State-of-the-art report: GCL shear strength and its measurement

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ABSTRACT: This paper presents a comprehensive source of information on the shear strength and shear strength testing of geosynthetic clay liners (GCLs). Essential concepts of shear stress–displacement behavior and shear strength interpretation are presented, including long-term performance issues, followed by detailed discussions on the laboratory measurement of the shear strength of GCLs and GCL interfaces. The paper also provides recommendations for the selection of design failure envelopes for stability analyses and checklists to assist users in the specification of GCL shear testing programs. North American practice is emphasized and discussions are focused primarily within the context of landfill bottom liner and cover systems. Conclusions are drawn with regard to GCL shear strength behavior and current GCL strength testing practice, improvements for GCL strength testing and reporting are suggested, and future research needs are identified.

KEYWORDS: Geosynthetics, Bentonite, Durability, Geosynthetic clay liner, Landfill, Shear strength, Shear test, Stability analysis

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1. INTRODUCTION

Internal and interface shear strengths of geosynthetic clay liners (GCLs) are needed for static and seismic stability analyses in the design of waste containment facilities and other facilities that incorporate these materials as hydraulic barriers. Particular attention is often given to these strengths because bentonite, the essential component of a GCL, is a very weak material after hydration and thus can provide a potential surface for slope failure. Reported values of GCL internal and interface shear strengths show significant variability due to variability in component materials and manufacturing processes, differences in testing equipment and procedures, and changes in the design, manufacture, and application of GCLs over time. As a result, it has long been recognized that design shear strength parameters for GCLs and other geosynthetics must be obtained using project-specific materials tested under conditions closely matching those expected in the field (Bove 1990; Eith *et al*. 1991; Gilbert *et al*. 1997; Koerner and Daniel 1993; Koerner *et al*. 1986). Shear strengths of GCLs and GCL interfaces are routinely measured using laboratory shear tests, and are dependent on many factors. Current understanding of the effect and importance of these factors has evolved over recent years, and new information on several issues (e.g. long-term performance), as well as a new US laboratory testing standard (ASTM D 6243), have become available. At present there is no single source that summarizes this information with commentary.

This paper presents a state-of-the-art report on the shear strength and shear strength testing of GCLs. Essential concepts of shear stress–displacement behavior and shear strength interpretation are presented, including long-term performance issues, followed by detailed discussions on the laboratory measurement of the shear strength of GCLs and GCL interfaces. The latter topic addresses assessment of shear test quality, specimen size, shearing devices, specimen gripping/clamping system, specimen selection and trimming, gap setting and multi-interface tests, normal stress selection and number of tests, hydration stage, consolidation stage, shearing stage, and final specimen inspection and water contents. The paper also provides recommendations for the selection of design failure envelopes for stability analyses and checklists to assist users in the specification of GCL shear testing programs. North American practice is emphasized and discussions are focused primarily within the context of landfill bottom liner and cover systems. Conclusions are drawn with regard to GCL shear strength behavior and current GCL strength testing practice, improvements for GCL strength testing and reporting are suggested, and future research needs are identified.

2. GCL PRODUCTS

A GCL is a manufactured hydraulic barrier consisting of bentonite clay bonded to a layer, or layers, of geosynthetic material. The first such products were developed in the early 1980s and consisted of sodium bentonite sandwiched between two woven geotextiles. The variety of GCL products has since greatly increased as manufacturers have attempted to improve performance and address specific applications. Nevertheless, all GCLs can be subdivided into unreinforced and reinforced products. Unreinforced GCLs contain no geosynthetic reinforcement across the bentonite layer, and therefore have shear strength equal to that of the bentonite. Unreinforced GCLs can be geotextile (GT)-supported, in which case the bentonite is contained by woven (W) and/or nonwoven (NW) geotextiles. An example of an unreinforced GT-supported GCL is the Claymax 200R® product manufactured by CETCO (Arlington Heights, IL, USA). Unreinforced GCLs can also be geomembrane (GM)-supported, in which case the bentonite is adhered to one side of a smooth geomembrane (GMS) or a textured geomembrane (GMX). An example of an unreinforced GM-supported GCL is the GundSeal® product manufactured by Gundle/SLT Environmental, Inc. (Houston, TX, USA). Encapsulated GCLs are constructed by placing a second GM over an unreinforced GM-supported GCL. Reinforced GCLs have also been encapsulated between two textured geomembranes in some applications. In this way, the bentonite is at least partially protected from hydration and will have a higher average shear strength than in the fully hydrated condition.

Reinforced GCLs are GT-supported and can be stitch-bonded (SB) or needle-punched (NP). Stitch-bonded GCLs contain parallel lines of stitching that run in the machine direction of the product and transmit shear stress across the geotextiles to resist shearing through the bentonite. An example of an SB GCL is the NaBento® product manufactured by HUESKER Synthetic GmbH (Gescher, Germany). Needle-punched GCLs contain fibers that extend from a NW GT, pass through the bentonite layer, and are anchored in a W or NW GT, forming a W/NW or a NW/NW product respectively. An example of a W/NW NP GCL is the Bentomat® ST product manufactured by CETCO (Arlington Heights, IL, USA). Some NP GCLs are subjected to a heat-bonding process that thermally fuses the needle-punched fibers to the anchoring geotextile. The line of Bentofix® Thermal Lock products is manufactured by Naue Fasertechnik GmbH (Lübbecke, Germany) using this process.

Other types of GCLs have been developed, including a GCL with an internal structure similar to a geonet that reinforces the bentonite and reduces lateral bentonite migration due to normal stress concentrations (Stark 1997, 1998) and a heat-bonded W/NW NP GCL with a polypropylene coating applied to the woven side (Lucas 2002). Both unreinforced GT-supported and NP GCL products are available with a thin (0.10 mm) mixed-density polyethylene membrane laminated to one side. These products are intended chiefly for high hydraulic gradient applications, such as pond liners. An NP GCL product is also available with a thicker (0.51 mm) high-density polyethylene (HDPE) GMX laminated to one side. More information on GCL product types can be found in Koerner (1998), Qian *et al.* (2002), commercial literature and websites of the various manufacturers, and the Specifier's Guide published annually by *GFR* magazine.

3. SHEAR STRESS–DISPLACEMENT BEHAVIOR

3.1. Shear stress–displacement relationships

Shear stress–displacement relationships for GCLs and GCL interfaces, as obtained from short-term shear tests, are used to determine shear strength parameters and to conduct stability analyses that yield estimates of displacement. Shear stress–displacement relationships can also provide an important indication of test data quality (see Section 6.2). Figure 1 shows a typical relationship between shear stress (τ) and shear displacement (Δ) as obtained from a laboratory direct shear test conducted on a hydrated GCL at constant shearing normal stress ($\sigma_{n,s}$). Shear stress increases rapidly to a peak shear strength (τ_p) at the beginning of the test. The corresponding displacement at peak (Δ_p) is usually, but not always, less than 50 mm. In general, values of Δ_p are smallest for unreinforced GCLs, larger for NP GCLs, and largest for SB GCLs. As displacement continues, all GCLs and most GCL interfaces experience post-peak strength reduction, in which the measured shear stress decreases and ultimately reaches a residual shear strength (τ_r), after which no further strength reduction occurs. The displacement associated with the residual

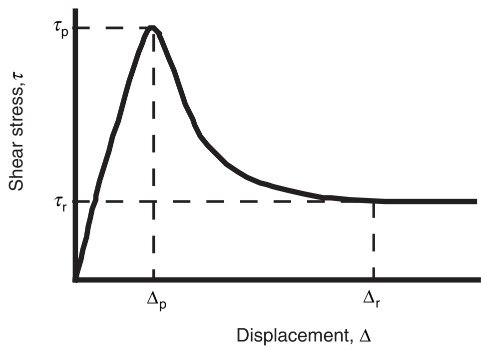


Figure 1. Typical shear stress–displacement relationship for an internal shear test of a hydrated GCL

strength (Δ_r) may be as large as 0.1–0.5 m or more, depending on the material(s) tested and the normal stress level. Residual shear conditions are best determined by plotting τ versus $\log \Delta$ to more clearly distinguish changes in τ (or the lack thereof) at large Δ (La Gatta 1970; Stark 1997). In cases where τ_r is not measured, a 'large displacement' shear strength (τ_{ld}) is often reported along with the displacement at which it was measured (e.g. a common notation is $\tau_{75\,mm}$ or τ_{75} for the shear strength at $\Delta = 75$ mm).

Post-peak strength reduction can result from several mechanisms, including clay particle reorientation at the failure surface, volume increase of material within the shear zone (e.g. soil), loss of roughness for interface geosynthetic materials (e.g. GMX), and failure of reinforcement or supporting geotextiles. Internal shear failure of NP GCLs generally occurs as the reinforcement fibers rupture and/or pull out of the geotextiles, whereas SB GCLs fail as the reinforcing stitches rupture or tear out of the geotextiles. Fiber pullout may be reduced if heat bonding or GM lamination is applied to the anchoring GT of an NP GCL. The residual strength ratio (τ_r/τ_p) for internal shear of GCLs varies widely, with reported values ranging from 0.04 to 1.0, depending on the product type and test procedure (e.g. hydration condition, magnitude of shear displacement). In general, τ_r/τ_p values increase in the following order: hydrated NP GCL < hydrated SB GCL < hydrated unreinforced GCL < dry unreinforced GMS-supported GCL < dry unreinforced GMX-supported GCL (Chiu and Fox 2004; Fox *et al.* 1998a). The term 'dry' denotes a GCL specimen that was tested in the as-received moisture condition.

3.2. Unreinforced GCLs

Two examples of shear stress–shear displacement (τ–Δ) relationships for internal shear of unreinforced GCLs, as obtained from direct shear tests, are shown in Figure 2a. The dry unreinforced GCL was encapsulated between two textured HDPE geomembranes, with the bentonite glued to the lower GMX ($\sigma_{n,s} = 96$ kPa, specimen size $= 300 \times 300$ mm, displacement rate = 1 mm/min). The second relationship was obtained for an unreinforced W/W GCL sheared in the fully hydrated condition ($\sigma_{n,s} = 72$ kPa, specimen size $= 406 \times 1067$ mm, displacement rate = 0.1 mm/min). The hydrated unreinforced GCL has low peak shear strength and $\tau_r/\tau_p = 0.4$. Hydrated unreinforced GCLs can sustain only small shear stresses without failure and are not appropriate for applications on slopes or applications on flat ground in which shear stresses are transferred from nearby slopes (Stark *et al.* 1998). The dry encapsulated GCL has much higher peak and residual shear strengths and a large displacement strength ratio of $\tau_{60}/\tau_p = 0.81$, indicating that significantly less post-peak strength reduction occurs in the dry condition. The high residual shear strength of a dry GCL is advantageous for designs in which the GCL is sheared beyond the peak (see Section 5). Values of Δ_p are relatively small (< 10 mm) for both unreinforced GCLs.

3.3. Reinforced GCLs

Needle-punched or stitched reinforcement is used to transmit shear stress across the weak bentonite layer of a hydrated GCL, with the needle-punched variety now being the more common choice in the US. The additional confinement provided by needle-punched fibers also decreases the water content of the hydrated bentonite and the potential for bentonite migration, although significant migration has been observed under severe loading conditions (Fox *et al.* 1996, 1998b, 2000; Stark 1998) and for one case study of a landfill liner slope (Stark *et al.* 2004). The peel strength test (ASTM D 6496) is routinely used as a quality control index test in the manufacturing of NP GCLs to assess the relative strength and density of fiber reinforcement.

Figure 2b shows examples of τ–Δ relationships for internal shear of a hydrated W/NW NP GCL and a hydrated W/W SB GCL ($\sigma_{n,s} = 72$ kPa, specimen size $= 406 \times 1067$ mm, displacement rate = 0.1 mm/min). These relationships display higher peak shear strengths than the hydrated unreinforced GCL in Figure 2a due to additional shear resistance provided by the geosynthetic reinforcement and lower residual strength ratios ($\tau_r/\tau_p = 0.06$ and 0.11 for NP and SB respectively) due to failure of the reinforcement. At this normal stress level, the SB GCL has a peak strength that is approximately one-half that of the NP GCL. The value of τ_r/τ_p for internal strength of hydrated NP GCLs can be as low as 0.04 (Fox *et al.* 1998a), indicating that reinforced GCLs can experience very large strength reduction if the peak strength is exceeded. Dry NP GCLs can also experience large post-peak strength reductions at low normal stress (Feki *et al.* 1997). In Figure 2b the SB GCL has a Δ_p value that is approximately twice that of the NP GCL. This is due to the ability of the supporting geotextiles to stretch around the lines of stitching prior to tearing of the geotextiles at the stitching (see Fuller 1995 for photograph of this effect). The essentially uniform reinforcement density of an NP GCL prevents this type of deformation, resulting in a lower Δ_p value.

3.4. Reinforced GCL interfaces

Shear stress–displacement relationships for four NP GCL interfaces are shown in Figure 2c. Three tests were performed with HDPE geomembranes and one was performed with silty sand. All peak interface strengths are smaller and all large-displacement strengths are larger than for the internal GCL shear tests shown in Figure 2b (at $\Delta = 200$ mm, τ_r for the GMS/NP GCL interface is also larger). The relationship for the GMS/NP GCL interface has the lowest τ_p and the highest residual strength ratio ($\tau_r/\tau_p = 0.82$), and is nearly independent of whether the W or NW side of the NP GCL is tested (Triplett and Fox 2001). As shown in Figure 2c, this independence does not hold for GMX/NP GCL interfaces. Peak and residual interface shear strengths for a GMX sheared against the NW side of an NP GCL are generally higher than those correspond-

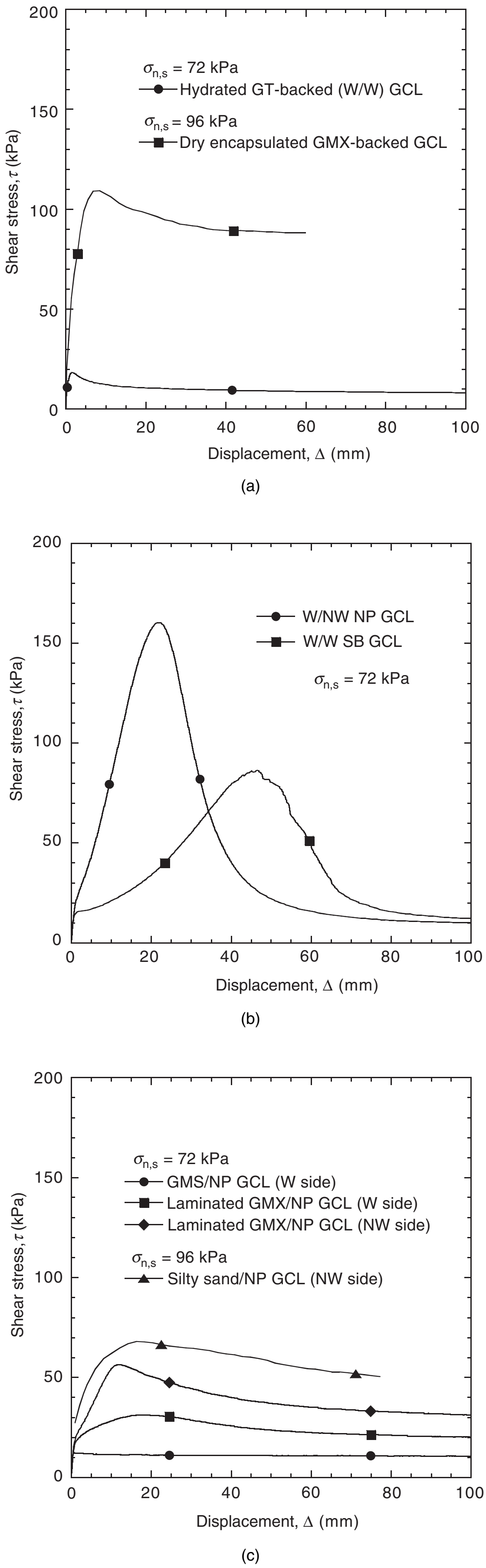


Figure 2. Examples of shear stress–displacement relationships for direct shear tests of: (a) unreinforced GCLs; (b) hydrated reinforced GCLs; (c) hydrated W/NW NP GCL interfaces

ing to the W side. von Maubeuge and Eberle (1998) also found that GMX/NP GCL (NW side) interfaces had higher shear strengths when the NP GCL was manufactured using a thicker NW GT. Differences in GM texturing process (e.g. laminated versus coextruded) have a relatively minor effect on GMX/GCL interface shear strength (Chiu and Fox 2004). Post-peak strength reductions are higher for GMX interfaces than for GMS interfaces due to higher levels of damage that occur during shear. Large displacement strength ratios for the GMX interfaces in Figure 2c are higher for the W side ($\tau_{200}/\tau_p = 0.57$) than for the NW side ($\tau_{200}/\tau_p = 0.47$). Although less published information is available, τ–Δ relationships for soil/GCL interfaces show considerable variability, depending on the soil type and method of preparation/compaction (Chiu and Fox 2004). The silty sand/NP GCL relationship in Figure 2c has an intermediate peak strength and moderate post-peak strength reduction ($\tau_{77}/\tau_p = 0.74$). Little to no post-peak strength reduction has been shown for shear tests conducted on dry sand/NP GCL interfaces (Garcin *et al.* 1995) and moist silty sand/SB GCL interfaces (Feki *et al.* 1997). Values of Δ_p for the GCL interfaces shown in Figure 2c are all less than 20 mm. In general, Δ_p values for most NP GCL interfaces are less than those for internal shear of NP GCLs (Chiu and Fox 2004).

4. SHEAR STRENGTH

4.1. Total versus effective normal stress

As with natural soils, shear stress–displacement relationships for GCLs and GCL interfaces are a function of the effective normal stress on the failure surface. Drained conditions are generally assumed for GCLs in stability analyses because, although field measurements have never been reported, excess pore pressures are assumed to be small. There is good justification for this assumption. Encapsulated GCLs remain essentially dry after installation except where GM defects, seam defects, or panel overlaps lead to local hydration (see Section 4.3). Hydrated GCLs and GCL interfaces are also unlikely to develop significant excess pore pressures after installation because: (1) GCLs are relatively thin and are often drained on at least one side; and (2) loading rates are typically slow relative to the rate of GCL consolidation (Gilbert *et al.* 1997). One possible exception is seismic loading.

GCL shear strengths from laboratory direct shear tests are expressed in terms of total normal stress on the failure surface. Thus the nature of pore pressure development during shear is a critical consideration. Internal shear of hydrated NP GCLs has been consistently observed to occur at a bentonite/GT interface (Eid *et al.* 1999; Fox *et al.* 1998a; Gilbert *et al.* 1996a). Fox *et al.* (1998a) measured negligible pore pressures on this interface when the GT was adjacent to a drainage surface. Triplett and Fox (2001) measured small positive pore pressures for GM/NP GCL interfaces at peak shear strength. Although pore pressure measurements in these studies can provide only qualitative trends (due to lack of backpressure and thus saturation), the data indicate that excess pore pressures on the failure surfaces were non-negative at peak strength and were small (positive or negative) at large displacements. Thus limited available information suggests that the current practice of characterizing GCL shear strength parameters in terms of total normal stress and then using these parameters for drained effective stress stability analyses is either appropriate or conservative.

4.2. Failure envelopes

Failure envelopes are prepared by conducting shear tests at various normal stress levels and plotting shear strength versus total shearing normal stress. As an example, Figure 3a shows τ–Δ relationships obtained from four internal direct shear tests of a hydrated W/NW NP GCL. The relationships are smooth and similar in shape, with both τ_p and τ_r increasing with increasing $\sigma_{n,s}$. Peak and residual shear strength failure envelopes, shown in Figure 3b, are slightly non-linear (i.e. curved) with stress-dependent tangent friction angles that decrease with increasing $\sigma_{n,s}$. In general, GCL internal peak strength failure envelopes are often non-linear, whereas peak strength failure envelopes for GCL interfaces can be linear, multi-linear (e.g. bilinear), or non-linear. Residual strength envelopes for GCLs and GCL interfaces are often nearly linear (Chiu and Fox 2004). Figure 4 shows several common models that can be used to characterize these envelopes. The normal stress range over which tests are conducted often dictates the degree of curvature in the resulting data and the appropriate model that should be used. Linear envelopes are the simplest and can have zero or non-zero intercepts. A multi-linear envelope, consisting of two or more line segments, gives an abrupt change in friction angle at the intersection point(s) and may reflect the true nature of shear strength behavior in some cases. Non-linear envelopes show a gradual change in tangent friction angle as shearing normal stress increases and may or may not pass through the origin. Failure envelopes that pass through the origin (i.e. have zero cohesion) are typical of GCL interface shear strengths and internal shear strengths of unreinforced GCLs. It is often unclear whether peak strength failure envelopes for reinforced GCLs have non-zero cohesion because of difficulties with adequate specimen gripping surfaces at low normal stress (see Section 6.6). Interestingly, Chiu and Fox (2004) found that the non-linear regression failure envelope for a large database of NP GCL internal shear strengths yielded zero cohesion. This supports the hypothesis that entanglement of needle-punched fibers in the anchoring GT is essentially a frictional mechanism (Fox *et al.* 1998a; Gilbert *et al.* 1996a). The reinforcement connection for SB GCLs, on the other hand, is not frictional, and thus these products display significant cohesion at zero normal stress.

