Recommended design strength for needlepunched geosynthetic clay liner products

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A needlepunched geosynthetic clay liner (GCL) is a composite material comprised of bentonite sandwiched between two geotextiles and reinforced by needle punching with synthetic fibers. The strength behavior of any GCL product is complex. Designs using a GCL must consider the internal strength of the composite product and the interface strength between its outer surfaces and adjacent materials.

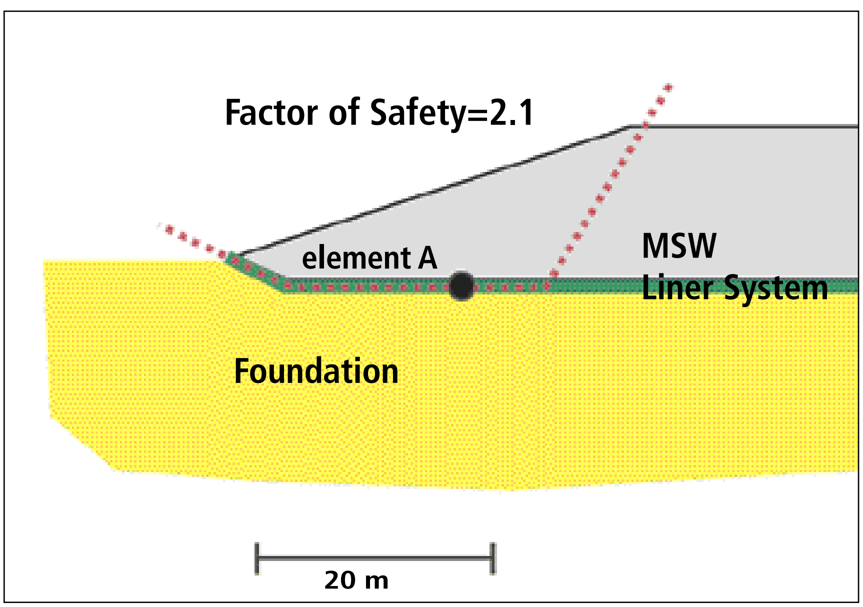
Figure 1 shows a cross section for a typical landfill with a composite liner system made up of the subgrade covered with a GCL, a geomembrane and a drainage geocomposite to form the primary liner system. A typical failure surface determined by stability analysis is shown. To illustrate concepts, consider an "average" element "A" located midway along the portion of the failure surface in the liner system. It experiences a normal stress of 69 kPa and an average shear stress of 12.9 kPa. These are stresses created by the force of gravity acting on the waste mass. An earthquake that causes an average acceleration above the liner system of 0.3 g will increase this average shear stress to 32 kPa for one or more instants in time, based on results from a pseudostatic stability analysis.

Figure 2 shows some typical test results obtained on materials in the liner system from laboratory tests using a direct shear box. One test shows the internal strength of the GCL where failure is forced to occur within the bentonite and free swell of the GCL under low normal stress has been prevented. The others show the interface strength between the GCL and other materials, including a textured geomembrane, a geocomposite and a clay soil. All tests used a normal stress of 69 kPa applied to hydrated materials and a shearing rate of 0.04 in./min. Hydration and consolidation of the materials were controlled to prevent squeeze out of bentonite onto the interfaces.

The internal peak strength of the GCL of about 150 kPa is the highest of all the potential failure surfaces included in **Figure 2**. But after reaching a high peak internal strength, the GCL loses strength with continued displacement. At large displacements the internal strength of the GCL is the lowest of all at 10 kPa and continuing to decrease. The high internal strength is provided by the needle punching fibers that act like reinforcement and hold the material together. After these fibers become stretched to the point that they pull out or break, their contribution to internal strength of the GCL decreases. With further displacement the strength contribution of the reinforcing fibers may become almost totally lost. The result is an internal shear strength at large displacement that is controlled by the shear strength of bentonite. Bentonite has a very low shear strength, which is characterized by a friction angle of 8° at a normal stress of 70 kPa and decreasing to 4° at 500 kPa (Olson 1974). These low shear strength values represent the lowest internal shear strength of the GCL and thus the residual internal strength. For the test shown in **Figure 2**, the internal strength of the GCL at 90 mm of displacement is 9°, but is still decreasing. While it is not the true residual strength, the flat slope of the stress-displacement curve indicates residual strength is almost reached. The close agreement in strength of the GCL at 90 mm of displacement and strength of bentonite obtained by the careful research work of Olson 30 years ago in a small triaxial cell suggests that the large shear box is giving a realistic measurement of residual shear strength of the GCL (or we are lucky enough to have compensating unknowns acting together in the shear box).

