DESIGNER'S FORUM

Designing with needlepunched reinforced GCLs: Stability fundamentals

By Gregory N. Richardson, Richard Thiel, and Robert Mackey

THE USE OF GEOSYNTHETIC CLAY liners (GCLs) as a barrier component in landfill-liner and final-cover systems is now well-established and generally is permitted with little controversy. This development has been aided by ASTM D-35 Committee on Geosynthetics standards and a growing list of successful applications and performance data. However, GCLs, when used alone or as components in a composite barrier system, cause significant stability concerns for the designer.

This Designer's Forum will review analysis, material property, and construction-quality assurance (CQA) issues concerning stability for needlepunched reinforced GCLs.

GCL types to consider for slope stability

There are essentially two categories of GCL to consider from the point of view of long-term stability: reinforced and non-reinforced. The primary functional difference is their internal or mid-plane shear performance, especially when hydrated.

The presence of reinforcement, or lack thereof, between the two geotextiles that form the GCL also will affect how the material handles during construction (deployment and covering). This is especially the case when the bentonite becomes hydrated accidentally. The use of a reinforced GCL eliminates the potential of a bearing-capacity stability failure caused by vehicle traffic over partially hydrated GCLs in liner systems or final covers. This stability case has been discussed previously by Richardson (1996) and will not be covered here.

For non-reinforced GCLs to provide adequate slope stability, either the slope must be very flat, or the clay must be encapsulated in geomembranes to severely limit bentonite hydration. Although one of the authors has developed a design methodology for designing with non-reinforced GCLs that are encapsulated in geomembranes, the discussion of that methodology is beyond the scope of this article. **Photo 1** shows a reinforced GCL that is encapsulated between two geomembranes, which limit wetting by seepage from the canyon walls on which the liner system is constructed.

If there are no top and bottom geomembranes to limit hydration, one must assume that the bentonite hydrates and moves relatively easily under differential normal stresses. Extreme care is required to design, deploy, and cover over a non-reinforced GCL that is allowed to hydrate to 40% moisture or more.

Both needlepunched and stitchbonded types of reinforced GCLs are available in the United States. However, market dynamics, cost and product development have strongly favored the use of needlepunched products in landfill covers and liners. All of the products can be used if their design and construction limitations are properly understood. However, the authors have chosen to focus this article on stability issues related to needlepunched GCLs.

The subject materials are supported by at least one nonwoven needlepunched (NWNP) geotextile on one side, and either a woven or NWNP geotextile on the other side. The GCL is reinforced by needlepunching through the bentonite, thus connecting the two sides of the GCL.

Stability models

Two general conditions must be considered for landfill-slope stability. One is for veneer-cover conditions, the other for landfill-bottom liners. The main difference between these conditions is magnitude of normal and shear stresses that will be transmitted through the GCL.

Cover- (veneer-) slope stability

The infinite-slope model is the simplest and most conservative method of evaluating GCL stability in the veneer-type systems common to landfill final-cover systems. The pseudo-static factor of safety and yield acceleration for the cover may be assessed using the following general equation for the stability of an infinite slope:

$$FS = \frac{c/(\gamma \times z \times \cos^2\beta) + \tan\phi\,[1-(u/(\gamma \times z))] - k_s \times \tan\beta \times \tan\phi}{k_s + \tan\beta}$$

