geotextile-GCL interface	24 hours
Geomembrane-GCL interface	Measure vertical displacement to get end of consolidation/swelling
GCL internal	Measure vertical displacement to get end of consolidation/swelling

ASTM D6423 and the revision of ASTM D5321 currently being balloted. They use typical values for t_{50} and typical displacements required to reach peak strength. Actual materials may have different values of t_{50} and displacement at peak. The shear rates and times given in Table 4 are those necessary to be sure that the shearing occurs under drained conditions. This is the only condition we can interpret in shear box tests because we cannot control drainage. Some of these times are very long. It is very clear that many of the tests being performed today involving clays and GCLs at shear rates of 0.04 in/min (1 mm/min) are not drained, and that the results from these tests are probably being interpreted incorrectly. The times in Table 4 are based on calculations about the rate consolidation that may be conservative. It might be possible to shorten these times if pore pressures could be measured on the shear plane. Such measurements would show directly how fast the test could be run, or even permit the test to be run much faster in an undrained mode with the measured pore pressures used to interpret the data in terms of effective stress. This is an area that deserves more research. It is common to see a specification requiring a shear rate of 0.04 in/min for all test conditions. For some interfaces and for internal strength of GCL, this rate is too fast to permit shear induces pore pressure to drain. The test becomes a partially drained test, neither drained or undrained. Actual field conditions may be undrained, drained or partially drained. Something that produces a rapid loading, such as an earthquake, is likely to cause undrained shear in interfaces involving clay. Slow loading, such as filling, permits shear induced pore pressures to dissipate during the filling, so shearing is drained. It is not obvious which condition, drained or undrained, is the worst case. The answer depends on the nature of excess pore pressure induced by shear and the rate of shear relative to the rate at which the material can consolidate. Some conditions produce positive excess pore pressures during shear. Positive excess pore pressures lower strength compared to the drained value. The critical design condition for a material that generates positive excess pore pressure during shear is an undrained condition. Other conditions produce negative excess pore pressure during shear. Negative excess pore pressure increases strength compared to the drained value. The critical design condition for a material that generates negative excess pore pressure during shear is the drained value of strength.

Clays placed wet of standard Proctor optimum moisture content and subjected to normal stresses above 100 kPa tend to generate positive excess pore pressures during shear. Short-term undrained loading gives the lowest shear strength for this condition. This condition occurs in many clay liners for landfills during waste placement.

Clays placed dry of standard Proctor optimum moisture content and those that become desiccated by wet-dry or freeze-thaw cycles tend to generate negative pore pressures during shear. Long-term, drained loading gives the lowest shear strength for these conditions. This condition occurs in many landfill caps and during construction of some liner systems.

Table 4: Representative Shear Rates for Shear Box Testing

Condition	T_{50} hrs	Displacement at peak, mm	ASTM Shear Rate, mm/min	Time to peak	Time to 76 mm (3 inches)
Clay (10^{-7} cm/sec) - Geotextile Interface	.1	10	17	1 min	5 min
Clay (10^{-8} cm/sec) - Geotextile Interface	1	10	2	6 min	46 min
Clay (10^{-7} cm/sec) - Geotextile Interfac	.1	10	0.008	20 hr	6 days
Clay (10^{-8} cm/sec) - Geotextile Interfac	1	10	0.0008	200 hr	63 days
GCL – Geotextile Interface	5	10	0.3	30 min	4 hr
Woven GCL – Geomembrane Interface	20	10	0.00004	170 days	1270 days
Woven GCL – Clay Interface	10	10	0.0003	21 days	160 days
GCL internal with normal stress above 100 kPa	5	20	0.001	10 days	40 days
GCL internal with normal stress below 25 kPa	10 0	20	0.00007	210 days	790 days

The nature of shear induced pore pressures in GCLs is not known. They are difficult to measure in this material and tests take a long time. One expects that shear induced pore pressures to be negative for a GCL hydrated under confined conditions with normal stresses below 100 kPa. These negative pore pressures will increase the peak and residual strength of the GCL. One expects positive shear induced pore pressures in a GCL hydrated at low normal stresses and then consolidated to normal stresses above 300 kPa. The excess pore pressure behavior in a GCL is essentially unknown for conditions between these two extremes.