Depending on the materials involved, the failure mode of a test specimen can change as the shearing normal stress increases. The resulting peak failure

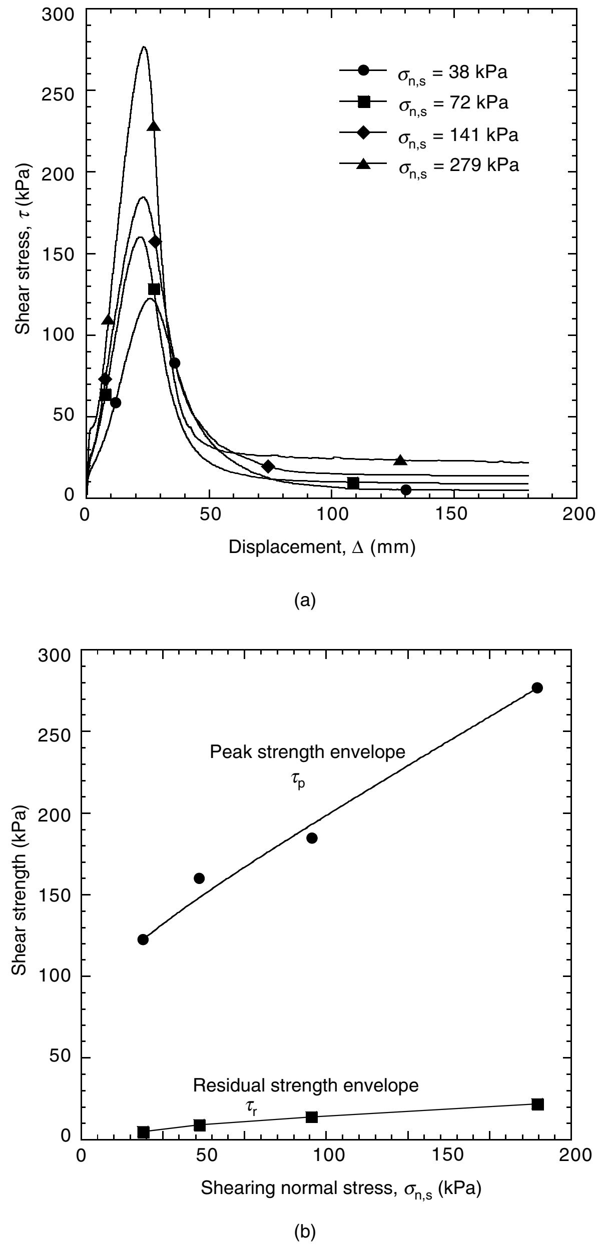


Figure 3. (a) Shear stress–displacement relationships for a hydrated W/NW NP GCL at four shearing normal stress levels; (b) peak and residual failure envelopes (Fox *et al.* 1998a)

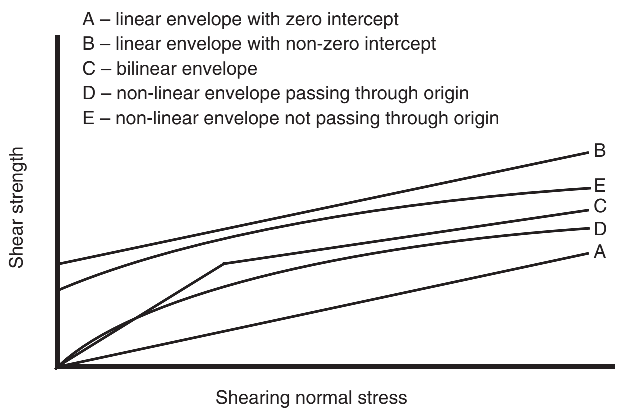


Figure 4. Typical failure envelopes for GCLs and GCL interfaces

envelope is often multi-linear and the residual failure envelope may contain one or more discontinuities. Byrne (1994) and Gilbert *et al.* (1996a) found that the shear of GMX/NP GCL (W side) specimens changed from interface to internal failure as the normal stress increased. The transition normal stresses were 96 kPa and approximately 15 kPa respectively. Using similar materials, however, Triplett and Fox (2001) found no such failure mode transition for normal stresses up to 279 kPa. Garcin *et al.* (1995) tested dry sand/NP GCL interfaces and reported interface failures for $\sigma_{n,s} < 100$ kPa and internal GCL failures for $\sigma_{n,s} > 100$ kPa. Clearly, the transition between failure modes and the normal stresses at which such transitions occur is highly dependent on the specific materials and testing conditions.

An example illustrating the effect of failure mode transition on peak and residual failure envelopes is presented in Figure 5 for a dry encapsulated GCL (Eid and Stark 1997). The bentonite was glued to a GMS and covered by a GMX in this study. Failure occurred at the bentonite/GMX interface for $\sigma_{n,s} < 65$ kPa and at the bentonite/GMS interface (adhesive failure) for $\sigma_{n,s} > 65$ kPa. As a result, the peak failure envelope is bilinear and the residual shear strength abruptly decreases at the failure mode transition due to lower shear resistance provided by the GMS. Similar failure mode transitions and failure envelopes were reported for dry GMX/GMX-supported and faille polyvinyl chloride (PVC) GM/GMS-supported encapsulated GCLs (Hillman and Stark 2001). Another particularly fine example of this effect is presented by Eid (2002). Multi-interface torsional ring shear tests were conducted on hydrated specimens of compacted silty clay overlain by an NP GCL, which was overlain by a GMX. For a shearing normal stress range of 17–400 kPa, shear strength of the composite liner system was controlled by three failure modes: soil/GCL at low $\sigma_{n,s}$, GMX/GCL at intermediate $\sigma_{n,s}$, and internal GCL at high $\sigma_{n,s}$. The resulting peak strength failure envelope was trilinear and the corresponding stepped residual strength failure envelope showed an abrupt shear strength reduction at each failure mode transition.

The familiar Mohr–Coulomb relationship is used to characterize linear and multi-linear failure envelopes. For internal shear strength, these relationships can be written as follows:

Peak internal failure envelope:

$$\tau_p = c_p + \sigma_{n,s} \tan \phi_p \tag{1}$$

Large displacement internal failure envelope:

$$\tau_{ld} = c_{ld} + \sigma_{n,s} \tan \phi_{ld} \tag{2}$$

Residual internal failure envelope:

$$\tau_r = c_r + \sigma_{n,s} \tan \phi_r \tag{3}$$

where c_p, c_{ld} and c_r are the peak, large displacement and residual cohesion intercepts, and ϕ_p, ϕ_{ld} and ϕ_r are the peak, large displacement and residual internal friction angles respectively, for each linear segment. The corresponding relationships for interface shear strength are mathematically identical, but are written using different notation for clarity:

Peak interface failure envelope:

$$\tau_p = a_p + \sigma_{n,s} \tan \delta_p \tag{4}$$

Large displacement interface failure envelope:

$$\tau_{ld} = a_{ld} + \sigma_{n,s} \tan \delta_{ld} \tag{5}$$

Residual interface failure envelope:

$$\tau_r = a_r + \sigma_{n,s} \tan \delta_r \tag{6}$$

where a_p, a_{ld} and a_r are the peak, large displacement and residual adhesion intercepts, and δ_p, δ_{ld} and δ_r are the peak, large displacement and residual interface friction angles respectively.

Although many GCL failure envelopes are non-linear, linear or multi-linear equations are commonly used to characterize these relationships for simplicity. In this case, c and ϕ (or a and δ) can only be fitted to a portion of the data and thus vary with $\sigma_{n,s}$. Figure 6 illustrates the most likely possibilities. Assume that the non-linear

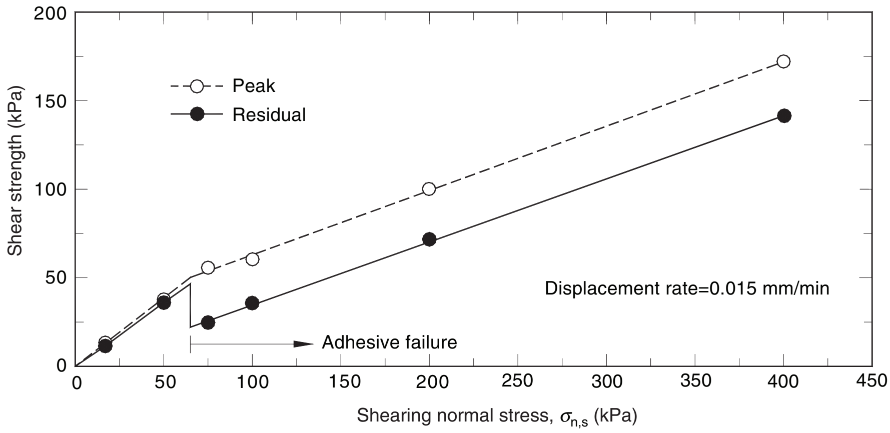


Figure 5. Peak and residual failure envelopes from torsional ring shear tests of a dry unreinforced GMS-supported GCL placed against a GMX (Eid and Stark 1997)

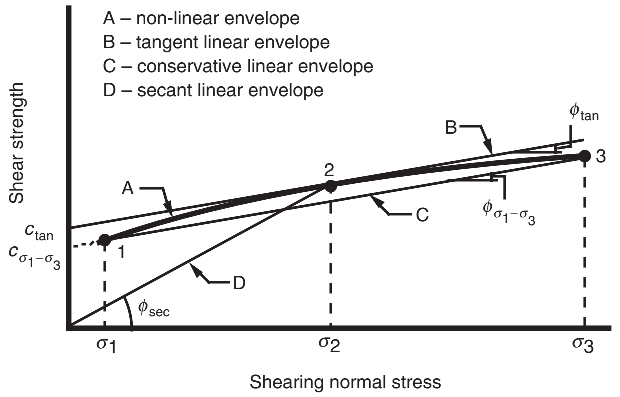


Figure 6. Linear characterization of a non-linear failure envelope

failure envelope (A) extending from point 1 to point 3 represents the actual material behavior and that the internal shear strength at shearing normal stress σ_2 is needed for design. A tangent linear failure envelope (B) is drawn at point 2 with friction angle ϕ_{tan} and intercept c_{tan}. For all shearing normal stresses except σ_2 envelope B overestimates the actual shear strength and is unconservative. Linear envelope C is drawn between points 1 and 3 and has friction angle $\phi_{\sigma_1-\sigma_3}$ and intercept $c_{\sigma_1-\sigma_3}$. Envelope C underestimates the shear strength at σ_2 and represents a conservative fit to envelope A for the normal stress range σ_1 to σ_3. If, however, envelope C is extrapolated outside this normal stress range, shear strengths will probably be overestimated. Another possibility is to define a secant friction angle (ϕ_{sec}) from the origin to point 2. The value of ϕ_{sec} bears no resemblance to the actual material friction angle and is intended to be used only with the shearing normal stress for which it is defined (σ_2). Finally, a bilinear envelope (not shown) can be fitted between points 1, 2 and 3, which will provide a good, yet slightly conservative, approximation to the data. Caution should always be exercised when linear (or multi-linear) equations are used to characterize non-linear shear strength data to avoid overestimating the actual shear strength. In an attempt to limit the misuse of GCL shear strength parameters in practice, Stark (1997) suggested that subscripts be added to reflect both the level of shear displacement and the applicable range of shearing normal stress. Examples of this notation for friction angles corresponding to peak, $\Delta = 50$ mm, and residual conditions and a $\sigma_{n,s}$ range of 100–300 kPa would be $\phi_{p,100-300\,kPa}$, $\phi_{50\text{ mm},\,100-300\text{ kPa}}$ and $\phi_{r,100-300\text{ kPa}}$ respectively.

A variety of non-linear equations can be used to characterize non-linear failure envelopes and thus avoid errors associated with fitting linear equations. Gilbert *et al.* (1996a) used the following equation from a hyperbolic stress–strain model (Duncan and Chang 1970) to characterize internal and GM interface shear strengths for an NP GCL:

$$\tau_p = \sigma_{n,s} \tan\left[\phi_o + \Delta\phi \log\left(\frac{\sigma_{n,s}}{P_a}\right)\right] \tag{7}$$

where ϕ_o and $\Delta\phi$ are constants determined from regression analysis and P_a is atmospheric pressure. Although undefined at the origin ($\sigma_{n,s} = 0$), Equation 7 can provide a satisfactory fit at low $\sigma_{n,s}$ values. Fox *et al.* (1998a) and Thiel *et al.* (2001) used the following equation for a *p*-order hyperbola with non-orthogonal asymptotes proposed by Giroud *et al.* (1993):

$$\tau_p = a_\infty + \sigma_{n,s} \tan\delta_\infty - \frac{a_\infty - a_o}{\left(1 + \frac{\sigma_{n,s}}{\sigma_o}\right)^p} \tag{8}$$

where a_∞, δ_∞, a_o, σ_o and p are constants. Although Equation 8 provides a general characterization of non-linear failure envelopes, a larger number of data points and possibly regression analysis (although Giroud *et al.* describe a simpler fitting technique) are needed to use this equation. Non-linear models may provide a better characterization of shear strength than linear models for certain GCLs and GCL interfaces. However, extrapolation of a non-linear model outside the stress range for which it was developed is not recommended. Such attempts may result in an overestimate or underestimate of the actual shear strength of the material. Shear strength parameters for both linear and non-linear equations can be determined through regression analysis. For this method to be accurate, the reliability of each data point should be approximately the same (i.e. no data from erroneous tests are included). More conservatively, an equation can be fitted as a lower bound to the data points.

An alternative to all the above equations and fitting methods is simply to use the failure envelope described by the test data directly for stability analysis (Stark *et al.* 2000). Most slope stability software programs allow a user to enter as many as 20 combinations of normal and shear stress to describe a failure envelope. The programs then linearly interpolate between these data points as needed to determine shear resistance for the normal stresses encountered along the failure surface. If the shear strength data points display a smooth trend, they can be entered directly and the method is straightforward. More difficulty is encountered if the data points display significant variability. For example, some variability is indicated for the peak strength envelope in Figure 3b, and is probably due to differences in needle-punched fiber density for the test specimens. In such cases, direct interpolation between the data points will produce a failure envelope with undulations that are not representative of the average material behavior and may introduce unwanted irregularities into a stability analysis. Thus a smooth fit through the data points is needed in some cases. The test data should be carefully reviewed to determine an appropriate smooth failure envelope. Coordinates representing the smooth envelope can then be entered into a slope stability software program instead of the actual test data points.

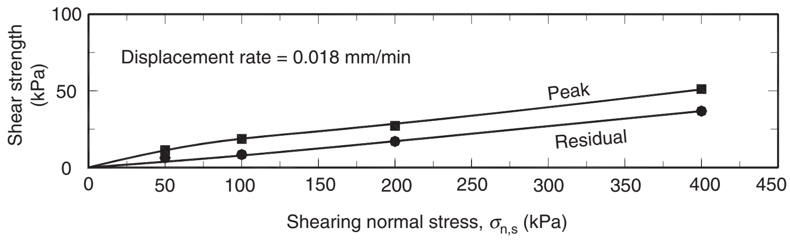


Figure 7. Peak and residual failure envelopes for a hydrated unreinforced GM-supported GCL

4.3. Unreinforced GCLs

The drained shear strength of hydrated sodium bentonite is the lowest of any natural soil (Mesri and Olson 1970). Figure 7 shows peak and residual failure envelopes obtained from torsional ring shear tests on a hydrated unreinforced GM-supported GCL. The friction angles are approximately $\phi_p = 8^\circ$ and $\phi_r = 5^\circ$. Fox *et al.* (1998a) measured similarly low friction angles ($\phi_p = 10.2^\circ$ and $\phi_r = 4.7^\circ$) for a hydrated unreinforced GT-supported GCL (Figure 8). These values of ϕ_r are in good agreement with the value of 4.0° measured from ring shear tests on sodium montmorillonite (Müller-Vonmoos and Løken 1989).

Encapsulating unreinforced bentonite between two geomembranes will significantly reduce the amount of bentonite hydration, resulting in higher shear strength and lower susceptibility for bentonite migration (Stark 1998; Thiel *et al.* 2001). Chiu and Fox (2004) showed that dry unreinforced GMX-supported GCLs generally have slightly lower internal peak strengths and much higher residual strengths than hydrated NP GCLs. The main design issue for unreinforced encapsulated GCLs thus becomes the amount of bentonite hydration that is expected, on average, as a result of liquid transmission through perforations, bad seams, and seam overlaps in the encapsulating geomembranes (diffusion of water vapor is expected to be negligible). Lateral moisture flow along GM wrinkles may also contribute to bentonite hydration (Cowland 1997). Thiel *et al.* (2001) and Giroud *et al.* (2002) presented theoretical analyses of long-term hydration of encapsulated bentonite for GM-supported GCLs. For landfill liner systems with 300 mm GM overlaps, Thiel *et al.* (2001) calculated that approximately 10–35% of the encapsulated bentonite will become hydrated over a design period of 250 years, depending on the moisture condition of the subgrade. Stability analyses for such a system are then conducted using pro-rated peak and residual strength envelopes based on the estimated ratio of dry and hydrated areas for the encapsulated GCL.

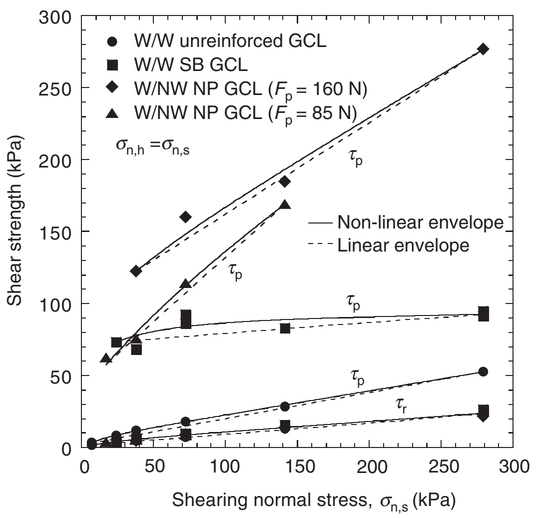


Figure 8. Peak and residual failure envelopes for hydrated unreinforced, stitch-bonded and needle-punched GCLs (Fox *et al.* 1998a)

4.4. Reinforced GCLs

Geosynthetic reinforcement greatly increases the peak shear strength of hydrated GCLs. Figure 8 shows peak and residual failure envelopes for a W/W SB GCL and two W/NW NP GCLs. The NP GCL specimens were taken from two rolls of the same commercial product having peel strengths (F_p) of 85 N/10 cm and 160 N/10 cm. Corresponding failure envelopes for an unreinforced W/W GCL are also shown for comparison. All GCL specimens were hydrated under the shearing normal stress using the two-stage accelerated hydration procedure described in Section 6.10.4. Each failure envelope is modestly non-linear and can be characterized by Equation 8 (Fox *et al.* 1998a). A linear envelope was also fitted between the endpoints of each non-linear envelope, as shown in Figure 8. The unreinforced GCL has the lowest peak strength at any normal stress and the linear failure envelope can be characterized by $c_p = 2.4$ kPa and $\phi_p = 10.2^\circ$. The peak shear strength of the SB GCL increases slightly with normal stress for $\sigma_{n,s} < 72$ kPa and is nearly constant at 91 kPa for $\sigma_{n,s} > 72$ kPa. The peak shear strength of the NP GCL increases sharply with $\sigma_{n,s}$ and shows good correlation with peel strength. Values of shear strength parameters (linear envelope) for the 85 N product are $c_p = 42.3$ kPa and $\phi_p = 41.9^\circ$, whereas values for the 160 N product are $c_p = 98.2$ kPa and $\phi_p = 32.6^\circ$. This finding is generally consistent with the work of Heerten *et al.* (1995) and von Maubeuge and Eberle (1998), in which internal stability of NP GCLs for a given slope angle and soil cover depth was directly related to peel strength. The residual failure envelope for each GCL product in Figure 8 is independent of reinforcement type and essentially equal to that of hydrated bentonite ($c_r = 1.0$ kPa, $\phi_r = 4.7°$). Thus the residual shear strength of hydrated GCLs can only be improved by increasing the residual shear strength of the hydrated bentonite. One possibility might be to incorporate a granular admixture (e.g. sand) into the bentonite layer (Fox 1998). In a related study, Schmitt *et al.* (1997) found that the peak shear strength ($\Delta \leq 10$ mm) of sodium bentonite can be increased by mixing it with granular expanded shale. The practicality of maintaining a sufficiently uniform mixing process on a production scale, such that GCL hydraulic conductivity remains uniformly low, is doubtful, however.