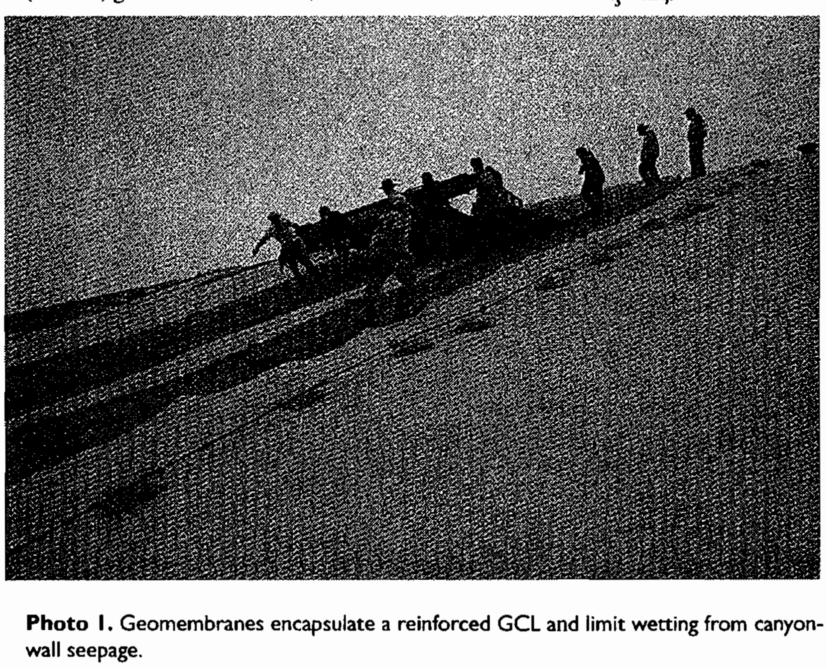

Photo 1. Geomembranes encapsulate a reinforced GCL and limit wetting from canyon-wall seepage.

where FS = factor of safety; k_S = seismic coefficient; γ = unit weight of slope material(s); c = cohesion; ϕ = angle of internal friction of the assumed failure interface or surface; z = depth to the assumed failure interface or surface; and u = pore water or gas pressure at the failure interface. The above equation yields the factor of safety explicitly for both cohesive ($c \neq 0$) and cohesionless soils ($c = 0$).

For final-cover designs, a nominal gas pressure should be assumed, since even those facilities equipped with active gas-recovery systems will experience maintenance shutdowns of sufficient duration to allow a build up of internal gas pressures. A minimum value of 20 psf (4 in. water pressure) commonly is assumed for u. By using the infinite-slope equation, the static side-slope shear stresses generated on and within a GCL barrier as part of a typical final cover can be predicted (**Figure 2**). For a 4H:1V side slope, design-shear stresses typically range from 50 to 80 psf.

Reinforced GCLs commonly are specified to have a minimum internal-shear strength of 500 psf so that the critical stability factor will be the minimum interface-friction angle. It is particularly important to verify this interface-friction angle for site-specific conditions when the reinforced GCL incorporates a woven geotextile face.

The designer must identify the critical shear plane, e.g., the plane that produces the lowest factor of safety. For a GCL, there are three potential planes of failure: either of the two surfaces, and the core. Therefore, the physical properties of concern are the interface-friction angles for each GCL surface and the internal-shear strength of the GCL. When possible, the critical failure surface should occur above the liner elements to minimize the impact of failure and cost of repair.

A minimal regulatory acceptable factor of safety for long-term loading is 1.5. However, designers are cautioned that this factor of safety has been questioned by some (see Liu et al. 1997), due to the infinite-slope equation's sensitivity at low normal loads. Uncertainties related to gas or pore-water pressures also raise doubts.

For seismic conditions, the factor of safety based on pseudo-static analysis can be as low as 1.0. This reflects the Resource Conservation and Recovery Act (RCRA) Subtitle D-mandated use of the U.S. Geologic Survey (USGS) 2120 peak-bedrock-acceleration map for a 90% probability event over a 250-year interval. Conditions represent a massive earthquake that has a return period exceeding 2400 years, such that the factor of safety of 1.0 is justified. If the factor of safety is less than 1.0 using the pseudo-static seismic analysis, then a displacement-type analysis must be performed to determine if the resulting seismic displacements are acceptable.

Bottom-liner slope stability

The stability of a bottom-liner system is highly site-specific. The value must take into account not only the ultimate landfill configuration, but intermediate operational configurations as well. It is very common that an intermediate configuration (intermediate being a relative term—today, often meaning the better part of a decade) is more critical than the final configuration.