The design engineer should carefully consider the various shear conditions that a clay or GCL may face and use a strength appropriate to that condition. This will require the engineer to instruct the laboratory on appropriate shear rates for interface and internal strength testing of conditions involving clays.

TEST SPECIFICATIONS

Most interface and internal shear testing performed today is related to landfill design and construction. Most of it is done to meet the requirements of a project specification or a government regulation. Many of these specifications contain insufficient information for the laboratory to determine how the test is to be run. Lacking this information, the laboratory may underestimate the effort, time and cost required to do the work properly. Clear and complete specifications that define the testing conditions to meet the requirements and intent of the design engineer are imperative. The laboratory cannot be left to choose important matters like hydrating times, consolidation stresses and times and shear rates because they do not know the design intent.

<u>The design engineer must specify the conditions for the test to the laboratory.</u> Unfortunately, the testing laboratory's client often does not know the conditions for which the test results will be used. For many laboratories, their immediate client is a contractor who is requesting a test because the specifications require him to. Most contractors don't have sufficient knowledge of these materials to specify test conditions. Ultimately it is the Engineer who requires these data. It is important that the Engineer includes this information in the specifications to inform the Contractor of what will be required and to give the Contractor a basis on which to instruct the laboratory what to do. Simply specifying that a strength test be performed according to ASTM D-5321 is not sufficient. This situation can become further complicated when other parties unfamiliar with this technology challenge various aspects of the testing protocol.

Engineers using geosynthetic materials need to become more familiar with their strength behavior so that they can include specific and meaningful requirements in their specifications. It is helpful to resort to one simple guiding principle when defining specifications for interface and internal shear strength testing – have the laboratory test follow the field conditions assumed in the design as closely as possible. This is the same guiding philosophy as the Stress Path Method developed by Lambe and Marr (1979) to determine material properties for soil.

The following phrases represent some typical language that if included in project specifications would make it clearer what the lab is to do and reduce the opportunity for disputes and delays.

> "A series of interface shear tests shall be performed according to ASTM D5321 (or a series of internal shear tests shall be performed according to ASTM D6423). A series shall consist of three separate tests using normal stresses of ......(specify the values).

"The geosynthetic materials should be attached to the shear box in a manner than causes uniform displacement to occur over all parts of the shear plane."

"Soil should be equilibrated to a moisture content of .... and compacted to a dry density of ...."

"A 144 psf (or other value supplied by Engineer) seating load shall be applied and the specimen shall be inundated with (tap water, leachate, special fluid) with the height of fluid a minimum of 10 mm above the top of the intended shear plane. These conditions shall be maintained for a period of (15 minutes, 1 hour, 24 hours, 7 days, 14 days, etc.) to allow the test specimen to come to complete moisture equilibrium. [Or, vertical displacement of the specimen shall be monitored during hydration and hydrating conditions be maintained until the measured vertical displacement shows that the specimen has reached moisture equilibrium]"

"After hydration the test specimen shall be consolidated to the required normal stress for shearing. The normal stress shall be applied in increments by doubling the stress in each increment. Each increment shall be maintained for enough time to allow the test specimen to consolidate by at least 50%. The final increment shall be maintained long enough to allow complete consolidation of the materials. The degree of consolidation shall be determined by measurement of change in height, separate consolidation tests, or other data on the consolidation behavior of the materials. "

"The specimen shall be sheared at a rate of ....(or ...a rate determined by the ASTM procedure based on measurements of the consolidation time of the materials.)"

"The specimen shall be sheared to a maximum displacement of 3 inches (75 mm) or until the shear stress remains constant for a displacement of at least 0.5 inches (12 mm)."

In addition to these requirements the specifications should be clear as to the specific interfaces that must be tested and the numbers of samples to be tested. It is important that the specifications include consideration of the time required to complete these tests. Many contractors have no idea that some of these tests may take weeks to complete. This can become a major problem if the test results do not meet the specified values. Clear guidance in the specifications can help avoid much agony later on.

I also urge engineers to avoid statements like "the material shall have a friction angle greater than .... degrees.) Some conditions may produce a high strength that consists of a cohesive component and a frictional component. The friction angle may be lower than the required value but the material still has adequate strength for stability due to the high cohesion. A better requirement would be one of the following:

"The material shall have a strength greater than that represented by a strength envelope defined by a cohesion of 0 psf and a friction angle of 13 degrees when tested with the prescribed conditions over a normal stress range of 1,000 to 5,000 psf."