In the Fox *et al.* (1998a) study, the contribution of stitched reinforcement to peak strength of the SB GCL was essentially independent of $\sigma_{n,s}$ and solely dependent on the tearing strength of the woven geotextiles. Thus the increase of τ_p with $\sigma_{n,s}$ for the SB GCL (Figure 8) was due to increased shear strength of the bentonite/W GT interface. The contribution of needle-punched reinforcement to peak strength of the NP GCL increased linearly with $\sigma_{n,s}$ and displayed a clear correlation with peel strength. This lends further support to the concept that needle-punched fiber connections are frictional in nature (see Section 4.2).

4.5. Other sources of shear strength data

The complete presentation and interpretation of available test data on the peak, large displacement, and residual shear strengths of GCLs and GCL interfaces, as well as Δ_p and τ_r/τ_p values, is beyond the scope of this paper. Fox *et al.* (2002) and Chiu and Fox (2004) present findings from a large database of unpublished and published test data that has been compiled on the internal and interface shear strengths of unreinforced and NP GCLs. McCartney *et al.* (2002) also present findings from a large database of GCL internal and interface shear strength data. Other sources of good-quality shear strength data include Gilbert *et al.* (1996a), Fox *et al.* (1998a), Thiel *et al.* (2001), and Triplett and Fox (2001).

4.6. Shear strength anisotropy

As for other geosynthetic products, the shear strength of a GCL or GCL interface may be different in the machine direction and the transverse direction (i.e. rotated 90°). This difference has no practical significance for simple, two-dimensional slope conditions because GCLs are always installed with the machine direction aligned to the slope direction. However, it may be necessary to measure GCL shear strengths in the transverse direction if three-dimensional failure effects or seismic loads are expected to mobilize strength in this direction. Although no information has been published on the shear strength of GCLs or GCL interfaces in the transverse direction, in-plane anisotropy is expected to be relatively small for internal strength of hydrated unreinforced GT-supported GCLs and NP GCLs. Internal shear strengths for dry GMX-supported encapsulated GCLs may show small directional effects due to anisotropy of the GM texturing.

A GCL or GCL interface may also have different peak shear strengths when sheared in opposite machine directions (i.e. rotated 180°). A particularly dramatic example of this effect is shown in Figure 9 for an SB GCL with a 101 single-thread chain stitch (Fox *et al.* 1998a). In this case, values of τ_p differ by a factor of 1.8 for specimens sheared in opposite machine directions. Triplett and Fox (2001) found that the shear strength of GMX/NP GCL interfaces varied, on average, by 12% for specimens sheared in opposite directions. These results suggest that it is necessary to first determine the weakest machine direction in shear (if one exists) for a given GCL or GCL interface. Once determined, remaining tests should be conducted in this direction as it would be difficult to ensure that GCL panels are consistently installed in the strong direction on all slopes (Smith and Criley 1995).

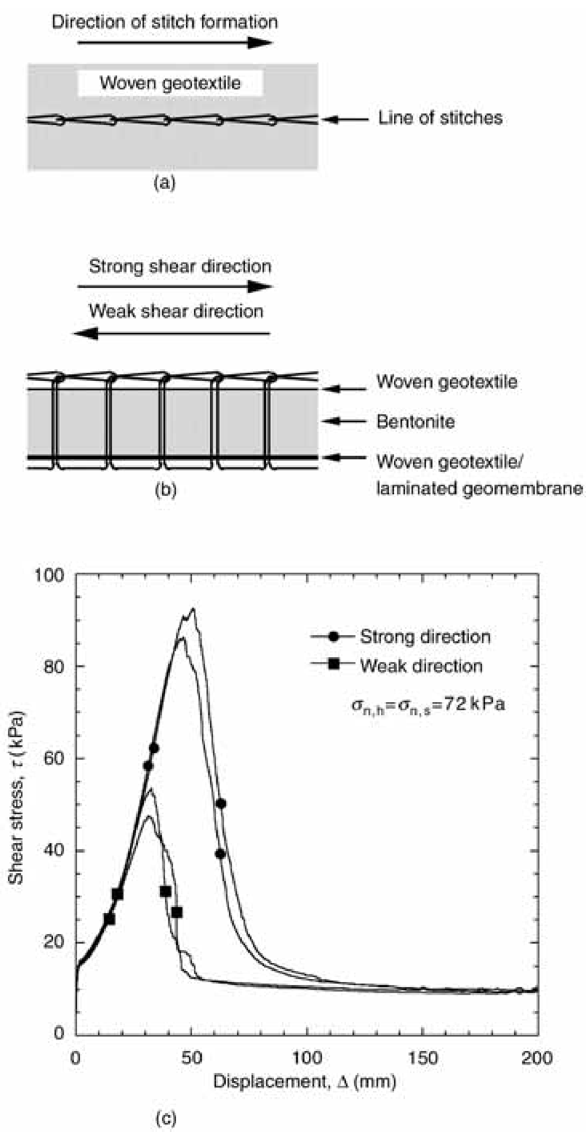


Figure 9. Shear strength anisotropy for an SB GCL: (a) plan view of stitching; (b) profile view of stitching; (c) stress–displacement relationships for hydrated specimens sheared in opposite machine directions (Fox *et al.* 1998a)

4.7. Effect of cyclic loading

Lai *et al.* (1998) have presented the only available information on the effects of cyclic loading on the shear strength of GCLs. They measured static and cyclic strengths of dry and hydrated specimens (dia. = 80 mm) of an unreinforced GM-supported GCL in direct simple shear. The dry product showed no strength reduction, and even a slight strength increase due to bentonite densification, under cyclic loading. When hydrated, GCL shear strength decreased under cyclic loading. Similar to natural soils, the number of cycles needed to cause failure decreased with increasing cyclic stress ratio (cyclic shear stress amplitude/static peak shear strength). No information is available on the performance of reinforced GCLs or GCL interfaces under dynamic loading conditions.

4.8. Long-term performance

4.8.1. *Importance of long-term effects*

The majority of research and essentially all design work involving shear strength of GCLs is based on data obtained from short-term strength tests (e.g. in which failure occurs in minutes to hours). However, in practice GCLs are expected to sustain shear loads over time periods ranging from years to centuries. The implicit assumption is that short-term strength data are relevant to the long-term stability of GCLs. The potential difficulty with this assumption is that the effects of GCL creep and GCL durability, both of which are central to the assessment of long-term stability, are not evaluated by short-term shear tests. Although relatively little research has been conducted on these issues, appreciation of their importance has grown in recent years. The development of a better understanding of long-term performance is one of the current pressing research needs for GCLs.

4.8.2. *Creep*

GCL creep is continuing shear displacement under constant normal and shear stress conditions. The creep stress ratio (or 'stress ratio') is defined as the applied shear stress divided by the short-term peak shear strength at the same normal stress. Creep occurs at a progressively decreasing rate for low stress ratios, ultimately leading to a stable condition. For high stress ratios, creep may begin to accelerate after a given time and lead to failure. The primary concern is that creep failure may occur for reinforced GCLs at a stress ratio less than 1 due to reinforcement fibers or yarns that elongate, break, or pull/tear out of the supporting geotextiles over time.

Relatively few creep tests have been conducted on GCLs because of the difficulty, time and cost involved. Koerner and Daniel (1993) reported that linear creep occurred for some types of hydrated NP GCLs at stress ratios less than 50%, and that SB GCLs were stable under similar conditions. Heerten *et al.* (1995) conducted a long-term inclined plane test on an NP GCL ($F_p = 30$N/10 cm, $\sigma_{n,s} = 25$ kPa) at a slope of 2H:1V and reported no failure for a test duration of 7500 h (313 days). Stable conditions were also reported by von Maubeuge and Eberle (1998) for a similar test on an NP GCL ($F_p = 29$N/10 cm, $\sigma_{n,s} = 52$ kPa) at a slope of 2.1H:1V for a test duration of 40 000 h (4.6 yr). Direct shear creep tests have been conducted by Siebken *et al.* (1997) and Trauger *et al.* (1997) for W/NW NP GCLs under incremental sustained loads. Trauger *et al.* also conducted an incremental-load creep test on an HDPE GMX/NP GCL (W side) interface. The stress ratio was 90–99% in the Siebken *et al.* study, and 23–70% in the Trauger *et al.* study. In both testing programs the materials experienced relatively small shear displacements and displacement rates decreased rapidly with time to a stable condition for each load increment. Koerner *et al.* (2001) conducted incremental-loading direct shear creep tests on one SB GCL and two NP GCL products. All three GCLs sustained stress ratios up to 60% without evidence of fiber pullout or breakage for a test duration up to 5000 h. Zanzinger and Alexiew (2000, 2002) reported the results of single-load and incremental-load direct shear creep tests on an SB GCL at $\sigma_{n,s} = 20$ kPa. Some of these data are presented in Figures 10 and 11. The short-term peak strength for the first and second series of tests was 67.6 kPa and 54.8 kPa respectively. Creep failure was not observed for stress ratios ranging from 45% to 95% and load durations up to 5000 h. Interestingly, at the end of the final single-load creep test, the stress ratio was increased from 90% to 110% over a 1740 h period without failure (Figure 11). The ability of the applied shear stress to exceed the short-term peak strength was attributed to stress redistribution within the reinforcement yarns over time. In a unique study, Thies *et al.* (2002) investigated the effect of elevated temperature (up to 80°C) on the creep rate of hydrated specimens of four non-commercial NP GCL products. Observed failure times were found to be strongly dependent on temperature and did not correlate with peel strength. This latter finding led Thies

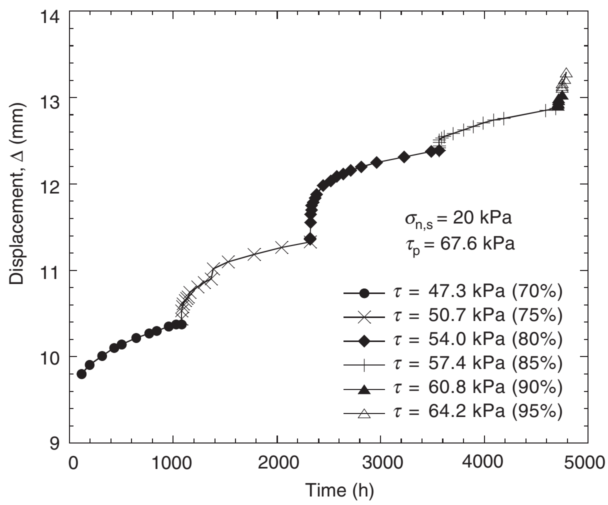


Figure 10. Shear displacement versus time for an incremental-load creep test of a hydrated SB GCL for stress ratio increasing from 70% to 95% (after Zanzinger and Alexiew 2002, used with permission)

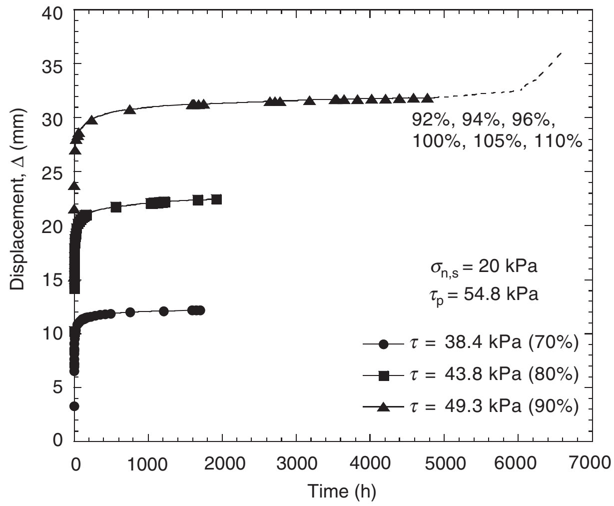


Figure 11. Shear displacement versus time for single-load creep tests of a hydrated SB GCL for stress ratios of 70%, 80% and 90%. The 90% test was then subjected to incremental loads up to 110% stress ratio (after Zanzinger and Alexiew 2002, used with permission)

et al. to conclude that short-term peel test data are not relevant for the assessment of long-term shear strength.

Zanzinger and Alexiew (2000) and Koerner *et al.* (2001) presented mathematical models for the extrapolation of GCL creep test results to long time periods. The necessary extrapolation is typically three or more orders of magnitude in time (e.g. 1000 h creep test to 100 yr GCL design life), and thus the predictions contain considerable uncertainty. More GCL creep data are needed to verify and calibrate these approaches.

4.8.3. *Durability*

Investigations of GCL durability (or 'aging') involve the change of GCL material properties over time, and constitute some of the most recent research on these products. Creep and aging are interrelated processes that occur simultaneously during the lifespan of a GCL. For the purposes of this report, durability concerns are limited to the degradation of polymeric materials, since bentonite cation exchange will only increase the shear strength of a GCL. Discussion is further restricted to the durability of reinforcement fibers that must sustain long-term tensile loads in NP GCLs, although it is recognized that stitch-bonding yarns also consist of individual fibers. Durability considerations for unreinforced GM-supported GCLs involve the durability of the carrier GM, and that discussion is left to the GM literature.

Hsuan and Koerner (2002) presented a comprehensive summary of the physical and chemical degradation mechanisms for polypropylene and polyethylene fibers, and suggested possible index and performance tests that can be used to measure such effects. The primary factors involved in fiber durability are stress level, environmental conditions (e.g. oxygen level), required lifetime, and polymer formulation details (e.g. type and amount of antioxidants). Fiber diameter is also an important factor for resistance to stress cracking (Seeger *et al.* 2002). Thomas (2002) measured long-term oxidation effects for a NW polypropylene GT, typical of those used in NP GCLs, in forced-air ovens at four temperatures (70–100°C) and for exposure times up to 400 days. Using a second-order kinetic model, the material was predicted to retain 50% of its strength for 30 yr at 20°C. A service lifetime approaching 100 yr was estimated for buried applications because the degradation rate in an 8% oxygen environment should be several times slower than the rate in air. Seeger *et al.* (2002) tested the degradation of over 500 individual polypropylene and polyethylene fibers in temperature-controlled (60 and 80°C) deionized water baths. Fiber rupture occurred after relatively small strains and load levels and often while creep rates were still decreasing. Hence times to failure could not be estimated using typical extrapolation methods for creep data. It was concluded that, as oxidation could be ruled out, environmental stress cracking was the most likely cause of fiber failure. This study illustrates the importance of performing long-term tensile tests in water for geotextiles with load-bearing fibers that will be in contact with water during their service life.

The above studies clearly indicate that more research is needed to identify the best testing methods to produce accelerated GCL polymer degradation that is most representative of long-term degradation in the field. Once the mechanisms and methods have been established, tests are needed to assess appropriate reduction factors for the prediction of long-term design shear strength of GCLs (see Section 4.8.5).

4.8.4. *Field tests*

Full-scale field tests and failure observations have played an important role in understanding the long-term internal and interface shear behavior of unreinforced and reinforced GCLs (Daniel *et al.* 1998; Feki *et al.* 1997; Stark *et al.* 1998; Tanays *et al.* 1994). The advantages of such tests are that GCL shear strength is mobilized under typical field conditions that may include the effects of geomembrane wrinkles, subgrade irregularities, panel overlaps, construction distress, changing climatic conditions (e.g. wet/dry, freeze/thaw), and bentonite hydration from humidity and native soil moisture. The disadvantages of field tests include the inability to closely control test conditions or to make precise load and displacement measurements for large test plots in the natural environment. For example, displacement measurement errors for wire extensometers used in field test plots were estimated as ±5 mm by Feki *et al.* (1997) and ±10 mm by Koerner *et al.* (1997). Thus the ability to obtain accurate creep measurements is questionable, and field test results are essentially limited to the obvious failure or no failure possibilities. The other main disadvantage is the high cost and time requirements for construction and monitoring of field test sections.

Tanays *et al.* (1994) and Feki *et al.* (1997) presented results for an SB GCL placed on 2H : 1V and 1H : 1V slopes at a municipal solid waste landfill in Montreuil/Barse, France. The subgrade soil was clayey and the

GCL panels were anchored at the top of each slope. GCLs for the 2H : 1V slopes were covered with 0.3 m of gravel or silty sand. Measured displacements were found to be small, and remained essentially unchanged for the 500-day period of observation. The GCL for the 1H : 1V slope was covered with a W GT and a silty sand layer (0.17 m thick) supported with geocells. One day after installation, GCL extension occurred at the top of the slope and the average strain was 5.5%. Measured displacements then decreased with time over the three-month observation period (no explanation is provided). Although the plot remained stable, it was concluded that partial failure of the GCL occurred at some measurement points due to the high tensile strain levels. Stark *et al.* (1998) presented a case study of a slope failure involving an unreinforced GM-supported (non-encapsulated) GCL in a landfill bottom liner system. It was concluded that failure occurred within the GCL due to hydration of the bentonite and the over-building of an interim landfill slope.

The Cincinnati, OH, USA, test plots have yielded the most significant information on field shear performance of GCLs (Daniel *et al.* 1998). Fourteen full-scale plots were constructed in November 1994 to test long-term internal shear strengths of unreinforced and reinforced GCLs on 3H : 1V and 2H : 1V landfill cover slopes. To date, all geosynthetic configurations on the 3H : 1V test slopes have performed satisfactorily, indicating that long-term shear stresses have not been problematic. Three failures occurred on the 2H : 1V slopes. The first two slides occurred 20 and 50 days after construction at the interface between SB and NP GCLs (woven slit-film GT in both cases) and an overlying HDPE GMX. The slides occurred without warning and were attributed to a reduction of GMX/GCL interface strength caused by time-dependent bentonite hydration from moisture in the underlying subgrade soil. A third slide occurred 495 days after construction as a result of an internal failure of an encapsulated unreinforced GMX-supported GCL. This slide was caused by unexpected bentonite hydration, possibly from edge drainage trenches or cuts made in the top GMX for instrumentation. The Daniel *et al.* (1998) study produced the following key observations:

- Interface shear strengths of reinforced GCLs were less than internal shear strengths for the low normal stress conditions tested.
- NW GT sides of reinforced GCLs had higher interface shear strengths than W GT sides when placed against a GMX.
- Hydrated bentonite migrated through the W GT sides of some reinforced GCLs and reduced GMX/GCL interface shear strengths over time.
- The 2H : 1V test plots were too steep to yield a safety factor that is normally considered adequate, and the 3H : 1V test plots yielded safety factors of at least 1.5 for conditions existing in the project.
- Observed failures and non-failures were consistent with limit equilibrium stability analyses using peak shear strengths obtained from short-term direct shear tests.

Based on this final observation, Daniel *et al.* (1998) concluded that the Cincinnati field test plots confirm the accuracy of current design methods, and thus field test sections should not generally be required.