Presume that the designer has accurately selected the critical slope-stability cross-section for evaluation and has correctly input the geometry and material properties into a slope-analysis software program. They still must determine which shear-strength properties should be used for the various interfaces. As related specifically to the GCL, the designer is left to question the appropriate normal loads and whether to use the peak or post-peak shear strength of the interface. Internal shear-strength values obtained from laboratory direct-shear testing also must be evaluated.

Several additional points deserve mention here. First, the designer should strive to have the critical shear plane above the barrier-layer system (in this case, consisting of at least a GCL, and usually with the GCL covered by a geomembrane). That way, any movement along the shear plane (either during construction, landfill operations, or as a result of a seismic event) will not rupture the barrier-liner system.

Second, the designer should conduct shear-strength testing during the design phase to validate the design and obtain an understanding of how the various components work together.

Third, the interface with the lowest peak shear strength will govern the strength of the section. Complex stability models that include all the layers associated with modern landfills are not required—the weakest interface controls the stability in such a way that only the single interface must be modeled.

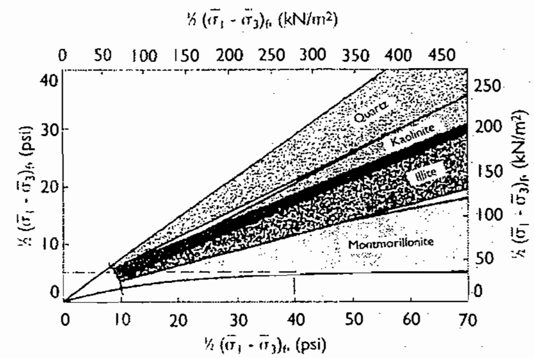

Figure 1. Typical effective stress-failure envelopes for hydrated clays (Olson 1974).

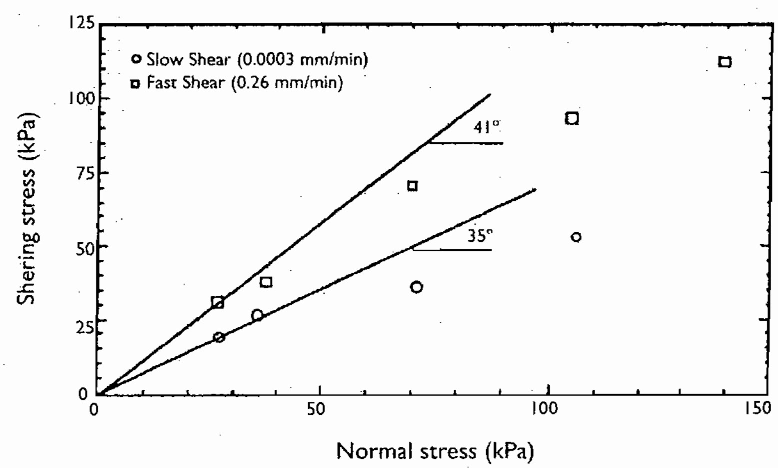

Figure 2. Results of direct-shear tests at normal stresses lower than in Figure 1 (Daniel et al. 1993).

Key stability-related physical properties

The previous discussion should make it apparent that the frictional properties of the GCL's surfaces and its internal-shear strength are critical to the physical stability of the overall system.

GCL frictional properties

The frictional properties of the GCL surface are influenced by the type of geotextile that forms it and the possible presence of hydrated bentonite that has moved through the composite. Available products use nonwoven geotextiles and woven geotextiles made of slit-film filaments. The latter provides both a lower interface friction and a greater potential for hydrated bentonite migrating to the GCL's surface.

Years of direct-shear testing by our industry have shown that a nonwoven geotextile provides exceptional interface friction with textured geomembranes (referred to as the "Velcro effect") and most soils. The authors are not aware of a single stability failure that developed between a nonwoven geotextile and soils or textured geomembranes.