The peak interface strength of the GCL with adjacent materials shown in **Figure 2** is less than the peak internal strength of the GCL. The peak interface strength between the GCL and the textured geomembrane is less than half the peak internal strength of the GCL. The peak interface strength between the GCL and the clay is about 1/3 the peak internal strength of the GCL. The peak interface strength between the GCL and the

Figure 1. Cross section for a typical landfill. Composite liner system consists of subgrade covered with a GCL, a geomembrane and a drainage geocomposite to form the primary liner.



geocomposite is about 1/5 the peak strength of the interface. If we sandwich these materials together to form a composite liner system and subject them to a shear stress, sliding failure will occur when the applied shear stress exceeds the peak strength of the weakest material or interface. Once failure is initiated, displacement will continue along that slip plane. For the materials and stress conditions used to obtain the data in **Figure 2**, the weakest location is the interface between the GCL and the drainage geocomposite. Substantial and rapid movements would develop along this interface once the shear stress exceeded the peak shear strength of 30 kPa (equivalent to a friction angle of 23° in this case). Movement would continue until something occurred to reduce the shear stress to less than about 23 kPa (the residual strength of this interface at larger displacements which is equivalent to a friction angle of 18° in this case). Since the GCL has an internal peak strength almost five times higher than the peak strength of this interface, it is inconceivable that a failure would occur inside the GCL, even though it has a very low residual strength.

The data for the cases shown in **Figure 2** indicate a design approach to use that will avoid shearing the GCL to its residual strength—select an adjacent material or interface that has a lower peak strength than the internal strength of the GCL and does not experience a large loss of strength with continued displacement. We, in effect, design the system to fail somewhere other than through the GCL. For the materials used in **Figure 2** and a normal stress of 69 kPa, shear failure would occur at the interface between the GCL and the geocomposite when the shear stress reachs 30 kPa.

Returning to the example in **Figure 1**, the average static shear stress in the liner system is 12.9 kPa. All components of the liner system have sufficient short-term peak strength to withstand this shear stress. That the GCL has a residual strength less than 12.9 kPa is not an issue because the GCL must first be stressed through its peak strength. Failure will occur at other weaker interfaces before that happens. The 0.3g earthquake increases the shear stress on the average point to 32 kPa. This is sufficient shear stress to exceed the shear strength of the GCL-geocomposite interface, and some slippage could occur at this interface until the temporary force from earthquake shaking is removed. As indicated by the results in **Figure 2**, this slippage and that from additional earthquake cycles may cause a reduction in the interface strength of the GCL-geocomposite to as low as 23 kPa. However, this reduced strength is still more than adequate to resist the static shear stress of 12.9 kPa that is maintained by gravity.

This example illustrates that failure will occur along the interface or in the material with the lowest peak strength and not the one with the lowest residual strength. Use of the lowest peak strength for design applies to most GCL applications in caps and cover systems and in bottom systems of enclosed landfills as supported by Koerner (2002). However, the example also indicates that if the interface or material with the lowest peak strength loses strength after straining through a peak, we should design for gravity forces using the residual strength of that interface or material, if there is any opportunity for that interface or material to be stressed beyond its peak strength. This situation may occur where progressive failure is anticipated (e.g., in seismic events, large settlements such as in waste materials resulting in downdrag, construction induced deformations, migration of bentonite from the GCL to the interface, non-uniform distribution of stresses such as in valley fills and sudden increases in pore pressure as discussed by Thiel and von Maubeuge 2002, and Gilbert 2001).