4.8.5. *Long-term design strength*

Similar to soil reinforcement applications, the reduction in long-term shear strength due to creep and aging of reinforced GCLs can be addressed by performing long-term creep shear tests and developing strength reduction factors that are applied to short-term strength data. Marr and Christopher (2003) presented a conceptual approach for the estimation of long-term internal design strength of NP GCLs using such factors. It is assumed that creep and aging affect only the strength of the polymeric reinforcement, and that the difference between peak and residual GCL shear strengths is due solely to the presence of the reinforcement. Short-term peak and residual internal shear strengths are first obtained according to appropriate testing procedures. Residual GCL shear strengths can be estimated using $c_r = 0$ and $\phi_r = 4\text{–}5°$ (see Sections 4.3 and 4.4). At each normal stress level, reduction factors are applied to the difference between the peak and residual shear strengths, and this value is then added to the residual strength to give a reduced peak strength. In the absence of project-specific test data, Marr and Christopher recommend a reduction factor of 3 for creep and reduction factors of 1.1 and 2.0 for 100 yr and 300 yr of aging respectively. Marr and Christopher also note that temperature, normally assumed to be 20°C, has a strong effect on GCL creep behavior (Thies *et al.* 2002) and should be considered in selecting appropriate reduction factors. As an example, Figure 12 shows the long-term peak strength

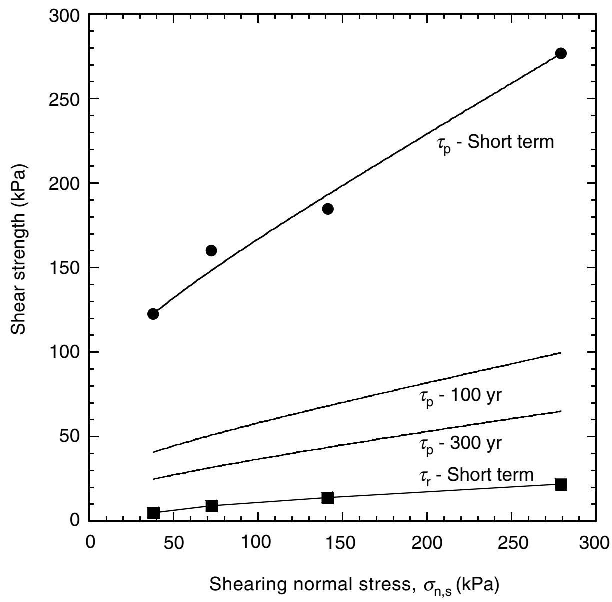


Figure 12. Measured short-term peak and residual internal failure envelopes for a hydrated W/NW NP GCL and calculated long-term peak internal failure envelopes using the method of Marr and Christopher (2003)

failure envelopes for a W/NW NP GCL at 100 yr and 300 yr calculated using the test data presented in Figure 3b. The above method considerably reduces peak strength values because of the high total reduction factors (3.3 and 6.0 for 100 and 300 yr respectively). Research is needed to determine whether such high factors are appropriate for reinforced GCLs.

5. SELECTION OF FAILURE ENVELOPES FOR DESIGN

5.1. Displacements in liner systems

Large displacements may occur within geosynthetic liner systems on landfill slopes due to construction activities (McKelvey 1994), thermal expansion/contraction, large strains needed to mobilize the passive resistance of a waste buttress on the base liner (Stark and Poeppel 1994), strain incompatibility between waste materials and geosynthetic interfaces (Eid *et al.* 2000; Reddy *et al.* 1996), earthquakes, waste placement procedures (Yazdani *et al.* 1995), and waste settlement (Long *et al.* 1995). These displacements can lead to progressive failure effects between the side slopes and base of a bottom liner system (Byrne 1994; Filz *et al.* 2001; Gilbert and Byrne 1996; Gilbert *et al.* 1996b; Stark and Poeppel 1994). Shear failure will occur at the interface with the lowest peak shear strength, not the interface with the lowest residual shear strength. Thus the residual strength of a GCL or GCL interface should be used for design only if the GCL or GCL interface exhibits the lowest peak strength in the system and it is anticipated that the corresponding displacement at peak (Δ_p) may be exceeded. The selection of design failure envelopes for a multi-layer system in which individual components display non-linear or discontinuous failure envelopes and post-peak strength reduction requires careful analysis, including consideration of the possibility of unrepresentative test data. This is discussed in the following sections for bottom liner systems and cover systems. Gilbert (2001) and Marr and Christopher (2003) also provide relevant discussion on the topic.

5.2. Bottom liner systems

The proper failure envelope for design of bottom liner systems that contain side slopes can be selected using the methodology suggested by Stark and Poeppel (1994):

1. Assign residual shear strengths to the side slopes and peak shear strengths to the base of the liner system, and satisfy a factor of safety greater than 1.5.
2. Assign residual shear strengths to the side slopes and base of the liner system and satisfy a factor of safety greater than 1.0. A safety factor of 1.1 should be satisfied if large displacement shear strengths are used instead of residual values.

This design methodology assumes that large displacements occur within a bottom liner system and failure progresses from the side slopes to the base liner. The main issue in applying this methodology involves the determination of which materials/interfaces in the system reach a residual shear condition and which do not. Because peak failure envelopes for geosynthetics are often non-linear, it may be necessary to construct combination design failure envelopes using segments from the individual failure envelopes of liner system components.

As an example, Figure 13 presents peak failure envelopes for three GMS interfaces that are assumed to constitute the weakest potential shear surfaces of a composite liner system:

- NW GT/GMS;
- clay/GMS; and
- geonet/GMS.

For $\sigma_{n,s}$ < 280 kPa, the geonet/GMS interface exhibits the lowest peak strength and is the critical interface. However, the clay/GMS interface is critical for $\sigma_{n,s}$ > 280 kPa. Therefore a combination design peak failure envelope, illustrated by the dashed lines in Figure 14, should be used to characterize the peak strength of the liner system. This envelope represents the lowest peak shear strength at each normal stress. Figure 15 shows the individual residual strength failure envelopes for the same interfaces and Figure 16 shows the combination design residual failure envelope (dashed) for the liner system. The combination design residual

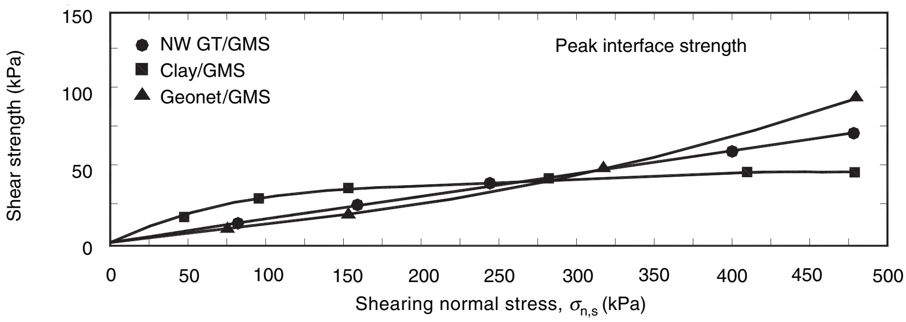


Figure 13. Peak failure envelopes for three interfaces of a composite liner system (Stark and Poeppel 1994)

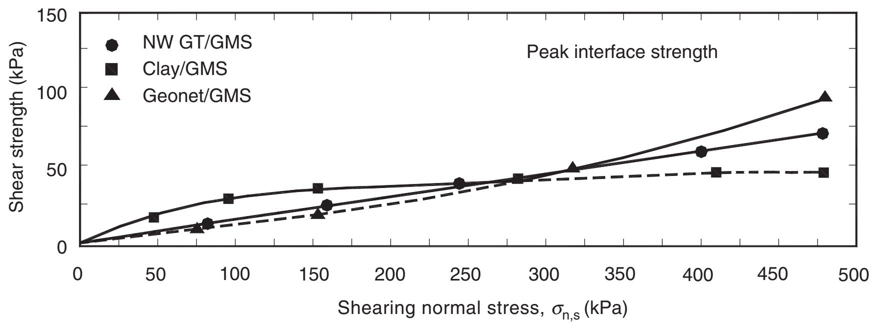


Figure 14. Combination design peak failure envelope for a composite liner system

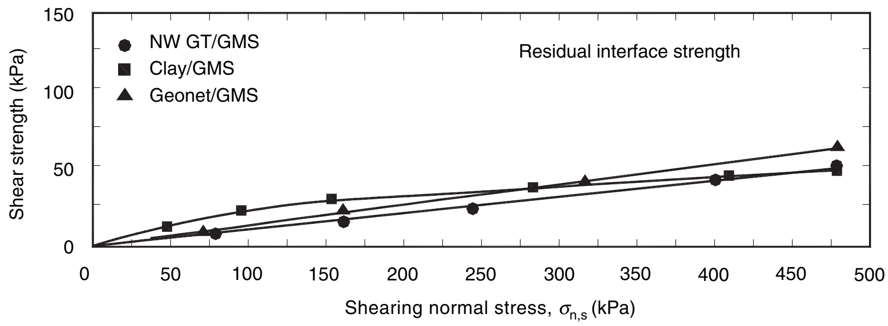


Figure 15. Residual failure envelopes for three interfaces of a composite liner system (Stark and Poeppel 1994)

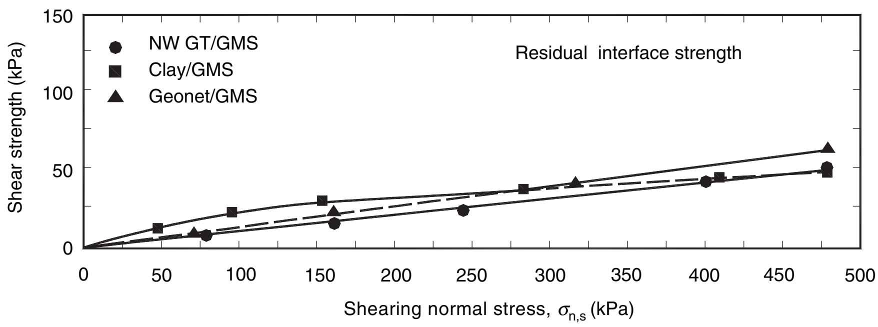


Figure 16. Combination design residual failure envelope for a composite liner system

failure envelope corresponds to the combination design peak failure envelope, and does not simply represent the lowest residual shear strength at each normal stress. Note that although the NW GT/GMS interface exhibits the lowest residual shear strength, this residual envelope is not used for design because the peak strength of the NW GT/GMS interface will not be exceeded (Figure 14), and thus a residual shear condition will not be reached along the NW GT/GMS interface. In summary, designers should not simply use the minimum residual failure envelope, but should determine which materials/interfaces will reach a residual shear condition and then use the corresponding combination residual failure envelope for design. As a note of caution, however, the type of analysis illustrated in Figures 13–16 is possible only if the various measured failure envelopes are representative. This requires quality, replicate shear testing using project-specific materials and conditions closely matching those expected in the field.

The proper selection of combination failure envelopes for design is particularly important when a bottom liner system contains a hydrated reinforced GCL because of its high peak strength and low residual strength. The residual failure envelope for any hydrated GCL will plot below the NW GT/GMS residual failure envelope in Figure 15. However, the peak strength envelope for hydrated and reinforced GCLs will likely be significantly higher than for many other interfaces in a liner system. If so, Δ_p of the GCL will not be exceeded, the GCL will not reach a residual shear condition, and the GCL internal residual envelope should not be used for side slopes (design steps 1 and 2) or the base liner (design step 2). Use of the GCL internal residual failure envelope under these conditions would be unnecessarily conservative. In this context, engineered 'weak-peak' interfaces having smaller τ_p and larger τ_r than a GCL (e.g. GT/GM, drainage geocomposite/GCL, sand/GMS) have been proposed as a means to increase the available residual strength within a GCL liner system and to contain shear displacement to an interface above the barrier layers (Gilbert 2001; Gilbert *et al.* 1996a; Luellen *et al.* 1999). In many cases, short-term peak strengths should be used for design of bottom liner systems because the majority of shear displacements probably occur during construction and filling, and because bottom liner slopes will often be buttressed by waste placement before long-term strengths are required.

5.3. Cover systems

The proper methodology for selection of the design failure envelope for cover systems is different from that for bottom liner systems because shear displacements are expected to be more uniform. Back-analyses of failures by the second author have indicated that peak strengths are mobilized throughout a cover system, largely because of the absence of waste placement, settlement, and buttressing effects. However, considerable shear displacements may occur in cover systems during construction. These displacements can be minimized by placing cover materials from bottom to top on slopes or by including veneer reinforcement (Koerner and Soong 1998). The stability of cover systems should be analyzed using the lowest available peak shear strengths, and the appropriate combination design peak failure envelope is constructed as shown in Figure 14. The only difference is that the normal stress range will be much smaller (possibly a single value), which may eliminate the need for a combination envelope. Long-term peak strengths are most appropriate in this case because cover systems must sustain permanent shear stresses.

6. LABORATORY MEASUREMENT OF GCL SHEAR STRENGTH

6.1. Role of laboratory shear tests

Considering the high cost and difficulty of conducting long-term tests, short-term laboratory shear tests are expected to remain the primary means by which shear strengths are obtained for GCLs and GCL interfaces. Reduction factors can then be applied to estimate long-term design strengths (see Section 4.8.5). Long-term laboratory shear tests and field performance tests will continue to be needed on a research basis to calibrate this design methodology.

6.2. Assessment of shear test quality

The results of shear tests on GCLs and GCL interfaces may be affected by many factors, including product type(s), product manufacturing conditions (e.g. new versus old needle boards), soil type and preparation conditions (if applicable), type of shearing device, equipment-specific factors (e.g. specimen gripping/clamping system), specimen size, hydration liquid, hydration procedure, consolidation procedure, drainage conditions, shearing normal stress range, direction of shear, shear displacement rate or constant applied shear stress, and maximum shear displacement. As improperly performed tests can give highly inaccurate results, it is important to carefully consider testing procedures and to examine test data for inconsistencies and potential flaws. Good-quality displacement-controlled shear tests will produce τ–Δ relationships that are generally similar in appearance to those shown in Figures 2 and 3a, and which exhibit smooth transitions from the start of loading to peak shear strength and then to large displacement/residual shear strength. Relationships obtained for replicate specimens should show good similarity as $\sigma_{n,s}$ increases (Figure 3a). Other examples of high-quality τ–Δ relationships are provided by Fox *et al.* (1998a) and Triplett and Fox (2001) for direct shear, and by Eid *et al.* (1999) for torsional ring shear. In contrast, Figure 17 shows τ–Δ relationships for a NW/NW NP GCL and a GMX/NP GCL (NW/NW) interface that suggest problems occurred during shear. These relationships display double peaks, poor similarity, and undulations that are non-physical. In addition, the internal shear relationships (Figure 17a) display high Δ_p values, unusually wide peaks, and an absence of post-peak strength reduction ($\sigma_{n,s} = 96$ kPa). The erroneous relationships in Figure 17 were probably caused by slippage due to poor specimen gripping surfaces (see Section 6.6). The resulting progressive failure effects will produce inaccurate (likely conservative) peak failure envelopes and inaccurate (likely unconservative) large displacement failure envelopes. Machine friction problems are another possible cause of erroneous τ–Δ relationships, and can result in unconservative peak and large displacement failure envelopes.

Examination of τ–Δ relationships is the best way to make a preliminary assessment of the quality of GCL shear test results. Currently, some production testing laboratories provide τ–Δ relationships along with peak and large displacement shear strengths, whereas other laboratories do not. It is recommended that τ–Δ relationships be routinely included as part of the test results package for GCL and other geosynthetic shear testing programs.

6.3. ASTM standard test procedure

ASTM D 6243 is the current standard test method for measurement of internal and interface shear strengths of GCLs in the US. This standard requires that GCLs be tested in direct shear with a minimum specimen dimension of 300 mm (square or rectangular specimens are recommended). The test specimen is sheared between two shearing blocks, each of which is covered with a gripping surface (i.e. rough surface) that transfers shear stress to the specimen. Clamping of geosynthetics at the ends of the shearing blocks is permitted to facilitate shearing at the desired location within the specimen. The gripping/clamping system should securely hold the specimen to the shearing blocks and not interfere with

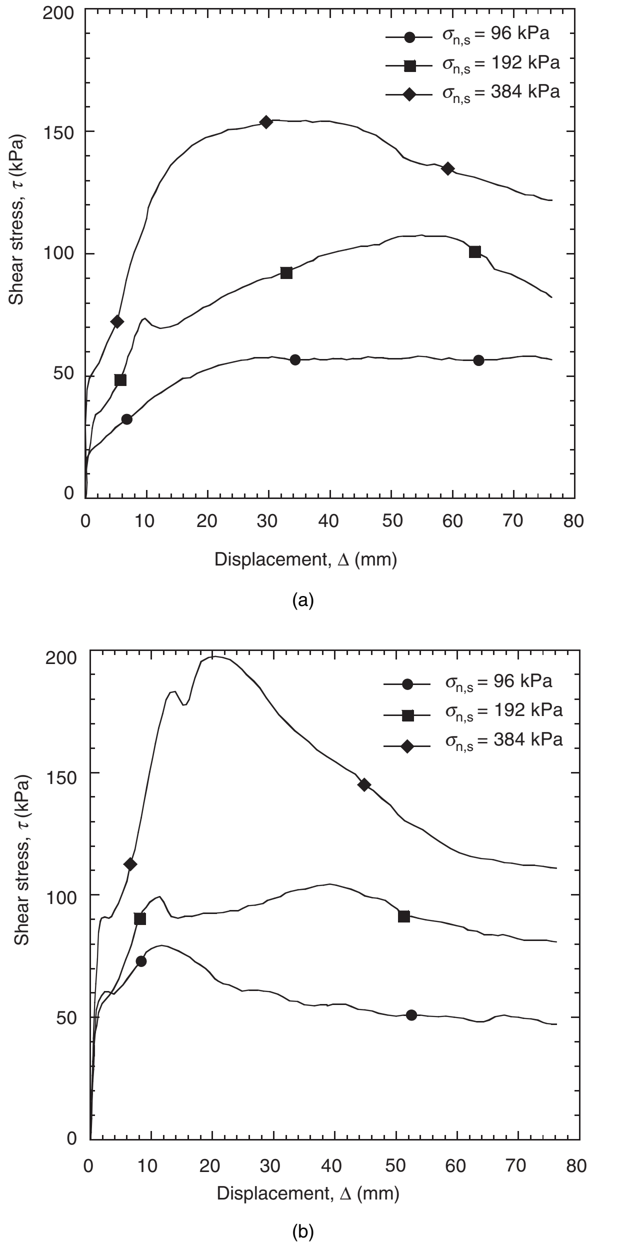


Figure 17. Examples of erroneous shear stress–displacement relationships obtained using a direct shear device: (a) hydrated NW/NW NP GCL; (b) hydrated HDPE GMX/NP GCL (NW/NW) interface

the measured shear strength. The gripping surfaces should also be rigid and permit free drainage of the specimen if necessary. Specimen conditioning procedures are specified by the user, including test configuration, soil compaction criteria (if applicable), hydration/consolidation procedures, normal stress level(s), and method of shearing. Specimens should be sheared to a minimum displacement of 50 mm using displacement-controlled (i.e. constant rate of displacement) or stress-controlled methods, the latter of which includes constant stress rate, incremental stress, and constant stress creep. Displacement control is needed to measure post-peak response. For displacement-controlled tests, ASTM D 6243 recommends the following equation (adapted from ASTM D 3080) to determine the maximum shear displacement rate, R:

$$R = \frac{\Delta_f}{50 t_{50} \eta} \tag{9}$$

where Δ_f is the estimated displacement at peak or large displacement shear strength as requested by the user; t_{50} is the time required for the GCL specimen to reach 50% consolidation (double-drained) under similar normal stress conditions; and $\eta = 1$ for internal shear of a GCL with drainage at both boundaries, 4 for interface shear between a GCL and an impermeable material, and 0.002 for interface shear between a GCL and a permeable material

If pore pressures are not expected to develop on the failure surface during a GCL interface shear test, ASTM D 6243 allows a maximum displacement rate of 1 mm/min. After shearing is completed, the failed specimen is inspected and the mode of failure is recorded.

6.4. Specimen size

The size of GCL specimens for internal and interface shear tests is almost always larger than for shear tests on natural soils. This is because:

- larger shear displacements are often required to reach peak strength and residual strength conditions;
- textural elements of many geosynthetics (e.g. geonet, GMX) are larger than for most soils; and
- the spacing of some types of GCL reinforcement (e.g. SB) may be as large as 100 mm.

Large specimens also tend to reduce edge effects and the effects of local variability in material strength (e.g. variations of needle-punched fiber density), making test results more reproducible. The disadvantages of shearing larger specimens are that tests are more difficult to perform, equipment is larger and more expensive, and the maximum possible normal stress may be reduced. For these reasons, Stark and Eid (1996) and Gilbert *et al.* (1997) recommended that shear tests performed on small specimens can be used to complement large-scale shear tests. Smaller specimens (100 mm × 100 mm) have also been recommended for shear tests on unreinforced GCLs (Zelic *et al.* 2002) and NP GCLs (Koerner *et al.* 1998). Olsta and Swan (2001) showed good agreement for internal shear strengths of a hydrated W/NW NP GCL obtained using large (300 mm × 300 mm) and smaller (150 mm × 150 mm) shear boxes. The smaller shear box was used to achieve very high normal stress conditions (1050–2800 kPa).