On the flip side (no pun intended), the authors do know of several failures between the woven side of a GCL and a geomembrane (whether textured or smooth). A few of the most notable nonconfidential examples involve the final-cover test plots in the EPA Cincinnati field test (Daniel et al. 1998). For this reason, acceptable service of nonwovens with smooth geomembranes is limited to slopes of less than 10 degrees.

To maximize stability of GCL/geomembrane composites on slope applications, reinforced GCLs constructed using a nonwoven geotextile on one face and a woven geotextile on the other should be placed with the nonwoven geotextile in contact with a textured geomembrane. The woven geotextile in such a composite has a significant number of cross-fiber "tufts" that roughen the woven's surface and typically provide a satisfactory interface with soils. However, both major U.S. manufacturers offer double-nonwoven products which, in the authors' opinion, provide superior slope stability, durability, and constructability performance, while still maintaining their environmental integrity and cost-effectiveness.

Internal-shear strength

The internal-shear strength of needle-punched reinforced GCLs is influenced by the bentonite clay and the needled fibers that penetrate through the composite's thickness. Each of these components provides an internal-shear strength that is affected by the degree of clay hydration, as well as by the normal load acting upon the GCL and the shear strain that has occurred across the composite. Unfortunately, precise details on each of these components for a given product are not available to the designer. Laboratory tests performed on GCLs measure the simultaneous contribution of all internal-shear strength components and do not provide a clear understanding of internal mechanisms.

The clay, exclusively bentonite, that forms the hydraulic-barrier component of the GCL has a hydrated-shear strength that is influenced by the degree of hydration and the normal loading. The shear strength of hydrated clays was evaluated by Olsen who produced typical effective stress-failure envelopes, as shown on **Figure 1** (1974).

Olsen's work focused on the larger normal loads appropriate for line systems. Results showed the *lower limit of bentonite clays' effective shear strength to be approximately 5 psi (720 psf) for normal loads exceeding approximately 40 psi (5760 psf).* At normal loads less than approximately 20 psi, the effective stress envelope is characterized as frictional ($c = 0, \phi \neq 0$), while at normal pressures greater than 40 psi the envelope is cohesive ($c \neq 0, \phi = 0$).

Later work by Daniel et al (1993) focused on normal stresses lower than those addressed by Olsen. Their results showed that the *drained-friction angle of the bentonite approaches 35 degrees at normal loads less than 5 psi* (see Figure 2). Between 5 psi and 40 psi, the shear strength of bentonite will transition between the two shear strength limits given above.

Stitched or needled fibers that penetrate through the thickness of a reinforced GCL contribute to the shear strength as the two geotextile surfaces move apart differentially. The amount of shear strength added by the fibers at low strains also may be influenced by the anchorage or tensioning of the fibers to the geotextiles. For example, Bentofix uses a proprietary process to thermally lock the needled fibers to the geotextile surfaces. Such fibers may develop a larger shear resistance at a smaller relative displacement of the two geotextiles than those that simply are needled.

In stitched GCLs, a similar stiffening may occur if the stitching tension is increased. At present, no data exists to clearly show the relationship between needling and stitching variables.

The contribution of needled-reinforcement fibers to the peak shear strength of a GCL is shown on **Figure 3**. Here, the internal total-stress peak shear-strength data is compared to the effective shear strength of bentonite, as determined by Olsen. The higher peak shear strength of the GCLs must be due to the contribution of the needled fibers. The contribution of the needled-reinforcement fibers is very significant across the full range of normal loads.

The continued shear of a reinforced GCL beyond the peak stress point produces a lower residual strength. Residual shear strengths for Bentofix and Bentomat are plotted as a function of normal load on **Figure 4**. (p. 25) Here, the residual total strengths are compared with Olsen's effective stress-failure envelope for montmorillonite and the peak strength values of an unreinforced GCL.