Design using the lowest peak strength assumes that the peak strength of the interfaces or materials do not change with time. The data in **Figure 2** were obtained by shearing in laboratory tests over a few hours. An obvious question is what happens to these materials over the much longer time that they must perform in the field. It is well known that polymeric materials in tension will eventually fail in creep at lower stresses than their short-term tensile strength. It is also known that the strength of polymeric materials can decrease with aging. There is very little information on the behavior of these materials in a confined condition subjected to shear stresses over a long time. Such tests are expensive to perform and take months to years to run. Data from the few long-term tests that have been done in shear boxes were for conditions that did not produce failure in the material (Trauger et al. 1966 and Herten et al. 1995). It seems very unlikely that creep will reduce the interface strength of a geosynthetic material against another geosynthetic material below the value measured in a large shear box displaced to the residual value. It also seems very unlikely that creep will reduce the interface strength between a geosynthetic and a soil below that measured in a large shear box displaced to the residual value at a rate slow enough to avoid the creation of excess pore water pressures along the interface during shear. This leaves open the question of the effects of long-term creep and aging on the internal strength of the GCL (and of geocomposites and geomembranes for that matter).

A significant decrease in strength of the polymeric materials in GCLs due to aging is unlikely because of the oxygen level in saturated bentonite is lower than the usual estimated amount in soil of 8%. The primary aging mechanism for polypropylene (PP) fibers used in GCLs is oxidation. Manufacturers reduce the oxidation degradation potential through the use of antioxidant additives in the polymer. The service life (50% strength loss) of one product with PP fibers with a good antioxidation package in an oxygen rich environment (circulating air at 21% oxygen) was estimated to be on the order of 90 years in research performed by the Federal Highway Administration FHWA (Elias et al. 1999). In lower, stagnant oxygen environments, such as in granular soil, the life of the polymer is significantly extended. The FHWA found a service life of 240 years for the previous polymer in 8% oxygen environment at 20° C. More recent studies by Thomas (2003) have confirmed longevity at low oxygen content. Based on aging tests performed on PP fibers taken from a GCL, they found a design life in buried applications with 8% air to be more than 300 years. The actual performance life may be even longer, because in partially saturated GCL, only a few percent oxygen would be anticipated with essentially no gas circulation (Hsuan and Koerner 2002). In a saturated GCL there would essentially be no oxygen. The low oxygen level is why straw in adobe and wood beneath water has lasted for thousands of years. The condition of fibers within the GCL should be of less concern than polymer degradation at other interfaces outside of the bentonite where the oxygen content would be greater. If there is a question concerning aging of polymers on projects, aging test can be performed on the polymer following the FHWA testing protocol as outlined in Elias et al. (1999). Temperature significantly effects aging. Where elevated temperature is anticipated at the liner, aging tests are recommended on all polymeric materials.

As discussed above, the large internal peak strength of a GCL comes from the polymeric fibers used to bind the bentonite between the external geotextiles. Shearing of the GCL places these fibers into tension. Much of the difference between peak and residual internal strength of a GCL must be created by these fibers working in tension. Over time, creep may cause some of these fibers to break or pull out, which will reduce the internal peak strength of the GCL, as shown by Thies et al. (2002). Creep of the fibers may be reduced by the composite soil-reinforcement interaction. As noted by Thiel and von Maubeuge (2002), the shear strengths exhibited at high normal loads, even under fully hydrated conditions, are much greater that the sum of the bentonite shear strength and geotextile tensile strength. The reduction in long-term strength due to creep can best be addressed by performing long-term shear strength tests and developing creep reduction factors to be applied to short-term peak strength test results, as is currently done for soil reinforcement applications. Creep is a time dependent issue that does not actually reduce the strength of the materials until incipient failure is approached. By applying a creep reduction factor, a stress level is achieved that can prevent long-term failure due to creep and can provide full peak strength during transient loads such as seismic events. An additional factor could be applied to account for aging.