6.5. Shearing devices

6.5.1. *Direct shear*

Shear strengths of GCLs and GCL interfaces have been measured primarily using direct shear methods. The direct shear device has several advantages. First, shear occurs in one direction, which matches field behavior and is important for GCLs and GCL interfaces that display in-plane anisotropy. Second, direct shear test specimens can be relatively large (see Section 6.4). Third, shear displacement is theoretically uniform over the specimen, which tends to minimize progressive failure effects and allows for accurate measurement of peak shear strength. In practice, shear displacement may not be uniform if the gripping surfaces are inadequate (see Section 6.6). The primary disadvantage of the standard 300 mm × 300 mm direct shear device is that the maximum shear displacement (typically 50–100 mm) is not sufficient to measure the residual shear strength of most GCLs and GCL interfaces. Fox *et al.* (1997) developed a direct shear device capable of shearing very large GCL specimens (406 mm × 1067 mm). The maximum displacement of that device (203 mm) was sufficient to achieve residual internal shear conditions for GCLs (Fox *et al.* 1998a), but was insufficient to achieve residual shear conditions for HDPE GMX/NP GCL interfaces (Triplett and Fox 2001). Another disadvantage of the direct shear device is that the area of the failure surface decreases during shear, which may increase the shearing normal stress and require an area correction for data reduction. To avoid this problem, many direct shear devices have a top shearing block that moves across a longer bottom shearing block. However, this results in the movement of previously unconsolidated and unsheared material into the failure surface, which can also potentially alter the measured τ–Δ response. The large size of standard direct shear specimens also increases the possibility for errors in the applied normal stress. A rigid loading plate that uniformly compresses a GCL specimen will provide a near-uniform normal stress distribution. However, the accuracy of the total applied normal load should also be verified by the laboratory or certified by the manufacturer, especially for devices that use an air bladder loading system (Marr 2001). This can be accomplished by placing load cells between the shearing blocks at the center and corners of the shear box to measure the actual load applied to the specimen. A normal stress calibration device such as this is available from some manufacturers. A correction factor can then be calculated by comparing the actual load with the theoretical load based on bladder air pressure.

6.5.2. *Torsional ring shear*

The torsional ring shear device has also been used for shear tests of GCLs and GCL interfaces, primarily for research purposes (Eid and Stark 1997; Stark and Eid 1996). This device is capable of unlimited shear displacement and can be used to obtain residual shear strengths. Unlike direct shear, the area of the failure surface is constant during shearing and normal stress is typically applied using dead weights. The ring shear device also has several disadvantages. Because shearing occurs simultaneously on an annular surface, shear displacement is not in one direction. Measured shear strengths instead represent an average of local shear resistance for all in-plane directions, and will be affected if a GCL or GCL interface displays significant anisotropy. This limitation has not been found to be significant for NP GCLs because most needle punching appears to be isotropic (Eid *et al.* 1999). The small size of ring shear specimens prevents shear testing of SB GCLs (due to anisotropy and large reinforcement spacing) and may necessitate additional replicate shear tests to verify that measured strengths are representative. The circular geometry of the test tends to make specimen preparation procedures more complex than for direct shear. Finally, shear displacement is not uniform across the width of the specimen, which can cause different parts of the specimen to fail at different times during the test (i.e. progressive failure). In the ring shear device, progressive failure theoretically proceeds from the outer edge of the specimen to the inner edge and, for materials that display post-peak strength reduction, can reduce the measured value of τ_p. The error is a function of the diameter ratio (inside diameter/outside diameter) of the device. The measurement of τ_r is unaffected by non-uniform displacement across the specimen. For data reduction purposes, shear displacement is taken at the average radius of the specimen, and the average shear stress is calculated from specimen geometry and the applied moment (Bishop *et al.* 1971; Bromhead 1979). Values of τ_p measured from ring shear tests are usually in agreement with those measured from direct shear tests if the diameter ratio exceeds 0.7 (Stark and Poeppel 1995). Comparative tests on dry bentonite/GMX and hydrated GMX/NP GCL interfaces using ring shear and direct shear devices yielded τ_p values, but not τ–Δ relationships, that were in close agreement (Eid and Stark 1997; Stark and Eid 1996). The modified Bromhead ring shear device used in these studies had a diameter ratio of 0.4.

The experience of the second author indicates that the torsional ring shear device is easier to use and produces more consistent results than the ASTM D 6243 direct shear device. Because of the limitations of the ring shear device and the requirement of direct shear testing in ASTM D 6243, a combination of test methods is used at the University of Illinois. Direct shear tests are first conducted on a given GCL product or interface and are used to calibrate the results of ring shear tests. If the τ–Δ relationships are in agreement, the ring shear device is used for research and/or production testing.

6.5.3. *Inclined plane shear*

The inclined plane (i.e. tilt table) shear device has been used to measure shear strengths of geosynthetic interfaces, particularly in Europe (Briancon *et al.* 2002; Gourc *et al.* 1996; Briancon *et al.* 2002). However, few results have been reported for GCLs (Alexiew *et al.* 1995; Heerten *et al.* 1995; von Maubeuge and Eberle 1998). Inclined plane and direct shear devices share many of the same advantages and limitations. For the inclined plane test, specimens are often larger (up to a meter or more in size), normal stress is limited to low values (typically < 50 kPa), displacement is measured as a function of tilt angle, and shearing is force-controlled (by gravity). Thus the inclined plane device is well suited for constant stress creep tests of landfill cover systems. Failure occurs quickly and post-peak response is not measured in the standard test. Large displacement strengths can be obtained with special equipment that limits the travel of the upper shear box after failure (Lalarakotoson *et al.* 1999). Another disadvantage of the inclined plane apparatus is that stress conditions on the failure surface become increasingly non-uniform with increasing tilt angle. This can be partially corrected by using a device in which the front and rear side walls of the upper box can be adjusted towards vertical during the test (Lalarakotoson *et al.* 1999).

6.5.4. *Practical implications*

Direct shear is expected to remain the preferred general test method for GCLs because it can be used for any type of GCL product, a large range of normal stress is possible, large specimens can be tested, post-peak response can be obtained, and shear strengths are measured in one direction with theoretically uniform shear displacement. Torsional ring shear and inclined plane devices will no doubt continue to be used for research purposes. Accurate values of τ_p can be measured using 300 mm × 300 mm direct shear specimens, but τ_r generally cannot be measured. The primary disadvantage of limited displacement in direct shear devices has been partially eliminated now that internal residual shear strengths of all hydrated GCLs are known to be essentially the same as that of hydrated bentonite (Fox *et al.* 1998a). However, torsional ring shear tests provide the only reasonable means to obtain residual shear strengths of some GCL interfaces (e.g. GMX/NP GCL). None of the above shearing devices is well suited for the control of drainage conditions or the measurement of pore pressures on the failure surface during shear. Pore pressure measurements could be used to indicate the maximum allowable displacement rate for drained shear conditions or to calculate effective normal stresses on the failure surface for faster undrained shear conditions. Application of backpressure may permit such measurements; however, this capability has yet to be developed.

6.6. Specimen gripping/clamping system

One of the most important aspects of a GCL shearing device is the gripping/clamping system that secures the test specimen to the shearing blocks. The most accurate shear strength data are obtained when the intended failure surface has the lowest shear resistance of all possible sliding interfaces. In this case, shear displacement is uniform at all points on the failure surface, τ_p occurs simultaneously everywhere, and the relationship between average τ (total shear force/area) and average Δ (relative displacement of shearing blocks) is representative of the actual material behavior. However, depending on the shear strength of the specimen and the type of specimen gripping surfaces, the intended failure surface may not have the lowest shear resistance and failure may occur elsewhere (e.g. between the specimen and one of the gripping surfaces), thus rendering a test invalid. The gripping surfaces in many GCL shear devices are composed of wood, plastic, or metal plates, sandpaper, or coarse soil and are not sufficiently rough to shear strong specimens (e.g. reinforced GCLs) without the use of end clamps. Thus, to avoid unsuccessful GCL shear tests, geosynthetic clamping is used in nearly all production testing laboratories to force failure at the intended interface (Figure 18). Clamping systems usually consist of a bolted bar or mechanical compression clamps that fix the geosynthetics to one or both ends of the shearing blocks. In some cases, the geosynthetics have been stapled to wooden shearing blocks (Bressi *et al.* 1995) or simply wrapped around the ends of shearing blocks and anchored with the applied normal load (Frobel 1996).

Due to extensibility of the geosynthetics, shear displacement will not be uniform on the failure surface for any GCL or GCL interface test in which the geosynthetics become tensioned at end clamps. In such cases, failure progresses across the specimen and the measured peak strength is less than the actual peak strength for materials/interfaces that experience post-peak strength reduction. The error depends on specimen size and geometry, relative strengths of the various interfaces involved, extensibility of the geosynthetics, and the shape of the true τ–Δ relationship for the intended failure surface. Thus geosynthetic clamping may introduce error into the test data, although the magnitude of the effect is currently unknown. To obtain accurate stress–displacement behavior, gripping surfaces should enforce uniform shearing of the test specimen over the entire failure surface at all levels of displacement. To achieve such a condition, the gripping surfaces must prevent slippage between the specimen and the shearing blocks. In addition, the gripping surfaces should not interfere with the measured shear strength over a wide range of normal stress and should provide excellent drainage for hydrated GCL tests.

A few studies have reported the development of effective gripping surfaces for GCLs and GCL interfaces. Nail plates molded in epoxy with a high density

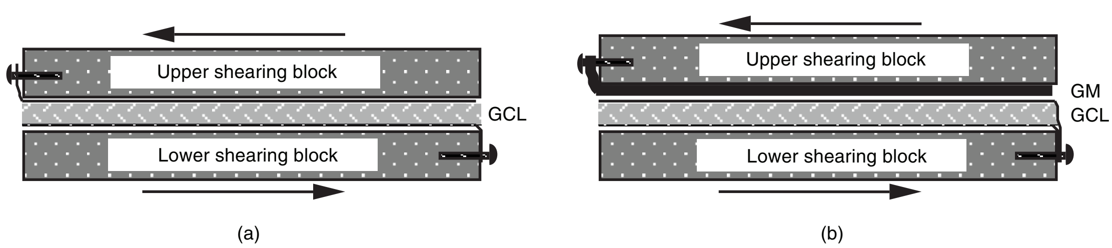


Figure 18. Direct shear test configurations illustrating geosynthetic end-clamping for: (a) GCL internal shear strength; (b) GM/GCL interface shear strength

(1 nail/cm^2) of short sharp nails, each 2 mm in height, have worked successfully (Zanzinger and Alexiew 2000). Good success has also been obtained using a 'textured steel grip' that consists of a parallel arrangement of wood working rasps attached to the shearing blocks (Olsta and Swan 2001; Pavlik 1997; Trauger *et al.* 1997). Fox *et al.* (1997) used modified metal connector plates (i.e. 'truss plates' used for wood truss construction), which have the advantage of providing a well-drained surface in addition to a large number of sharp 1–2 mm tall triangular teeth that uniformly grip a GCL specimen (1 tooth/1.1 cm^2). These plates provided sufficiently aggressive gripping that even very strong NP GCLs could be sheared internally without the use of end clamps (Fox *et al.* 1998a). Triplett and Fox (2001) glued single-sided GMX specimens to the top shearing block for GMX/NP GCL interface strength tests. This method prevented slippage of the GMX but was limited to lower normal stresses by the shear strength of the glue ($\sigma_{n,s}$ < approx. 280 kPa). Gluing is not recommended for GCL specimens because of possible interference with the failure mechanism (e.g. pullout of fibers, rupture of stitches). Gluing has been used for NP GCLs tested in ring shear (Eid *et al.* 1999; Stark and Eid 1996); however, careful steps were followed to ensure that the glue was not applied to materials near the failure surface.

Figure 19 shows the effect of geotextile end-clamping on measured τ–Δ relationships for internal shear of a W/NW NP GCL. For the first test, the shearing blocks were covered with medium-coarse sandpaper and the ends of the supporting geotextiles were clamped to the shearing blocks. At $\Delta = 112$ mm the woven GT failed in tension just behind the clamp. A post-test inspection revealed that the woven GT slipped on the sandpaper, and the GCL did not fail internally. Interestingly, the measured τ–Δ relationship shares some similar features (wide peak, double peak) with those depicted in Figure 17. A second test was performed using rougher, coarse sandpaper and the same clamping system. In this case, the geotextiles became tensioned at the clamps as before but the GCL specimen failed internally. The resulting τ–Δ relationship has well-defined peak and large displacement shear strengths. A third replicate shear test was conducted using the truss plate gripping surfaces without clamping. Compared with the relationship obtained using coarse sandpaper, the truss plates produced a higher peak strength, a smaller corresponding displacement at peak, and a slightly lower residual strength. Inspection of the failed specimen revealed a uniform internal shear failure at the bentonite/W GT interface.

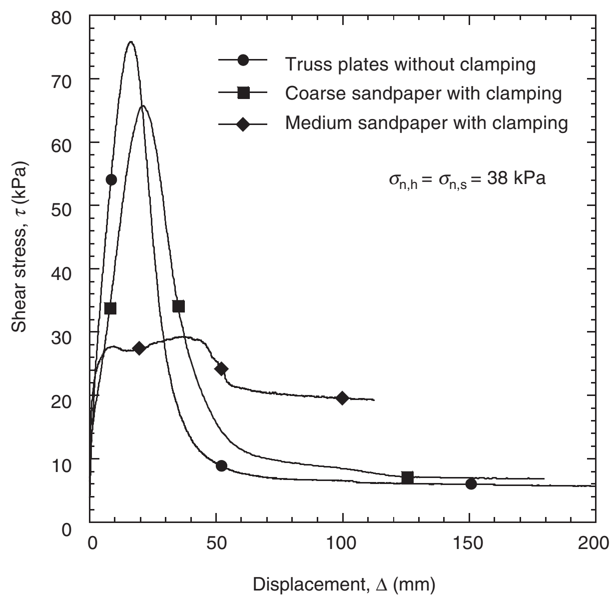


Figure 19. Effect of geotextile end-clamping on measured stress–displacement relationships for internal shear of a hydrated W/NW NP GCL (Fox *et al.* 1997)

Although still a point of debate, it is recommended that the GCL shear testing profession move towards a standardized specimen gripping surface that does not require end-clamping to be successful. Clamping systems may still be used to secure the ends of a test specimen but should not participate significantly in the shearing process. The gripping surface described by Fox *et al.* (1997) has worked well, although there may be similar materials that do not require extensive modification (i.e. extensive machining) to perform equally well. With regard to GM/GCL interface tests, gripping surfaces with a high density of small, sharp-angled teeth that can 'bite' into the back of a GMS or single-sided GMX specimen would probably not require clamping and be much easier and more reliable than gluing. The debate on this issue is whether or not such aggressive gripping surfaces simulate field shear conditions. One view is that laboratory shear tests should reveal true material behavior under uniform shear conditions and without the effects of progressive failure. Progressive failure on a field scale can then be taken into account using an appropriate stability analysis method. The other view is that, in the field, aggressive gripping probably does not occur, and this leads to local progressive failure effects that should be reflected in the laboratory shear test.

6.7. Specimen selection and trimming

An important source of uncertainty in GCL and GCL interface shear testing is associated with the selection of test materials. Obtaining representative samples of NP GCLs is especially challenging because of variations in needle-punched fiber density. For example, assume that samples of an NP GCL product are submitted for testing that have a lower average fiber density than the material delivered for construction. Based on measured shear strengths, a designer may conclude that the critical failure mode is internal GCL shear, whereas an interface failure may actually occur in the field. Ideally, GCL specimens that are tested to obtain or verify design strength parameters should be selected from rolls delivered to the actual project site. However, this will often be impractical because the testing will delay installation or expose the installer to substantial risk if GCL rolls are installed before the tests are finished. Another possibility is to test GCL rolls that are designated for the specific project before the rolls leave the manufacturing plant. The next best alternative is to obtain samples of the same product from the same manufacturing plant that have recently been shipped to another site. If none of these options is possible, samples may be obtained directly from the manufacturer if conformance testing is performed at the time of construction to establish that the delivered materials are at least as strong as the original test materials. It is important to establish who has the responsibility for properly conducting and interpreting such tests (Evans *et al*. 1998; Smith and Criley 1995).

Once GCL and other geosynthetic sample rolls have been delivered for testing, the conservative approach is to take specimens from the weakest areas of the rolls. For example, these areas could be determined for internal strength of an NP GCL using peel tests (Marr 2001). Test specimens should be trimmed using a sharp utility knife or scissors such that the geosynthetics are not damaged, the reinforcement is not damaged (if applicable), and a minimal amount of bentonite is lost. One method to reduce bentonite loss is to wet the periphery of each GCL specimen a few minutes prior to cutting. The effects of poor specimen trimming procedures are unlikely to be of primary importance for shear strength testing due to the large size of standard GCL direct shear specimens.

6.8. Gap setting and multi-interface tests

In direct shear devices, the upper half of the shear box is separated from the lower half prior to shear using a gap setting. The gap should be vertically aligned with the intended failure surface, taking into account possible volume change of a GCL specimen and any underlying materials during hydration/consolidation. An improper gap setting can interfere with the failure mechanism or may allow friction to develop as the shear boxes slide over one another. Further separation of the shear boxes can allow several interfaces to be tested simultaneously (i.e. a multi-interface test) and thus failure to occur along the weakest interface. This can reduce the number of required tests and lead to better understanding of the shear behavior and potential weaknesses in a bottom liner or cover system. A properly designed and conducted series of multi-interface tests will directly yield combination design peak and residual failure envelopes, rendering the analysis presented in Section 5 unnecessary. The main disadvantage of multi-interface tests is that strength parameters are obtained only for the failure surface and not for the other materials/interfaces. Thus no information is obtained on how close the other materials/interfaces were to failure. This is of no consequence if the materials and test procedures are truly representative of field conditions. If, however, there is uncertainty in the test data, which is always the case, then knowing that a material/interface with a significantly lower residual strength (such as a hydrated GCL) almost failed could be important. Another difference for the multi-interface test is that measured shear displacements will be equal to the cumulative displacement of all materials/interfaces and will be larger than that of the failure surface alone. Multi-interface tests are thus more difficult to perform and interpret than single-interface tests, which requires that the engineer and testing laboratory have even more experience to avoid errors. If concerns arise, the critical materials/interfaces should be tested singly to check design strength parameters.

6.9. Normal stress selection and number of tests

It is important to select the proper normal stress range for GCL shear testing because failure envelopes are commonly non-linear and because the normal stress level can affect the failure mode of a test specimen (see Section 4.2). The normal stress sequence during GCL hydration and consolidation may also affect measured shear strength (see Sections 6.10, 6.11). GCLs in bottom liner systems are subjected to a normal stress that is initially low and increases to a high value (as large as 1000 kPa or more) with time. If not encapsulated, these GCLs will hydrate under low normal stress and consolidate to higher stress levels as the waste fill is placed. Stability analyses and associated GCL strength tests must be conducted for low, intermediate, and high normal stress conditions in this case. On the other hand, GCLs placed in cover systems are subjected to a low normal stress (approx. 10–25 kPa) that is nearly constant after construction. Stability analyses and shear tests need only be conducted for a much smaller normal stress range in this case.

ASTM D 6243 requires that a minimum of three GCL shear tests be performed to define a failure envelope for a GCL or GCL interface over the appropriate normal stress range for a given application. More tests should be conducted if the required normal stress range is large or if the initial data points show significant scatter or deviation from linearity. If shear strengths are needed for a small normal stress range, such as for a landfill cover system, a minimum of three tests are still recommended to account for material/test variability and to characterize the failure envelope for the critical interface. If shear strengths are needed only at a single normal stress, then a minimum of two replicate tests should be conducted.

6.10. Hydration stage

6.10.1. *Need for hydration*

The shear strength of GCLs and GCL interfaces is affected by hydration liquid and hydration procedure. Shear tests should therefore be conducted under hydrated conditions when GCL hydration is expected in the field. Full hydration should always be expected in the field unless the bentonite is encapsulated between two geomembranes (Daniel *et al*. 1993; Gilbert *et al*. 1997; Stark 1997). Mathematical models have been developed to predict the amount of bentonite hydration that will occur within an encapsulated GCL over the design life of a waste disposal facility (Giroud *et al*. 2002; Thiel *et al*. 2001), although the actual amount is unknown due to lack of test data. If an analysis of encapsulated bentonite hydration is performed, GCL shear tests may be needed for dry and fully hydrated moisture conditions to construct pro-rated failure envelopes (see Section 4.3).

6.10.2. *Hydration liquid*
GCL specimens can be hydrated, although not saturated, by inundation in a shearing device. Tap water is almost always used as the hydration liquid because of convenience and because its chemistry is comparable to the pore water in most soils. A site-specific liquid can also be used. GCL shear strengths have been obtained for different hydration liquids, with distilled water, tap water, mild leachate, harsh leachate, and diesel fuel yielding progressively higher values (Koerner 1998). GCL hydration with tap water is therefore conservative. In general, if a hydration liquid increases the free swell of bentonite, the shear strength of a hydrated GCL or GCL interface is expected to decrease due to higher water content of the bentonite, higher potential for bentonite extrusion into interfaces, and greater stretching of reinforcement (Gilbert *et al.* 1997).