Data presented by Scranton indicates that the residual strength of an unreinforced GCL is approximately 0.6–1.0 times the peak strength (1996). The data on Figure 4 clearly shows that, at large internal-shear displacements, the shear strength of a reinforced GCL approaches that of one that is not reinforced. This also was observed by Gilbert et al. (1996).

The polymers associated with the needled fibers and those with the bentonite may creep (i.e., deform) when subjected to long-term loadings. Recently published reports by Koerner (1996), Siebken et al. (1996), and Trauger et al. (1996) have shown that the majority of internal-shear displacements occur during the first 100 hours of loading. This testing implies that if field conditions do not change and the installation survives the initial week of loading, the reinforced GCL is stable.

This is one observation to come out of the GCL slope tests performed by the U.S. Environmental Protection Agency (EPA)

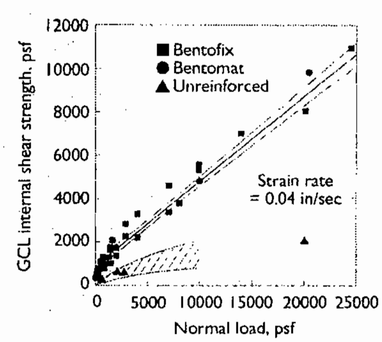

Figure 3. GCL peak shear strength vs. normal load.

over the past two years in Cincinnati (see Scranton 1996). At this site, reinforced GCLs have remained stable with little or no ongoing deformation on final cover slopes as steep as 2H:1. This implies a minimum static slope-stability factor of 1.5 when applied to 3H:1V slopes.

Design-allowable shear stress

The stability analysis requires interface friction and internal shear-strength values, as measured with the ASTM D 6243 Direct Shear Test for GCLs, which was approved this year. Peak and residual interface-friction values must be obtained for both surfaces of the GCL under saturated conditions. Values for the peak and residual internal-shear strength also must be determined with a hydrated GCL and applied under normal loads that reflect actual field conditions.

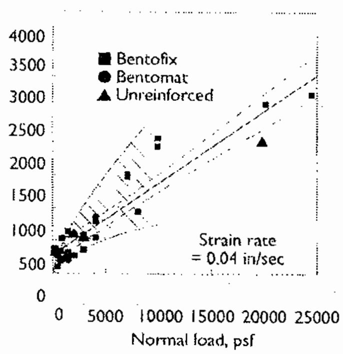

Figure 4. GCL residual shear strength vs. normal load.

It is common practice to hydrate the GCL for 24 hours in a water bath with only nominal normal loads, then transfer it to the shear box for testing. Thus, three series of direct-shear tests must be performed to evaluate the shear-strength parameters for a given application. The direct-shear test is expensive, slow and tedious to perform. For short-term conditions and non-seismic designs, the factors of safety against slope-stability failure can be evaluated using the peak interface friction and internal shear-strength values. However, for facilities in highly seismic regions and those where large strains may be anticipated, the stability evaluations should be based on residual shear-strength values for the critical shear plane.

The critical shear plane is typically the interface (or internal plane) that has the lowest peak-shear strength—a factor that can be identified only through laboratory testing. Also, the critical shear plane may be located on different surfaces at different points along the landfill cross-section, depending on the normal load. For example, liner systems on the bottom of a deep landfill experience large normal loads, and the surface-friction values developed may be sufficient to force the critical shear plane to occur within the GCL. Conversely, in shallow side slopes or in final-cover systems, the normal loads are so small that a failure at one of the interfaces is more probable than an internal failure of the GCL.

CQA for GCL stability

The slope stability of a GCL is affected by its internal-shear strength and the interface friction of its two surfaces. All of these provide the field inspector with difficult properties to evaluate quickly. Unfortunately,

the tests required for ASTM D 5321 Direct Shear are costly and time-consuming. The following recommendations are the authors' current approaches to evaluating these properties in the field.