"The material shall have the following peak strengths for the indicated normal stresses: 360 psf at 1,000 psf, 910 psf at 2,500 psf and 1,820 psf at 5,000 psf. The material shall have the following post-peak strengths for the indicated normal stresses: 210 psf at 1,000 psf, 530 psf at 2,500 psf and 1,060 psf at 5,000 psf."

The numerical values in this requirement should be replaced with values for specific project conditions.

Some projects are specifying a test involving multiple interfaces within the same setup (a "sandwich" or "floating" test). The aim is to simulate the actual field conditions and to reduce the number of tests that have to be run. This greatly complicates running and interpreting a test that is already complicated enough. Such tests should only be run by personnel highly experienced in running and interpreting interface shear strength tests. Measurements of relative displacement of the individual layers must be taken during the test to determine which layer initiates shear. Careful study should be made of the individual components as the test setup is dismantled to determine where failure occurred. Failure may occur at different interfaces for different normal stresses. Strength at large displacements from this setup may not be representative of the material with the lowest residual strength because the failure was initiated through a different interface. The critical interface determined in this test should be tested in a separate test of just that interface to determine strength values for design. It will be difficult for another lab to duplicate the results of a test of this type. Too many things may go wrong to rely on a sandwich test for most situations.

Results from interface and internal shear testing should be interpreted by the person who required the tests. Only he or she knows the particular application and the assumptions used in the design that depend on the strength values. The laboratory's responsibility is to run the tests as specified, record data accurately and completely, and provide any related observations that may assist or affect the interpretation of the data.

CLOSING

Interface and internal shear testing for geosynthetic materials has improved significantly over the past ten years. Qualified laboratories can provide reproducible and consistent data with sufficient accuracy for design. However, there are many important testing details that can have a major influence on the test results. Unqualified laboratories can miss these details without knowing they have done so. Inexperienced engineers may not recognize the potential problems. The consequence may be delays of weeks or months to a project while the problems are sorted out. Engineers should provide specific requirements in their specifications for these test details to help reduce some of the problems that are occurring.

Interface and internal shear strength have a major impact on the design of many landfill facilities. There remain a number of questions about which we have little to no information. Designers should be aware of these questions. Hopefully, additional research will find some answers. These include the following.

<u>Behavior at high normal stresses</u> – Most of our experience is limited to normal stresses up to about 10,000 psf (500 kPa). Normal stresses may exceed this value in landfills greater than about 150 ft (50 m) in height. Limited evidence indicates that the strength envelopes for internal and interface strength are curved with slopes that decrease with increasing normal stress. This means that linear extrapolation of strength data to higher stresses may significantly overestimate the actual strength. As mega landfills approach 400-500 ft (120-150 m) in height, the strength behavior at higher normal stresses becomes a major uncertainty.

<u>Undrained and drained behavior</u> - As described in the beginning of this paper, excess pore pressure may develop in soils and GCLs and affect their strength behavior. Little is known about pore pressure behavior in these conditions. Some field loading conditions produce excess pore pressures that should be considered in design. More experimental work that measures excess pore pressures on the shear plane is needed.

<u>Creep behavior of GCL</u> – Long-term behavior at shear stress levels above half of the peak strengths for conditions involving internal strength of GCLs is not well defined. Much of the peak strength of a GCL comes from the tensile strength of the plastic fiber reinforcement as demonstrated by the large difference between peak and residual strength of this material. It is well established that creep reduces the long-term strength of plastics. Limited data show that GCLs do not lose strength due to creep for shear stresses up to 50% of their short-term strength (Koerner et al, 2000) However, as these materials are used in applications with higher stress levels, creep effects on the internal strength of GCLs will become an important consideration.

<u>Index tests for internal strength of GCLs</u> – Internal shear strength tests may take a long time to run properly. Since a major part of a GCL's peak strength comes from the plastic fiber reinforcement, it seems reasonable to expect the peak strength to be related to the force required to peel the top geotextile from the bottom geotextile. More work is needed to develop this relationship to the point that peel strength might become a useful indicator of internal strength.

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