6.10.3. *Hydration normal stress and time*
GCL specimens should be hydrated under the normal stress expected in the field at the time of hydration. Daniel *et al.* (1993) and Stark *et al.* (1998) showed that hydration of non-encapsulated GCLs occurs relatively quickly, within a few days or weeks, when placed next to damp soil or a compacted clay liner. The appropriate hydration normal stress ($\sigma_{n,h}$) in the laboratory will therefore often be a low value. Ideally, a GCL specimen should be hydrated to equilibrium (i.e. until volume change ceases), a procedure that may require a hydration time (t_h) as long as 3 weeks (De Battista 1996; Gilbert *et al.* 1996a; Stark and Eid 1996) and, depending on $\sigma_{n,h}$, may even reveal a change from compression to expansion behavior during hydration (Marr 2001). As a practical alternative, Gilbert *et al.* (1997) suggested that a GCL can be considered fully hydrated when the change in thickness is less than 5% over a 12 h period. However, use of this criterion may still require $t_h = 10$ to 20 days. An alternative method is to monitor change in thickness until the specimen has reached 100% primary swelling as determined by ASTM D 4546. The time required for full GCL hydration depends on drainage conditions and generally decreases with increasing $\sigma_{n,h}$. Gripping surfaces or loading platens that do not provide adequate drainage pathways may prevent a GCL specimen from becoming fully hydrated (Gilbert *et al.* 1997). Free drainage conditions on both sides of a GCL specimen will shorten the hydration time. If only one side is freely draining, as in the case of a GCL placed against a GM, a longer hydration time will probably be necessary.

The difficulty with the foregoing procedures is the long duration of the hydration stage for a GCL shear test. Most production testing laboratories hydrate GCLs for 1 to 2 days. Incomplete hydration may result in the measurement of unconservative shear strengths, especially for GCL internal strength tests. Full GCL hydration is also important for measurement of interface strengths because of possible bentonite extrusion to interfaces. Thus, accelerated hydration procedures are needed that do not alter measured shear strength.

6.10.4. *Accelerated hydration procedures*
Hydration to equilibrium is unlikely to be practical for production testing in which GCL specimens are hydrated in the shearing device. There are two ways to circumvent this problem. First, some direct shear devices have separate shearing frame and shear box assemblies so that multiple GCL specimens can be hydrated and consolidated simultaneously outside the shearing frame. As a result, shear tests are not delayed by the lengthy time required to hydrate and consolidate each specimen. Second, a two-stage accelerated hydration procedure can be used to reduce the in-device time for GCL specimens to reach hydration equilibrium (Fox *et al.* 1998a). According to this method, a GCL specimen is initially hydrated outside the shearing device for two days under a low normal stress by adding just enough water to reach the expected final hydration water content (estimated from previous tests). The specimen is then placed in the shearing device and hydrated with free access to water for two additional days under the desired $\sigma_{n,h}$. Most GCL specimens attain equilibrium in less than 24 h using this procedure (Fox *et al.* 1998a; Triplett and Fox 2001).

Figure 20 illustrates the performance of the two-stage accelerated hydration procedure for two W/NW NP GCL specimens. One specimen was placed dry in the shearing device and hydrated with free access to water under $\sigma_{n,h} = 38$ kPa. A second specimen was hydrated using the accelerated procedure. In this case, the GCL specimen was placed in a shallow pan, brought to a water content of 185%, and cured for two days under a 1 kPa normal stress (applied using dead weights). The specimen was then transferred to the shearing device and hydrated with free access to water under $\sigma_{n,h} = 38$ kPa for an additional two days. Measurements of vertical

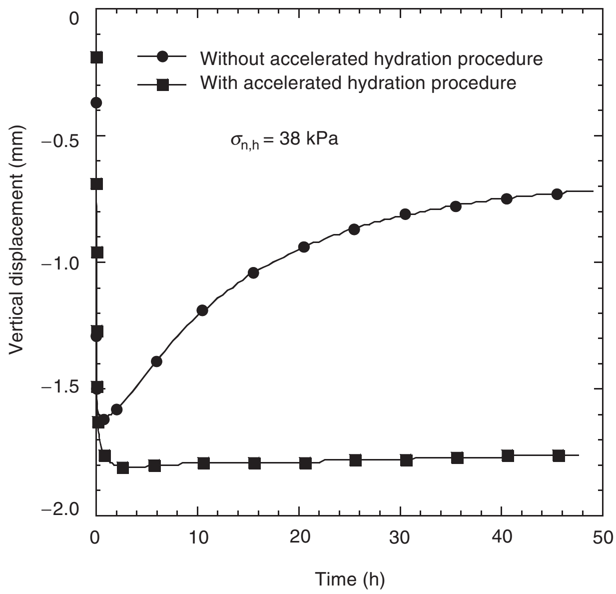


Figure 20. Effect of two-stage accelerated hydration procedure for a W/NW NP GCL (Fox *et al.* 1998a)

displacement of the loading platen during hydration in the shearing device (Figure 20) indicate that volume change essentially ceased in 2 h for the GCL specimen hydrated using the accelerated procedure. Zero vertical displacement corresponds to the condition just before $\sigma_{n,h}$ was applied and negative values indicate compression. Subsequent shear tests on these specimens show that the two-stage accelerated hydration procedure had essentially no effect on the measured τ–Δ relationship (Fox *et al.* 1998a).

6.11. Consolidation stage

6.11.1. *Need for consolidation*
If the shear strength of a GCL or GCL interface is desired at the hydration normal stress, then shearing can begin once the GCL is fully hydrated. However, normal stress often increases on a GCL after hydration in the field and shear strength values are needed at higher normal stress levels. For bottom liner applications in which a GCL is expected to hydrate soon after installation, GCL specimens should be fully hydrated under the appropriate low normal stress (initial value in the field) and then consolidated to various shearing normal stresses that span the range needed for stability analysis. Shearing of each specimen can begin once consolidation is completed under the desired shearing normal stress. The procedure for cover system applications is more straightforward because consolidation is generally not required; each GCL specimen can be fully hydrated and sheared under constant (low) normal stress.

6.11.2. *Effect of hydration/consolidation procedure*
It is important to follow the same normal stress sequence for hydration/consolidation in the laboratory as expected in the field because this sequence can affect measured shear strengths. At a given $\sigma_{n,s}$ the internal shear strength of unreinforced GCLs decreases with increasing bentonite water content (Daniel *et al.* 1993; Zelic *et al.* 2002). As an example, Figure 21 presents peak and residual failure envelopes for five GMX/GM-supported encapsulated GCL specimens that were hydrated at the shearing normal stress (Eid and Stark 1997). The bentonite was able to hydrate because the inner and outer edges of the specimens were in contact with a water bath in the ring shear device. Figure 22 presents corresponding peak and residual shear strengths for a second series of tests in which replicate specimens were hydrated at $\sigma_{n,h} = 17$ kPa and then consolidated in small increments to $\sigma_{n,s} = 50$, 100, 200 and 400 kPa. The failure envelopes from Figure 21 are also shown in Figure 22 for comparison. A 25–30% reduction in measured shear strengths occurred when specimens were hydrated at $\sigma_{n,h} = 17$ kPa and then consolidated prior to shearing. Hydration at low normal stress results in more

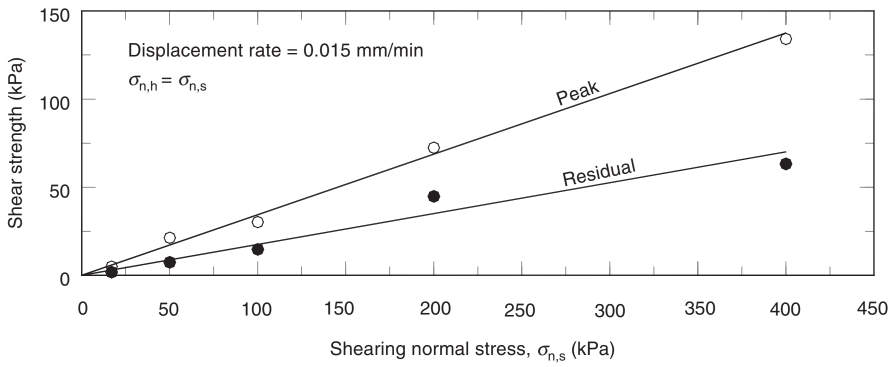


Figure 21. Peak and residual failure envelopes for a GMX/GM-supported encapsulated GCL hydrated at the shearing normal stress (Eid and Stark 1997)

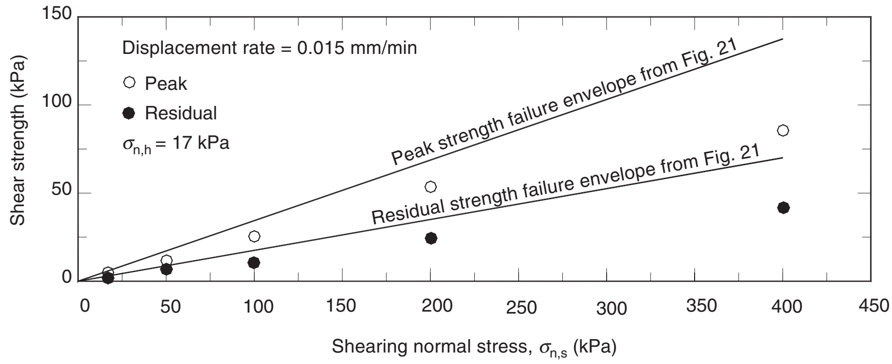


Figure 22. Peak and residual shear strengths for a GMX/GM-supported encapsulated GCL hydrated at $\sigma_{n,h} = 17$ kPa and then consolidated to each shearing normal stress (Eid and Stark 1997)

water being adsorbed into the double layers of the bentonite particles, not all of which is expelled during subsequent consolidation. This resulted in higher bentonite water contents for the second test series and consequently lower shear strengths.

Hydration/consolidation procedure may also affect the internal shear strength of reinforced GCLs. Hydration of NP GCLs causes tensioning of the reinforcement fibers. Although definitive test data are not available, some fibers may break or pull out of the geotextiles during the swelling process, possibly reducing τ_p. Stark and Eid (1996) suggested that bentonite extruded during hydration/consolidation may also facilitate the pullout of needle-punched fibers by lubricating frictional connections between the fibers and the anchoring GT. If an NP GCL is then consolidated to a higher normal stress, the fibers will relax and Δ_p will be larger during shear (Eid *et al.* 1999). The stitches of an SB GCL are much stronger and more widely spaced than needle-punched fibers and are probably not affected by bentonite hydration.

Specimen hydration/consolidation procedure can affect GCL interface shear strengths. Laboratory and field tests have shown that swelling bentonite can extrude through the supporting geotextiles of a GCL and smear onto adjacent materials, forming a slippery interface (Byrne 1994; Daniel *et al.* 1998; Eid *et al.* 1999; Gilbert *et al.* 1997; Stark and Eid 1996; Pavlik 1997; Triplett and Fox 2001). In these studies, extruded bentonite has been more commonly observed on the W side of a W/NW NP GCL than on the NW side. It is expected that the amount of bentonite extruded through a supporting GT will generally increase as the GT becomes thinner, the bentonite becomes softer (i.e. lower $\sigma_{n,h}$), and as more water flows into the interface from the GCL during consolidation (i.e. larger consolidation increment, larger transmissivity of the interface, or larger hydraulic conductivity of the adjacent material). The normal stress sequence during hydration and consolidation has also been shown to affect the peak and large displacement shear strengths of GMX interfaces with NP (NW side) and SB GCLs (Hewitt *et al.* 1997). In the Hewitt *et al.* study, interfaces that were consolidated to higher normal stress levels were sheared 15 min after the application of the final consolidation load increment to approximate undrained shear conditions.

6.11.3. *Recommended consolidation procedure*

Little information is currently available on the most appropriate consolidation procedure for GCLs. A single rapid normal stress change from $\sigma_{n,h}$ to $\sigma_{n,s}$ is not appropriate for a hydrated GCL specimen unless the change is small (e.g. $\sigma_{n,s}-\sigma_{n,h} \leq 0.5\sigma_{n,h}$) or unless the change simulates an actual field condition as requested by the user. Instead, consolidation loads should be applied in small increments to avoid extrusion of bentonite from the specimen. Continuous-loading (i.e. ramp-loading) and incremental-loading consolidation procedures have been used with success. The maximum rate of stress increase for a continuous-loading procedure will depend on GCL type, $\sigma_{n,h}$, and experience. Incremental-loading procedures are more common, with consolidation loads generally applied using daily or half-day increments. Merrill and O'Brien (1997) reported that consolidation was effectively completed within 10 h for a W/NW NP GCL subjected to a normal stress increment from 69 to 138 kPa. Vertical displacement measurements are sometimes used to establish the duration of each load increment. A succeeding load increment can be applied even if consolidation is not completed for the current increment. However, a GCL should be fully consolidated under the final load increment so that no excess pore pressures exist within the specimen at the start of shearing. End of consolidation can be estimated using vertical displacement data in a similar manner as for standard oedometer tests (e.g. $\sqrt{t}$ or $\log t$ graphical construction procedures). The optimal load-increment ratio (LIR), equal to the change of normal stress/previous normal stress, is unknown. A value of LIR = 1 (i.e. normal stress doubled each time) may cause bentonite extrusion, and a smaller value (e.g. 0.5) is recommended. If bentonite extrusion is observed with any chosen LIR, the test should be repeated using a smaller LIR.

The unavoidable drawback for the consolidation stage is the time required. Using an LIR = 0.1, Eid *et al.* (1999) reported that 3 to 13 days were required to complete the consolidation stages for NP GCLs tested in ring shear ($\sigma_{n,h} = 17$ kPa; $\sigma_{n,s} = 100$, 200 and 400 kPa). There is no accelerated procedure available to rapidly consolidate hydrated GCLs. The only way to avoid the impact of long consolidation time on a testing program is to simultaneously hydrate/consolidate multiple GCL specimens in separate shear boxes outside the shearing frame (see Section 6.10.4). A consolidation stage is typically not included for current production GCL shear testing. Instead, GCL specimens are usually sheared at the hydration normal stress. If a consolidation stage is included, the consolidation load is often applied as a single increment and permitted to remain on the specimen for only a few hours (or less) before shearing begins. Such a procedure may yield inaccurate shear strengths due to bentonite extrusion and the presence of positive pore pressures in the specimen at the start of shearing.

6.12. Shearing stage

6.12.1. *Importance of displacement rate*

With the exception of stress-controlled creep shear tests, GCL shear tests should be displacement-controlled so that post-peak behavior can be measured. The only issue that remains unresolved for the shearing stage is the appropriate rate(s) of shear displacement. The maximum allowable displacement rate is important because it greatly affects the cost and time required to perform GCL shear tests and thus has an impact on the marketability and acceptance of GCL products. It might be expected that the shear strength of hydrated GCLs would be rate-dependent because shear-induced pore pressures may be generated in the bentonite, and because both hydrated bentonite and geosynthetics display creep and strain-rate effects. Conversely, the strength of dry unreinforced GCLs should show minimal displacement rate effects. Eid and Stark (1997) demonstrated that, indeed, peak and residual shear strengths of dry encapsulated GCLs are essentially constant for displacement rates less than 1 mm/min (Figure 24: see Section 6.12.3). Therefore the industry default displacement rate of 1 mm/min is recommended for such tests. The remainder of Section 6.12 is concerned with appropriate displacement rates for hydrated GCLs.

Further insight into the mechanisms responsible for displacement rate effects can be gained by examination of Figure 23, which presents peak and residual internal shear strengths of filled and unfilled specimens of a hydrated W/NW NP GCL tested at displacement rates ranging from 0.015 mm/min to 36.5 mm/min ($\sigma_{n,h} = \sigma_{n,s} = 17$ kPa). The dry powdered bentonite was not removed from the GCL specimens for the filled tests. For the unfilled tests, the bentonite was removed prior to hydration by holding the specimens vertically and lightly tapping the geotextiles with a finger (Stark and Eid 1996). Internal peak shear strengths of both the unfilled and filled specimens are approximately constant for displacement rates less than 0.04 mm/min. For displacement rates between 0.04 and 1.5 mm/min, peak strengths of the unfilled and filled specimens show similar increases. This suggests that the source of the displacement rate effect lies within the geosynthetics and may include rapid tearing or pullout of reinforcement fibers. However, the peak strength relationships diverge for displacement rates greater than 1.5 mm/min. This may have resulted from the generation of positive pore pressures in the filled specimens, which reduced the measured peak strengths. Thus Figure 23 suggests that the relative importance of different mechanisms that control internal shear resistance may vary with displacement rate. The data also show that residual shear strength was essentially independent of displacement rate for both filled and unfilled GCL specimens.

6.12.2. *Maximum displacement rate*

Unless a specific application requires rapid shearing to simulate field conditions (e.g. seismic loading), maximum allowable displacement rates have been established with the intent of minimizing the generation of pore pressures on the failure surface during shear. Conceptually, such rates should vary with product/interface type, hydration/consolidation conditions, shearing normal stress, and drainage conditions. In lieu of a rigorous theoretical analysis, ASTM D 6243 recommends Equation 9 for the maximum displacement rate R. If shear-induced pore pressures are not expected on a GCL interface, ASTM D 6243 recommends $R = 1$ mm/min.

Use of Equation 9 can lead to very slow displacement rates that may not be practical for production testing of GCLs (Gilbert *et al.* 1996a; Marr 2001). Based on consolidation data for four GCL products, Shan (1993) estimated that maximum displacement rates for internal shear tests would range from 0.001 to 0.0001 mm/min. Shear tests conducted to $\Delta = 50$ mm at these rates will require 34.7 and 347 days respectively. Such test durations are clearly prohibitive for production work. An implicit assumption in the use of Equation 9 for internal shear tests is that failure occurs through the center of the hydrated bentonite. However, several studies have indicated that internal shear failures of hydrated reinforced GCLs occur at a bentonite/GT interface (Eid *et al.* 1999; Fox *et al.* 1998a; Gilbert *et al.* 1996a). If this GT is a drained boundary of the GCL, then shear-induced pore pressures on the failure surface should be small and drained (or nearly drained) shear

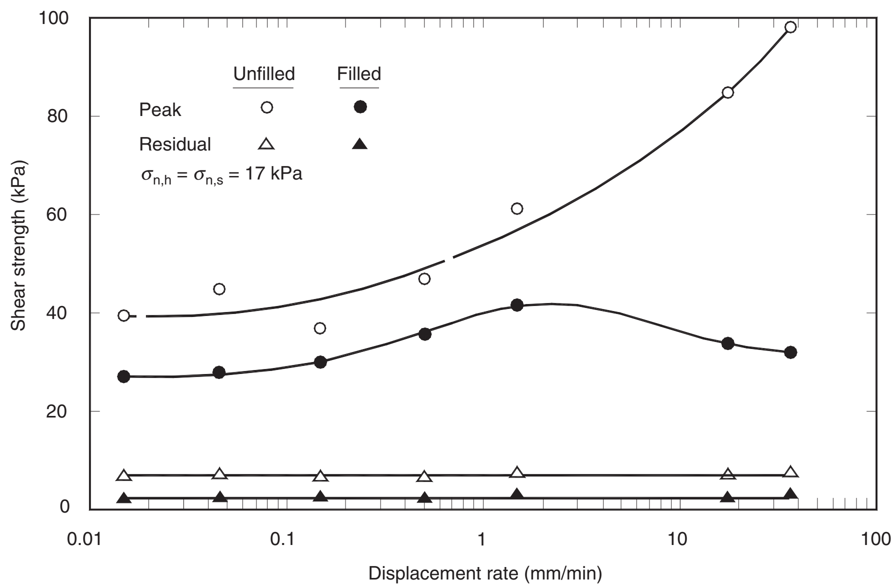


Figure 23. Effect of displacement rate on internal shear strength of filled and unfilled hydrated W/NW NP GCL specimens at $\sigma_{n,h} = \sigma_{n,s} = 17$ kPa (Stark and Eid 1996)

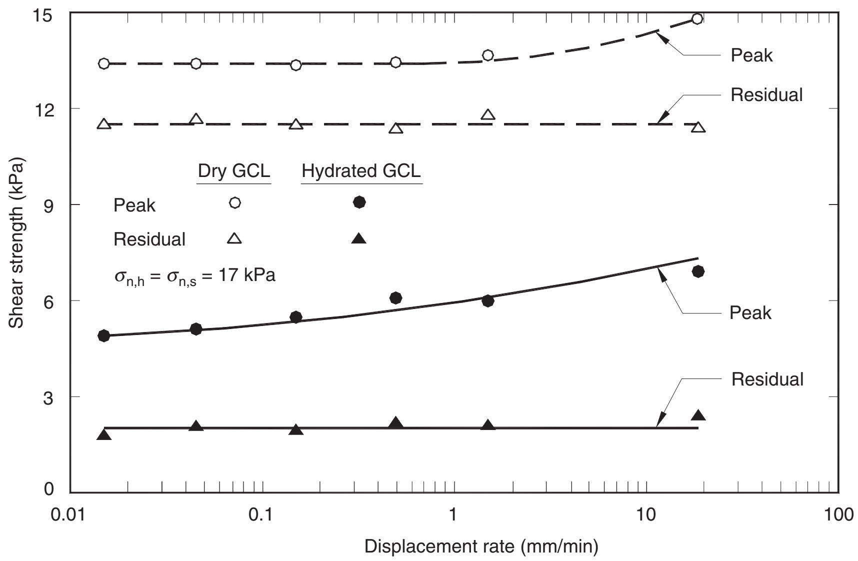


Figure 24. Effect of displacement rate on shear strengths of dry and hydrated GMX/GMS-supported encapsulated GCLs at $\sigma_{n,h} = \sigma_{n,s} = 17$ kPa (Eid and Stark 1997)

strengths should be obtained. Equation 9 is inappropriate for such cases. Thus the practicality and applicability of Equation 9 for shear testing of hydrated GCLs is questioned. As a result of the uncertainty surrounding Equation 9, production laboratories typically use a displacement rate of 1 mm/min, which may or may not be adequate depending on the GCL/interface type and testing conditions.