Interface friction

Reinforced GCLs must be evaluated closely to ensure that the required interface friction is achieved and that bentonite migration is not a problem at large strains. (Typically, bentonite migration is only an issue with woven geotextiles.) It is important that site-specific interface-friction tests be performed before design and during field acceptance. No change to the geotextile or geomembrane should be allowed without retesting the proposed alternative materials.

Internal-shear strength

The internal-shear strength of a hydrated GCL depends on the strength of the geotextiles and cross-fibers. To ensure both components are adequate, the tensile strength and its peel strength between the geotextiles must be tested. Richardson (1996) and Heerten et al. (1995) previously have shown that the peel strength between geotextiles correlates to the internal-shear strength of the hydrated GCL.

Given the simplicity of the peel test, designers may wish to specify an increased peel-testing frequency to ensure that the GCL material is uniform. They also may wish to require one or two direct-shear tests per installation. This allows a roll-by-roll confirmation of the degree of reinforcement without the delays and costs associated with direct-shear testing.

The authors recommend a minimum peel strength of 25 lb. Note that the peel strength is given in pounds, and that the test currently is performed on a 4-in. wide sample. No formal test method has been developed for specifically determining the peel strength of GCLs. ASTM D 4632 has been used to compare GCL peel strengths and to develop relationships between peel and internal-shear strength. Sometimes the test is performed on samples of varying width, so it is important that the designer either specify the sample width (we recommend 4 in.) or that the resulting strength be specified in lb/in.

Recommendations

GCLs have proven their ability to replace and outperform compacted-clay liners. This article has reviewed the slope-stability design considerations and specifications for needlepunched reinforced GCLs. Currently, the authors have the following stability-related recommendations regarding the use of such GCLs in landfill-liner or final-cover systems:

- Only reinforced GCLs with a peak internal-shear strength greater than 500 psf under low normal loads (.250 psf) should be used. An unreinforced material should not be used unless the impact of free hydration of the GCL can be avoided or accounted for with certainty.

- A GCL's peel strength should equal or exceed 25 lb, as measured on a 4-in.-wide sample using ASTM D 4632 (preferably specified as minimum average roll value [MARV]) and should be verified in the field CQA program.

- Secondary fixation (e.g., heat burnishing) of the needlepunched geotextile fibers is recommended when the GCL internal stability depends on the long-term integrity of the needlepunching reinforcement.

- The designer should perform shear-strength testing of the various interfaces during the design phase to obtain an understanding of the peak and post-peak shear-strength behavior of the liner-system elements.

- The designer should understand where the critical slip plane is and consider using a post-peak strength as its value. A design-basis statement should be developed that clearly describes the rationale for selecting the critical shear plane for use in the stability analysis.

- Diligent verification of material properties during CQA is important in cases where slope stability is sensitive to small variations in material strengths and interface-friction values.

- Work by Gilbert et al. and others has shown that at large strain, the shear strength of a reinforced GCL will approach that of one without reinforcement (1996). The designer can rely on peak-strength values only if design-interface strains are small. Residual interface and internal-shear strengths are appropriate for the critical shear plane in conditions such as a seismic design requiring a displacement analysis.

The authors hope that this article will stimulate discussion of stability issues related to GCLs and lead to development of better guidelines for ensuring the internal stability and hydraulic integrity of these materials. Designers are encouraged to become active in ASTM D-35s Subcommittee on GCLs, chaired by Robert Mackey. This subcommittee currently is developing a draft standard practice for the application of GCLs to landfill-liner and final-cover systems. They encourage the design community to participate further in this process. GFR

Gregory N. Richardson, P.E., Ph. D., is principal of G.N. Richardson and Associates, Raleigh, N.C. **Rick Thiel**, P.E., has his own consulting and design-engineering firm—Thiel Engineering in Oregon House, Calif. **Robert Mackey**, P.E., is chief engineer at S2L Inc. in Maitland, Fla.

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