In the absence of long-term data for GCLs, a reduction factor on the order of 3 could be applied to the portion of the GCL internal strength from reinforcement to account for the contribution to peak strength from the polymeric fibers (assuming the reduction factor for PP in tension in Koerner 1998). In the absence of aging data and considering the buried, low oxygen condition, an aging factor of 1.1 to 2.0 as recommended by FHWA (2001) would appear to be conservative reduction for a 100 year to 300 year performance period, respectively. In **Figure 2** the difference between peak and residual internal strength for the GCL is 140 kPA, which is the assumed contribution of the polymeric fibers to the

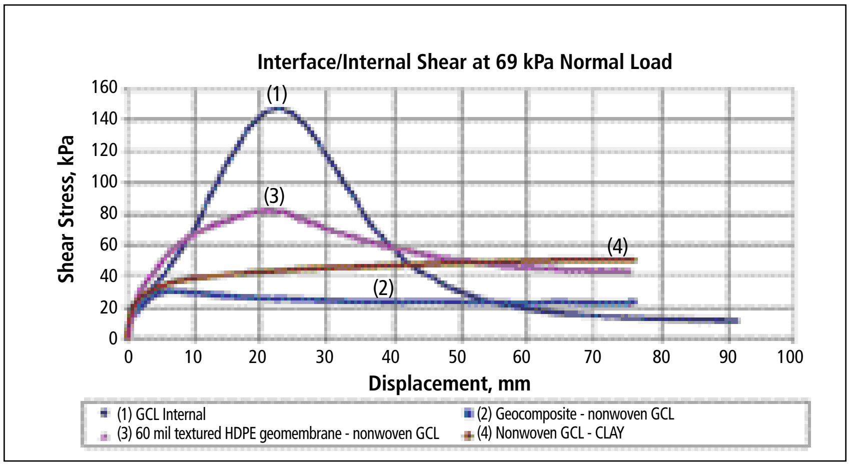


Figure 2. Typical test results obtained on materials in the liner system from laboratory tests using a direct shear box.

GCL internal peak strength. We could obtain a lower bound estimate of the long-term creep and aging reduced internal strength of the GCL by adding 140/3.3 for the 100 year performance period to the residual strength of 10 kPa, which equals 52 kPa at a normal stress of 69 kPa This value is well above the peak strength on the GCL-geocomposite interface. It is highly unlikely that long-term creep or aging would further reduce the strength of the GCL to the point that failure would occur internal to this GCL. For the 300 year case, a reduced internal strength of at least 33 kPa would be anticipated, which also exceeds the peak strength of the GCL-geocomposite interface. Even with conservative reduction factors to account for strength loss due to creep and aging, the internal strength of this GCL is higher than most geosynthetic interfaces. Further protection is provided by requiring a factor of safety in stability analyses that is greater than one.

If all designs include at least one interface with a peak strength less than this reduced value for internal peak strength of the GCL, it will be unlikely that conditions will ever develop that would reduce the internal strength of the GCL to its residual value. Failure would most likely occur at other locations first. This conclusion has a conservative bias because it ignores the fact that the strength for other interfaces may also reduce with time due to creep and aging.

Some designers choose the internal strength of the GCL at some value of displacement in the shear box, such as 2 in., as their design strength for the GCL. In **Figure 2**, the internal shear resistance provided by the GCL at 2 in. of shear box displacement is 30 kPa. We do not agree with this approach. The displacement pattern developed within the GCL in a shear box test does not match the displacement patterns that develop in the field. Assuming the internal strength measured at 2 in. in the shear box equals the strength available at 2 in. of field displacement is incorrect. Test details, including how the GCL is gripped to cause failure by internal shear, affect the shape of the measured load-displacement curve. Shear box tests by different laboratories on the same material for the same test conditions can give substantially different results at displacements beyond the peak strength. This is an issue being addressed by ASTM that appears to result from differences in gripping systems used by different laboratories. Regardless of the outcome of this testing issue, it is not good practice to design any material for conditions where an additional increment of displacement causes a decrease in shear strength. Such a condition is inherently unstable.

In summary, we recommend the following approach to obtain long-term internal design strength for GCLs:

Measure the short-term peak strength of the GCL in a fully hydrated state at normal stresses representative of field conditions and a displacement rate of 0.04 in./min. in accordance with ASTM D 6243.

Apply reduction factors based on long-term tests to this peak strength to obtain the long-term internal design strength of the GCL. In the absence of project-specific test data, use a factor of three for creep together with a factor of 1.1 for 100 years of aging and 2.0 for 300 years of aging applied to the difference between peak and residual strength. Add this to the residual strength to this reduced value to obtain the long-term internal design strength of the GCL. [Note: Temperature, normally assumed to be at 20° C, will affect creep results (Thies 2002), and should be considered in selecting appropriate reduction factors.]