6.12.3. *Displacement rate effects for hydrated unreinforced GCLs*

Several studies have investigated the effect of displacement rate on measured shear strengths for hydrated unreinforced GCLs. Daniel *et al.* (1993) tested small specimens (dia. = 60 mm) of an unreinforced GM-supported GCL at several water contents and for displacement rates equal to 0.26 and 0.0003 mm/min ($\sigma_{n,s} = 27$–139 kPa). Peak strengths were significantly higher (often 100–200%) for the faster shear tests at all water content levels. Using a ring shear device, Eid and Stark (1997) measured shear strengths of hydrated unreinforced GMX/GMS-supported encapsulated GCLs ($\sigma_{n,h} = \sigma_{n,s} = 17$ kPa) at displacement rates ranging from 0.015 to 18.5 mm/min (Figure 24). Peak strengths increased approximately 13% per log cycle of displacement rate and residual strengths were concluded to be independent of displacement rate (although the τ_r data points in Figure 24 suggest a slightly increasing trend). All failures occurred at the hydrated bentonite/GMX interface. Gilbert *et al.* (1997) conducted direct shear tests to evaluate displacement rate effects for unreinforced GCLs. Specimens were hydrated for 24 h at a shearing normal stress of 17 or 170 kPa. Two or three tests were conducted at each normal stress and displacement rate to account for variability. Normalized peak shear strengths ($\tau_p/\sigma_{n,s}$) are plotted as a function of displacement rate in Figure 25. Peak strengths generally increase with increasing displacement rate, especially at the lower normal stress level, where the strength at 1.0 mm/min is approximately 40% higher than the strength at 0.0005 mm/min. Displacement rate effects are less pronounced at the higher shearing normal stress. Zelic *et al.* (2002) showed that, on average, peak strengths of a hydrated unreinforced GT-supported GCL increased 54% and end-of-test ($\Delta = 15$ mm) strengths increased 111% when the displacement rate was increased from 0.0015 to 1.2 mm/min. Zelic *et al.* also concluded that cohesion intercept is influenced more by total test duration and final bentonite water content, whereas friction angle is controlled by displacement rate. The above results suggest that a

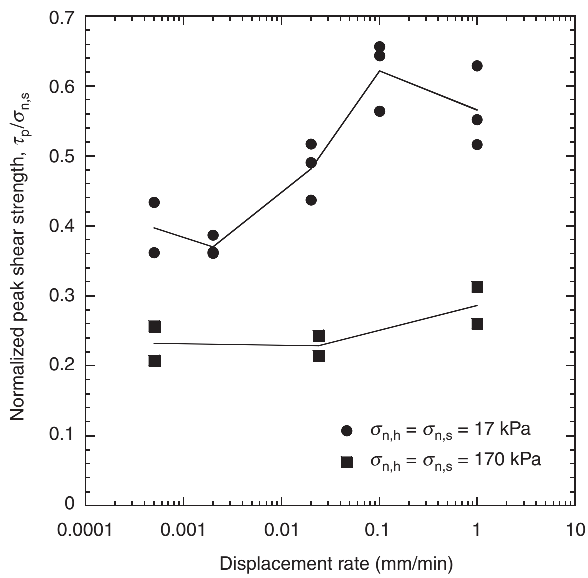


Figure 25. Effect of displacement rate on normalized peak shear strength of a hydrated unreinforced GCL at two normal stress levels (after Gilbert *et al.* 1997, used with permission)

displacement rate of 1 mm/min is too fast for hydrated unreinforced GCLs and that a rate of 0.01 mm/min or less may be needed.

6.12.4. *Displacement rate effects for hydrated reinforced GCLs*

Several studies have also investigated the effect of displacement rate on measured shear strengths for hydrated reinforced GCLs. Stark and Eid (1996) found that the peak shear strength of a hydrated W/NW NP GCL was not significantly affected until the displacement rate exceeded 0.04 mm/min, and that the residual strength was independent of displacement rate (Figure 23; see Section 6.12.1). Berard (1997) conducted direct shear tests on several hydrated W/NW NP GCLs ($\sigma_{n,s} = 25$–100 kPa) and showed that increasing the displacement rate from 0.01 to 1 mm/min resulted in a 41% average increase in peak shear strength. Fox *et al.* (1998a) found that displacement rate had a relatively minor effect on the internal shear strengths of reinforced GCLs. Figure 26 shows τ_p and τ_r values for W/W SB and W/NW NP GCLs obtained at $\sigma_{n,h} = \sigma_{n,s} = 72$ kPa and displacement rates ranging from 0.01 to 10 mm/min. Both values increase 3–5% for each log cycle of displacement rate. The increase in strength with displacement rate can be attributed to shear-induced porewater suctions (i.e. negative pore pressures), creep of the bentonite and/or geosynthetic components, or rapid pullout effects for the reinforcement during shear. Fox *et al.* attributed the displacement rate effect to drained creep of the hydrated bentonite, because: (1) the failure surface for each test was immediately adjacent to a drainage boundary and thus pore suctions should have been small; and (2) τ_r values showed the same general effect as τ_p values, which eliminates possible explanations associated with the geosynthetic reinforcement.

The most comprehensive studies of displacement rate effects for internal shear of hydrated NP GCLs, conducted by Eid *et al.* (1999) and McCartney *et al.* (2002), have yielded contradictory results. In both studies, tests were performed over a large normal stress range that included values above and below the swell pressure of bentonite (approx. 130–140 kPa: Shan and Daniel 1991; Stark 1997). Eid *et al.* (1999) tested a heat-bonded W/NW NP GCL in ring shear with displacement rates ranging from 0.015 to 36.5 mm/min. Peak strengths (Figure 27) were nearly constant for $\sigma_{n,s} = 200$ and 400 kPa and displacement rates of 1 mm/min or less. For $\sigma_{n,s} = 17$ and 100 kPa the relationships display maximum values, with lower peak strengths on either side. Residual strengths were again found to be essentially independent of displacement rate at all normal stress levels. McCartney *et al.* (2002) conducted a similar investigation for a W/NW NP GCL in direct shear with displacement rates ranging from 0.0015 to 1 mm/min. Peak strengths *decreased* linearly with increasing log displacement rate at $\sigma_{n,s} = 520$ kPa and increased linearly with increasing log displacement rate at $\sigma_{n,s} = 50$ kPa. The explanation for the fundamentally different behavior observed in these studies at high normal stress levels is unknown.

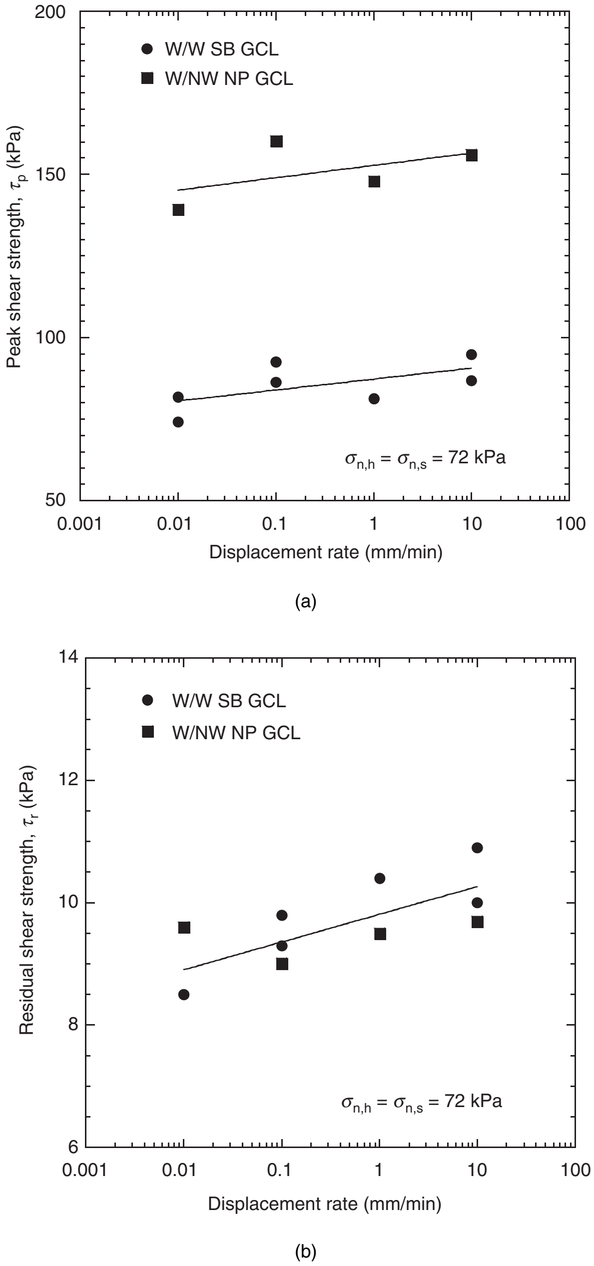


Figure 26. Effect of displacement rate on (a) peak, and (b) residual internal shear strengths of hydrated reinforced GCLs (Fox *et al.* 1998a)

The above results suggest that a displacement rate of 1 mm/min is too fast for hydrated reinforced GCLs and that a rate of 0.01 mm/min or less may be needed.

6.12.5. *Displacement rate effects for hydrated GM/NP GCL interfaces*

Triplett and Fox (2001) found that displacement rate had no effect, on average, for interface shear strengths between the W side of an NP GCL and various HDPE geomembranes at $\sigma_{n,h} = \sigma_{n,s} = 72$ kPa (Figure 28). This is in agreement with the findings of Stark *et al.* (1996), in which displacement rate did not significantly affect measured peak and residual strengths of GMX/NW GT interfaces. These results suggest that a displacement

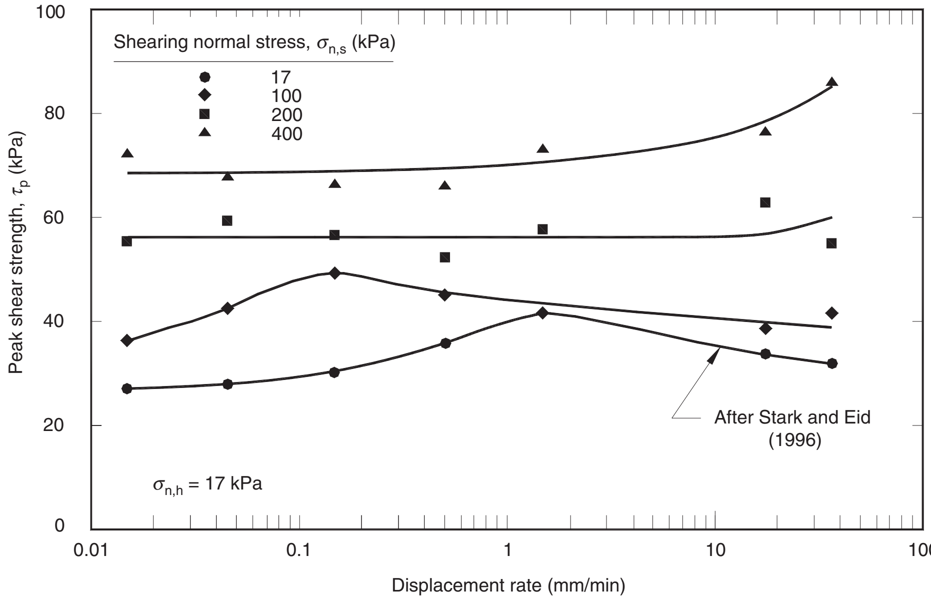


Figure 27. Effect of displacement rate on peak shear strength of a hydrated heat-bonded W/NW NP GCL at four normal stress levels (Eid *et al.* 1999)

rate of 1 mm/min is acceptable for hydrated GM/NP GCL interfaces.

6.12.6. *Recommended maximum displacement rates*
Available data indicate that dry encapsulated GCLs and hydrated GM/NP GCL interfaces show essentially no displacement rate effects and can be sheared at 1 mm/min. No information is available on displacement rate effects for other GCL interfaces (e.g. drainage geocomposite/GCL, soil/GCL, GM/SB GCL). The maximum allowable displacement rate for internal shear of hydrated GCLs remains unclear. Most studies show that internal shear strength increases with increasing displacement rate, although some key studies have produced contradictory results. Taken together, the data suggest that internal shear tests of hydrated GCLs should ideally be conducted at a maximum displacement rate of 0.01 mm/min. This rate may, however, be too slow for production purposes. Until this issue is resolved, a maximum displacement rate of 0.1 mm/min is recommended for hydrated GCL internal shear tests (Fox *et al.* 2004). More research is needed on this issue, as it continues to persist as a dilemma for GCL manufacturers, designers, and production testing laboratories.

The maximum allowable displacement rate for multi-interface test specimens is equal to the maximum allowable displacement rate for the failure surface. However, the failure surface is unknown prior to performing the test. It is recommended that multi-interface specimens containing GCLs be sheared at a maximum displacement rate of 1 mm/min in accordance with ASTM D 5321. If failure occurs within a hydrated GCL, the test should be repeated using a maximum displacement rate of 0.1 mm/min.

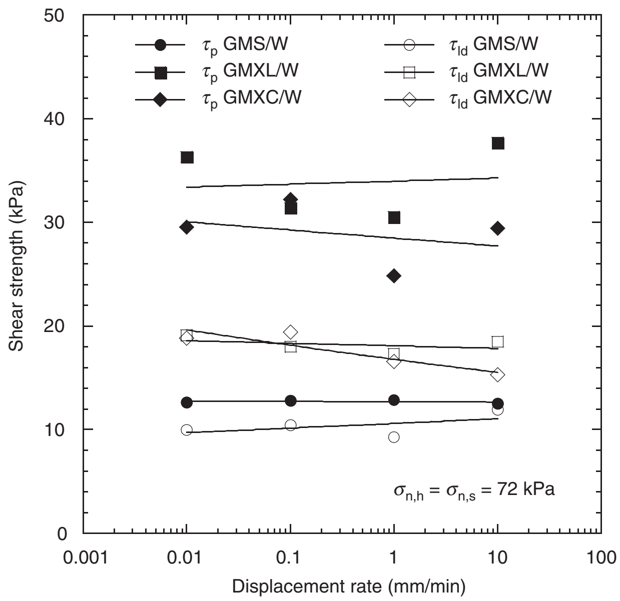


Figure 28. Effect of displacement rate on peak and large displacement ($\Delta = 200$ mm) interface shear strengths between the woven side of a hydrated NP GCL and smooth, laminated textured (GMXL), and coextruded textured (GMXC) HDPE geomembranes at $\sigma_{n,h} = \sigma_{n,s} = 72$ kPa (Triplett and Fox 2001)

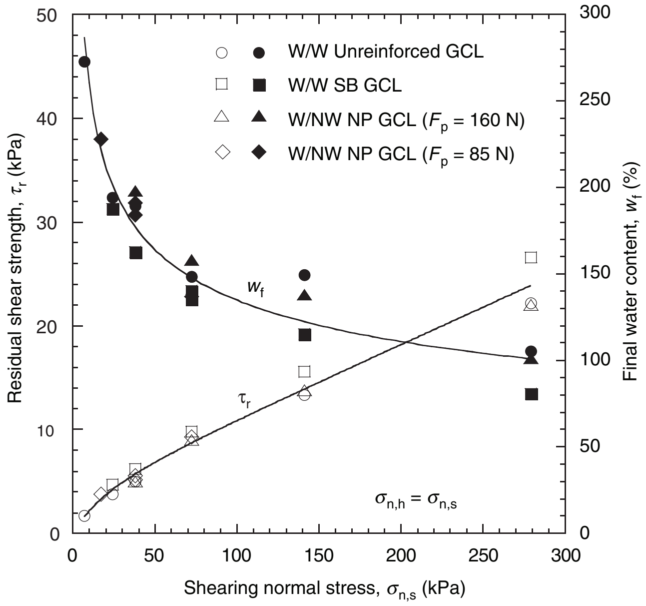
6.12.7. *Accelerated procedure for shearing stage*
Many studies have shown that the internal peak shear strength of hydrated NP GCLs increases with increasing displacement rate. A possible technique to accelerate the shearing of these products and not compromise the measurement of internal peak strength is to increase the displacement rate after τ_p is measured (Eid *et al.* 1999).

For example, a displacement rate equal to 0.01 mm/min could be imposed until Δ_p has been exceeded. The displacement rate could then be increased to 1 mm/min until a residual strength is obtained or until the travel limit of the shear device is reached. The correct large displacement/residual shear strength should be measured if τ_r is independent of displacement rate. Available test data for reinforced GCLs are somewhat contradictory on this point. The data of Stark and Eid (1996) and Eid *et al.* (1999) suggest that τ_r of W/NW NP GCLs is essentially independent of displacement rate, whereas the data of Fox *et al.* (1998a) show a small displacement rate effect for τ_r of both SB and W/NW NP GCLs (see Section 6.12.4). Another possibility would be to increase the displacement rate after τ_p and then reduce it again to 0.01 mm/min near the end of the test to measure τ_r.

6.13. Final inspection and water contents

The failed GCL or GCL interface specimen should be inspected carefully after shearing to assess the surface(s) on which failure occurred and the general nature of the failure. Unusual distortion or tearing of the specimen should be recorded and may indicate problems with the gripping surfaces. The condition of the geosynthetics at specimen end clamps (if present) should also be recorded. Evidence of high tensile forces at the clamps, such as tearing or necking of the geosynthetics, indicates that progressive failure may have occurred during shear. Depending on the extent of localized distress, such a test may be invalid and may need to be repeated using improved gripping surfaces.

Final water contents (w_f) of the GCL specimen and subgrade soil (if applicable) should be taken after shearing to assess the level and uniformity of hydration that was achieved. A minimum of five water content measurements is recommended for the GCL specimen. The shearing device must be disassembled fairly quickly for w_f values to have validity. Figure 29a shows a plot of final water content and internal residual shear strength versus shearing normal stress, as obtained from direct shear tests of several GCL products. Specimens were hydrated using the accelerated procedure described in Section 6.10.4 and sheared at the same normal stress (i.e. $\sigma_{n,h} = \sigma_{n,s}$). A corresponding plot for torsional ring shear tests on heat-bonded W/NW NP GCL specimens ($F_p = 27$ N/10 cm) is shown in Figure 29b. These specimens were hydrated at $\sigma_{n,h} = 17$ kPa and then consolidated to the desired shearing normal stress in small increments. Both plots show that w_f decreases non-linearly with increasing $\sigma_{n,s}$. Although good general agreement is observed, final water contents tend to be higher and residual strengths tend to be lower for the direct shear tests. The reason for this discrepancy is unclear (the trend is contradictory to Section 6.11.2), but presumably reflects differences in the GCL products at the time of testing, as well as differences in testing apparatus and procedures. It is noted, however, that the direct shear w_f values were taken from whole samples of the respective GCL specimens whereas the ring shear w_f values were taken from bentonite sampled in the immediate vicinity of the failure surface using a spatula.