Provide another material or interface with a short-term peak strength less than the long-term internal design strength of the GCL to prevent failure from occurring inside the GCL. Define the strength of this material or interface as the design peak strength. Define the residual strength of this material or interface as the design residual strength. Use a mimimum factor of safety for global stability of 1.5 for design with the design peak strength and 1.1 for design with the design residual strength.

For earthquake loads with a pseudo static factor of safety less than 1 using the design residual strength, perform a deformational analysis using the design residual strength.

For conventional landfill design, we think it unnecessarily conservative to design with the internal residual strength of a GCL that is sufficiently needlepunched to give it a high short-term peak strength relative to adjacent interfaces, provided the GCL is not permitted to free swell under normal stresses less than 10 kPa.

References

Elias, V., Christopher, B.R. and Berg, R.R. with DiMaggio, J.A. as technical consultant. 2001. *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines*. FHWA-NHI-00-043. U.S. Department of Transportation. Federal Highway Administration, Washington D.C.

Elias, V., Salman, A., Juran, I., Pearce, E., and Lu, S. 1999. *Testing Protocols for Oxidation and Hydrolysis of Geosynthetics*. FHWA-RD-97-144. National Technical Information Service, Springfield, Va.

Gilbert, R.B. 2001. "Peak Versus Residual Strength for Waste Containment." *Proceedings of 15th GRI Conference: Hot Topics in Geosynthethics II*. Geosynthetic Institute, Folsom, Pa. p.29–30.

Heerten, G., Saathoff, F., Scheu, C., von Maubeuge, K.P. 1995. "On the long-term shear behavior of geosynthetic clay liners (GCLs) in capping sealing systems." Nuremberg, p. 141–150.

Hsuan, Y.G. and Koerner, R.K. 2002. "Durability and lifetime of polymer fibers with respect to reinforced geosynthetic clay barriers, i.e. reinforced GCLs." *Clay Geosynthetic Barriers*. Zanzinger, Koerner and Garling (eds). Swets & Zeitlinger, Lisse. p. 73–86.

Koerner, R.M. 1998. *Designing with Geosynthetics*. Prentice Hall, New Jersey.

Koerner, R.M. 2002. "Recommendation to Use Peak Shear Strengths for Geosynthetic Interface Design." *Proceedings of 16th GRI Conference: Hot Topics in Geosynthetics III*. Geosynthetic Institute, Folsom, Pa.

Marr, W.A. 2001. "Interface and Internal Shear Testing Procedures to Obtain Peak and Residual Values." *Proceedings of 15th GRI Conference: Hot Topics in Geosynthethics II*. Geosynthetic Institute, Folsom, Pa. p. 1–28.

Olson, R.E. 1974. "Shearing strengths of kaolinite, illite, and montmorillonite." *Journal of Geotechnical Engineering Division*. American Society of Civil Engineers, Alexandria, Va. v. 100, no. GT 11, pp. 1215–1229.

Thiel, R. and von Maubeuge, K.P. 2002. "Current Industry Performance and Construction Issues Related to GCLs." *Proceedings of 16th GRI Conference: Hot Topics in Geosynthetics III*. Geosynthetic Institute, Folsom, Pa.

Thies, M., Gerloff, C., Muller, W., and Seeger, S. 2002. "Long-term shear testing of geosynthetic clay liners." *Clay Geosynthetic Barriers*. Zanzinger, Koerner and Garling (eds.). Swets & Zeitlinger, Lisse. pp 73–86.

Thomas, R.W. 2003. "Thermal Oxidation of a Polypropylene Geotextile Used in a Geosynthetic Clay Liner II—Oxidation Rates in 8% Oxygen and the Effect of Air Flow Rate on Oxidation." *Second Symposium on Geosynthetic Clay Liners*. ed. Mackey, R.E. and von Maubeuge, K.P. ASTM International, Denver.

Trauger, R.J., Swan, R.H. & Zehong, Y. 1996. "Long-term shear strength behavior of a needlepunched geosynthetic clay liner." Testing and Acceptance Criteria for Geosynthetic Clay Liners. Ed. Well, Larry W. ASTM International. STP 1308, 103–120.

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