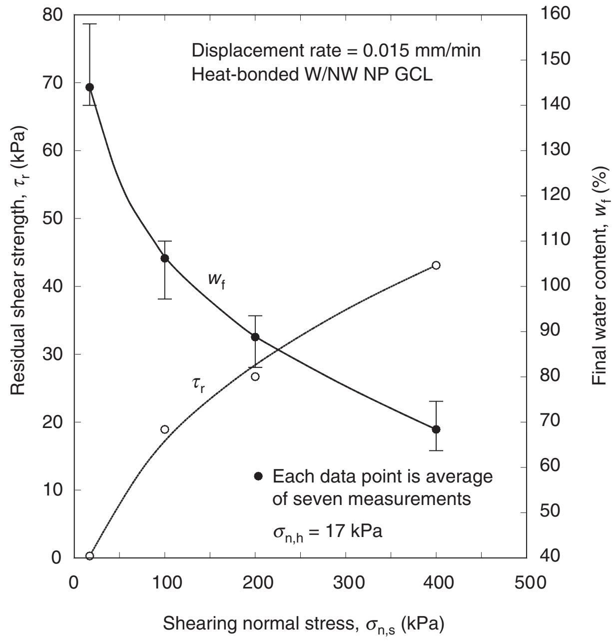


Figure 29. Final water contents and residual internal shear strengths for hydrated GCLs tested in: (a) direct shear (Fox *et al.* 1998a); (b) torsional ring shear (Eid *et al.* 1999)

Figure 29 may be useful in forensic investigations to estimate residual shear strength from measured GCL water contents in the field. Such measurements would need to be taken soon after failure occurs to have validity. Figure 29 may also be used to determine whether complete hydration occurred in a laboratory shear test. If the average final water content reported by a testing laboratory is significantly lower than the values obtained from Figure 29, complete hydration may not have been achieved.

7. SPECIFICATION OF TESTING PROGRAM AND DELIVERY OF TEST RESULTS

Shear tests of GCLs and GCL interfaces should be conducted in accordance with appropriate standard testing procedures (e.g. ASTM D 6243 in the US). However, simply requiring that tests be conducted according to ASTM D 6243 is not sufficient. Test conditions should be specified by the responsible engineer. Marr (2001) provides examples of language that can be included in project specifications to reduce the opportunity for disputes and delays involving GCL shear testing. This section presents a list of additional considerations from Fox *et al*. (2004) that deserve particular attention to ensure that quality test results are obtained.

When contracting for GCL shear tests, a user should *require* the following:

- regular calibration of shear testing device for accuracy of normal stress and shearing force (minimum once per year recommended);
- specimen gripping surfaces that can impart uniform shearing to the test specimen without slippage;
- full GCL hydration is achieved (if applicable) before consolidation of the GCL to the desired shearing normal stress (if applicable);
- consolidation of a GCL in small increments to minimize bentonite extrusion;
- measurement of specimen volume change during hydration, consolidation, and shearing;
- thorough inspection of failed specimen(s); and
- measurement of initial and final GCL water contents and subgrade soil water contents (if applicable).

When contracting for GCL shear tests, a user should *provide* the following:

- GCL material(s) (from actual project site or designated for actual project site if possible);
- subgrade soil(s) (if applicable);
- geosynthetic interface material(s) (if applicable); and
- hydration liquid (if different from tap water).

When contracting for GCL shear tests, a user should *specify* the following:

- specimen selection, trimming, and archiving procedures;
- number and type of tests;
- specimen configuration (bottom to top);
- soil compaction criterion (if applicable);
- number of interfaces (single or multiple) to be tested at the same time;
- orientation of GCL or GCL interface (machine or transverse direction);
- hydration normal stress and hydration time duration (or termination criterion);
- consolidation procedure, including load increments (or load-increment ratio) and load increment duration (or termination criterion); and
- shearing procedure, including shearing normal stress levels, minimum magnitude of shear displacement, and shear displacement rate.

When receiving the results of GCL shear tests, a user should *expect* the following:

- description of specimen selection, trimming, and archiving procedures;
- description of testing equipment;
- description of specimen configuration and preparation conditions;
- description of test conditions (hydration, consolidation, shearing);
- shear stress–displacement relationships;
- specimen volume change data during hydration, consolidation, and shearing;
- peak and large displacement shear strengths and possibly shear strength parameters (see below);
- location and condition of failure surface(s) within test specimens; and
- initial and final GCL water contents and subgrade soil water contents (if applicable).

Based on measured shear strengths, most testing laboratories construct failure envelopes (often by linear regression) and report shear strength parameters for GCLs and GCL interfaces. In simple cases, such as linear shear strength data with small variability, this is a straightforward process and there is little room for interpretation. However, failure envelope construction may not be straightforward if the data indicates a multi-linear, non-linear, or possibly discontinuous failure envelope, or displays significant variability (see Section 4.2). In general, it is recommended that shear strength parameters be determined by the responsible engineer based on project-specific considerations. If strength parameters are provided in a GCL testing report, the engineer should objectively determine whether the methodology used to determine these parameters is consistent with the true trend of the data in the normal stress range of interest, the appropriate shear displacement (for large displacement envelopes), the displayed variability, and the conservatism needed for a given project. However, there are some circumstances, such as conformance testing for construction quality control (CQC) and/or construction quality assurance (CQA), in which a laboratory is asked by a manufacturer, installer, or CQA subcontractor to determine whether certain GCL or GCL interface materials meet a specification (often based on shear strength parameters). The testing laboratory may be best qualified to interpret the shear strength data and make this decision if an engineer is not involved in this stage of the project.

8. CONCLUSIONS

The foregoing discussion of shear strength and shear strength measurement for geosynthetic clay liners (GCLs) and GCL interfaces has led to the following conclusions:

- Measured values of GCL internal and interface shear strengths can show significant variability due to variability in component materials and manufacturing processes, differences in testing equipment and procedures, and changes in the design, manufacture, and application of GCLs over time. As a result, shear strength parameters for final design purposes must be obtained using project-specific materials tested under conditions closely simulating those expected in the field.
- All GCLs and most GCL interfaces experience post-peak strength reduction, in which the measured shear strength decreases after a peak value and ultimately reaches a residual value, after which no further strength reduction occurs. Dry unreinforced textured geomembrane-supported GCLs generally have comparable internal peak strengths and much higher internal residual strengths than hydrated reinforced GCLs.
- Limited available information suggests that the current practice of characterizing GCL shear strength parameters in terms of total normal stress and then using these parameters for drained effective stress stability analyses is either appropriate or conservative.
- Depending on the materials tested and the shearing normal stress range, peak strength failure envelopes for GCLs and GCL interfaces can be linear, multi-linear or non-linear, whereas residual strength failure envelopes are often approximately linear. Changes in the mode of failure of a test specimen with increasing normal stress may produce a residual strength failure envelope that contains discontinuities. Unconservative fitting practices must be avoided in the construction of failure envelopes from test data, especially when linear envelopes are used to characterize non-linear data. Shear strength parameters should not be extrapolated outside the normal stress range for which they were obtained. Under some conditions these problems can be avoided by directly entering data points that describe a failure envelope into slope stability software.
- The shear strength of GCLs and GCL interfaces can display in-plane anisotropy. Shear strength will probably be different in the machine and transverse directions (i.e. rotated 90°), but may also be different in opposite machine directions (i.e. rotated 180°). Direct shear tests of GCLs and GCL interfaces should be conducted in the weakest machine direction (if one exists) to avoid overestimating shear strength in the field.
- In long-term field and laboratory creep tests, GCLs and GCL interfaces have sustained design-level shear loads without failure for periods up to 9 years. Failures that have occurred were successfully predicted using short-term shear strengths. As with applications involving reinforcement geosynthetics, it may be possible to predict long-term design shear strengths for reinforced GCLs by applying strength reduction factors for creep and durability to short-term strengths measured in laboratory shear tests.
- The selection of appropriate peak and residual failure envelopes for design should be based on considerations of expected shear displacements in a bottom liner or cover system. Because peak failure envelopes for geosynthetics are often non-linear, it may be necessary to construct combination design failure envelopes using segments from the individual failure envelopes of system components. The combination design peak failure envelope corresponds to the lowest peak shear strength of all components at each normal stress. The combination design residual failure envelope corresponds to the combination design peak failure envelope and does not simply represent the lowest residual shear strength of all components at each normal stress.
- Good-quality shear testing will produce smooth shear stress–displacement relationships that display good similarity and do not contain double peaks or large undulations. Relationships from internal strength tests on hydrated reinforced GCLs should display sharp peaks and large post-peak strength reduction. Relationships from internal strength tests of dry encapsulated GCLs and hydrated GCL interfaces will generally display broader peaks and less post-peak strength reduction. Examination of shear stress–displacement relationships is the best way to make a preliminary assessment of the quality of GCL shear test results. Shear stress–displacement relationships should be routinely included as part of the test results package for GCL shear testing programs.
- Direct shear is expected to remain the preferred general test method for GCLs because it can be used for any type of GCL product, a large range of normal stress is possible, large specimens can be tested, post-peak response can be obtained, and shear strengths are measured in one direction with theoretically uniform shear displacement. Residual shear strengths are generally not available due to the limited travel of standard 300 mm × 300 mm direct shear devices. Torsional ring shear provides the only reasonable means to obtain residual shear strengths of some GCLs and GCL interfaces, and can provide a reasonable substitute for direct shear when small-specimen testing is warranted.
- One of the most important features of a GCL shear device is the specimen gripping surfaces (i.e. rough surfaces that cover the shearing blocks). Ideally, gripping surfaces should be rigid, provide good drainage, and prevent slippage between the test specimen and the shearing blocks. Because some gripping surfaces do not provide sufficient resistance to slippage, a wrap-round mechanism, bolted bar, or mechanical compression clamps are often used to hold the ends of the geosynthetics during shear. These clamping procedures may result in the development of tension in the geosynthetics, and may cause progressive failure of the specimen. The effect of progressive failure is to reduce the peak shear strength and increase the large displacement (but not residual) shear strength.

- An important source of uncertainty in GCL and GCL interface shear testing is associated with the selection of materials and, in particular, GCL samples with a representative amount of reinforcement. Project-specific shear tests are meaningful only if test specimens are representative of field materials. Specimens that are tested to obtain or verify design strength parameters should ideally be selected from rolls delivered to or designated for the actual project site. However, given that shear tests to obtain design parameters must usually be conducted well in advance of construction, conformance testing should be performed at the time of construction to establish that the delivered materials are at least as strong as the original test materials.
- Multi-interface shear tests can reduce the amount of testing required and provide a better simulation of field conditions. Such tests will automatically give the peak and residual combination failure envelopes for design, but are limited in that strength parameters are obtained only for the failure surface. Multi-interface tests are more difficult to perform and interpret than single-interface tests, which requires that the engineer and testing laboratory have even more experience to avoid errors.
- It is important to select the proper normal stress range for GCL shear tests because failure envelopes are commonly non-linear and because the normal stress level can affect the failure mode of a test specimen. Normal stress sequence during GCL hydration and consolidation may also affect measured shear strength and should generally follow the loading sequence expected in the field.
- GCL specimens should be fully hydrated under the normal stress expected in the field at the time of hydration. Full hydration should always be expected in the field unless the bentonite is encapsulated between two geomembranes. Encapsulated GCLs may be sheared in the dry and fully hydrated moisture conditions and the data used to construct pro-rated peak and residual strength envelopes for design.
- After hydration, a GCL specimen should be consolidated to the shearing normal stress (if applicable) using small load increments to minimize bentonite extrusion. The specimen should be fully consolidated under the final increment, which may take several days, so that excess pore pressures are dissipated prior to the start of shearing.
- The most appropriate maximum shear displacement rates for GCL internal and interface shear tests remain a point of continuing debate. Available data indicate that dry encapsulated GCLs and hydrated geomembrane/needle-punched GCL interfaces show essentially no displacement rate effects and can be sheared at 1 mm/min. No information is available on displacement rate effects for other GCL interfaces. The maximum allowable displacement rate for internal shear of hydrated GCLs remains unclear. Most studies show that internal shear strength increases with increasing displacement rate, although some key studies have produced contradictory results. Taken together, the data suggest that internal shear tests of hydrated GCLs should ideally be conducted at a maximum displacement rate of 0.01 mm/min. This rate may, however, be too slow for production purposes. Until this issue is resolved, a maximum displacement rate of 0.1 mm/min is recommended for hydrated GCL internal shear tests. A maximum shear displacement rate of 1 mm/min is recommended for multi-interface tests containing GCLs. If failure occurs within a hydrated GCL, the multi-interface test should be repeated using a maximum displacement rate of 0.1 mm/min.
- Failed test specimens should be inspected carefully after shearing to assess the surface(s) on which failure occurred and the general nature of the failure. Unusual distortion or tearing of the specimen may invalidate the test results and require that a test be repeated using improved gripping surfaces. Final water contents of the GCL specimen and subgrade soil (if applicable) should be taken after shearing to assess the level and uniformity of hydration that was achieved.

9. FUTURE RESEARCH NEEDS

The following research needs are identified for the shear strength testing of GCLs and GCL interfaces:

- The single most important source of error in current production GCL shear testing is probably associated with the use of inadequate specimen gripping surfaces that allow slippage of the test specimen during shear. There is a need to develop a standardized specimen gripping surface that does not require geosynthetic end-clamping to be successful. Clamping systems may still be used to secure the ends of a test specimen but should not significantly participate in the shearing process.
- Another important source of uncertainty in GCL and GCL interface shear testing is associated with the selection of test materials (especially for reinforced GCL products). There is a need to standardize selection of GCL shear test materials and procedures to verify that materials delivered to a project site are at least as strong as the original test materials.
- High-quality test data are available in the open literature for GCL internal shear strengths and geomembrane/GCL interface shear strengths. Much less information is available on shear strength behavior for other common GCL interfaces (e.g. soil/GCL, drainage geocomposite/GCL), especially at high normal stress conditions (Chiu and Fox 2004). Although published data cannot be substituted for project-specific testing, additional comparative studies are needed to illustrate strength behavior for other common GCL interfaces.
- Although best-practice considerations mandate GCL hydration/consolidation to equilibrium, most testing laboratories use a shorter hydration time (typically

two days). This time period is too short if a GCL is to be hydrated under low normal stress and then consolidated to a higher normal stress. Additional comparative studies are needed to determine the effect of hydration/consolidation procedure on the shear strength of various GCLs and GCL interfaces. Further studies are also needed to identify accelerated GCL hydration/consolidation procedures that do not alter shear strength behavior.

- Selection of shear displacement rates that are most appropriate for field loading conditions remains an open question for GCL shear testing. Nearly all production laboratories use a rate of 1 mm/min. Additional studies are needed to verify the recommendations presented in this paper regarding maximum displacement rates for different GCLs and GCL interfaces. For those materials that show a significant rate effect, a consensus needs to be reached on the appropriate rate(s) for shear testing. The viability of increasing the displacement rate after peak strength to more rapidly reach large displacement/residual conditions also needs to be investigated.
- GCLs are often utilized for landfill construction in seismic regions. Only one study (Lai *et al.* 1998) has been conducted on the response of dry and hydrated specimens of an unreinforced geomembrane-supported GCL to dynamic loading. No information is currently available on the behavior of encapsulated GCLs, reinforced GCLs, or GCL interfaces under dynamic loading conditions.
- The long-term performance of GCLs and GCL interfaces under sustained loads remains largely unknown, especially at elevated temperatures that may be relevant to landfill bottom liner and cover systems. There is an immediate need for additional research on the possible use of strength reduction factors for the calculation of long-term design strengths for GCLs and GCL interfaces in routine design work. Similar to geosynthetic reinforcement applications, such reduction factors could be applied to short-term strength data to account for creep, durability (i.e. chemical and biological degradation), and installation damage (see initial study by Fox *et al.* 1998b).
- Limited studies have indicated that shear strength anisotropy can occur for GCLs and GCL interfaces sheared in opposite machine directions (i.e. rotated 180°). No information has been published on GCL shear strength in the transverse direction. Index tests are needed that will allow for quick identification of the weak shear direction for a GCL or GCL interface. One possibility for GCL interfaces is a tilt table for measurement of index friction angle (Narejo 2003).
- Based on information presented by Eid *et al.* (1999), Fox *et al.* (1998a), Heerten *et al.* (1995), Richardson (1997), von Maubeuge and Eberle (1998), and von Maubeuge and Lucas (2002), there probably exists a correlation between peel strength and peak internal shear strength parameters (c_p and ϕ_p) for NP GCLs. Research is needed to determine the reliability of and possible practical uses for such correlations. Another possibility may be the shear tensile index test (Eichenauer and Reither 2002), although von Maubeuge and Ehrenberg (2000) report inconsistent results with this method.
- Comparative studies are needed to assess the viability of a multi-interface test for GCLs and GCL interfaces. Standardized procedures will be needed if practice moves toward such a test as an alternative to the current single-interface test.

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NOTATIONS

Basic SI units are given in parentheses.

a_{ld}	adhesion intercept for large displacement interface failure envelope (N/m^2)
a_o	constant (N/m^2)
a_p	adhesion intercept for peak interface failure envelope (N/m^2)
a_r	adhesion intercept for residual interface failure envelope (N/m^2)
a_∞	constant (N/m^2)
c_{ld}	cohesion intercept for large displacement internal failure envelope (N/m^2)
c_p	cohesion intercept for peak internal failure envelope (N/m^2)
c_r	cohesion intercept for residual internal failure envelope (N/m^2)
c_{tan}	cohesion intercept for tangent linear failure envelope (N/m^2)
$c_{\sigma_1-\sigma_3}$	cohesion intercept for linear failure envelope drawn between σ_1 and σ_3 (N/m^2)
F_p	peel strength of NP GCL (N/m)
p	constant (dimensionless)
P_a	atmospheric pressure (N/m^2)
R	maximum shear displacement rate (m/s)

t_h hydration time (s)
t_{50} time required for double-drained GCL specimen to reach 50% consolidation (s)
w_f final GCL water content (dimensionless)
δ_{ld} friction angle for large displacement interface failure envelope (°)
δ_p friction angle for peak interface failure envelope (°)
δ_r friction angle for residual interface failure envelope (°)
δ_∞ constant (°)
Δ shear displacement (m)
Δ_f estimated displacement at peak or large displacement shear strength (m)
Δ_p displacement at peak shear strength (m)
Δ_r displacement at residual shear strength (m)
$\Delta\phi$ constant (°)
ϕ_{ld} friction angle for large displacement internal failure envelope (°)
ϕ_o constant (°)
ϕ_p friction angle for peak internal failure envelope (°)
$\phi_{p,\sigma_1-\sigma_3}$ peak internal friction angle corresponding to normal stress range $\sigma_1 - \sigma_3$ (°)
ϕ_r friction angle for residual internal failure envelope (°)
$\phi_{r,\sigma_1-\sigma_3}$ residual internal friction angle corresponding to normal stress range $\sigma_1 - \sigma_3$ (°)
ϕ_{sec} secant internal friction angle (°)
ϕ_{tan} internal friction angle for tangent linear failure envelope (°)
$\phi_{\Delta,\sigma_1-\sigma_3}$ internal friction angle corresponding to displacement Δ and normal stress range $\sigma_1 - \sigma_3$ (°)
$\phi_{\sigma_1-\sigma_3}$ internal friction angle for linear failure envelope drawn between σ_1 and σ_3 (°)
η constant (dimensionless)
$\sigma_{n,h}$ hydration normal stress (i.e. normal stress during hydration, N/m^2)
$\sigma_{n,s}$ shearing normal stress (i.e. normal stress during shearing, N/m^2)
σ_o constant (N/m^2)
$\sigma_1, \sigma_2, \sigma_3$ values of shearing normal stress (N/m^2)
τ shear stress (N/m^2)
τ_{ld} large displacement shear strength (N/m^2)
τ_p peak shear strength (N/m^2)
τ_r residual shear strength (N/m^2)
τ_Δ shear strength at displacement Δ (N/m^2)

ABBREVIATIONS

GCL = geosynthetic clay liner
GM = geomembrane
GMS = smooth geomembrane
GMX = textured geomembrane
GMXC = coextruded textured geomembrane
GMXL = laminated textured geomembrane
GT = geotextile
HDPE = high-density polyethylene (GM)
LIR = load-increment ratio
NP = needle-punched (GCL)
NW = nonwoven (GT)
PVC = polyvinyl chloride (GM)
SB = stitch-bonded (GCL)
W = woven (GT)

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