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GCL design guidance series
Part 2: GCL design for slope stability

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This is a continuation of a three-part series summarizing key aspects of the industry's first comprehensive GCL design guidance document (the *GundSeal GCL Design Manual*, Thiel et al. 2001) available from the second author. This part of the series focuses on slope stability. Relative to the role of GCLs when evaluating slope stability, the primary question focuses on what shear strength values should be used for analyses. This article explores that question and proposes answers that are dependent on the type of GCL selected and the boundary conditions.

General slope stability considerations

Geosynthetics, such as geomembranes and GCLs, often provide a preferential slip plane along which a slope failure may occur. For designs utilizing GCLs, the bentonite layer and interfaces adjacent to the GCL are obvious locations to evaluate as potential critical slip planes. Although the basic geotechnical principles used to evaluate the slope stability of bottom lining systems, cover systems, and surface impoundments are the same, it is crucial to consider applications separately due to the magnitudes of the forces and varying sensitivities to pore pressures.

Bottom liner systems are typically under a wide range of relatively high normal loads compared to veneer cover systems, which are typically under a narrow range of low normal loads. Surface impoundments incorporate additional hydraulic and buoyant forces created from elevated liquid levels. The project-specific range and distribution of normal loads has a significant effect on the shear-strength parameters to be used for many materials. This article focuses primarily on considerations related to GCLs utilized in bottom liner systems.

GCL shear strength measurement

For geosynthetic lining systems, the internal and interface (i.e., friction resistance) shear strength is normally determined using the direct shear test in accordance with ASTM D 5321. For GCL internal and interface shear strength evaluation, direct shear testing is conducted in accordance with ASTM D 6243. Several factors can influence the shear strength of all GCL products. The most important factors include (1) rate of displacement, (2) bentonite moisture content, (3) normal stress, (4) location of the shear plane, (5) peak vs. post-peak vs. residual shear strength, and (6) hydration liquid. These six GCL-related testing parameters and project-specific considerations are discussed by Thiel et al. (2001).

Since the shear strength properties of bentonite have been the subject of investigation for many decades and are well understood, it is possible to assume the shear strength properties of bentonite with a great degree of confidence. This is, however, not usually the case with most natural soils and geosynthetics. Recommended shear strength equations for dry and hydrated bentonite are presented by Thiel et al. (2001). When GCLs are placed directly on an earthen subgrade, without a geomembrane between the subgrade and the bentonite, slope stability analyses should always be performed assuming the fully hydrated shear strength properties of the bentonite. Depending on the construction and subgrade conditions, the designer may wish to select either the peak hydrated shear strength, the residual hydrated shear strength, or something in between. The dry shear strength should only be used for the case of encapsulated bentonite as discussed later.

Appropriate factor of safety

A commonly accepted value for the factor of safety in geotechnical engineering slope stability analyses is $FS \geq 1.5$. Many engineers accept this value while remaining unclear or uninformed of its basis. The origin of this value was the empirical result of analyzing the relative success and failure of dams constructed over the past century. Experience proved that when an analysis was performed correctly, assuming reasonable and prudent material properties, an earthen structure with a factor of safety of 1.5 can be expected to remain stable even when some of its structural geometry and material properties varied from those assumed in the analysis. Of course, this presumes the analysis is performed correctly.

The lead author defines a “design condition” as the anticipated actual long-term conditions that an interface will experience, and requires a factor of safety ≥ 1.5. The decision as to the appropriate long-term shear strengths the designer selects is project-specific (there are many variations), and a complete discussion of this topic is beyond the scope of this paper. Next, the author follows the guidance of Gilbert and Byrne (1996) and verifies that the stability under the worst-case shear strength conditions (e.g., hydrated residual shear strength) results in $FS > 1.0$. This latter test is often the more significant.

A good example of the above approach and selection of GCL shear-strength criteria for a bottom-liner design involves the encapsulation of unreinforced bentonite between two geomembranes **(Figure 1)**. The design scenario presumes that most of the bentonite will remain dry for at least several decades to centuries, and the basic slope stability analysis is performed on this basis. A second analysis is performed, however, to verify that the stability factor of safety is greater than unity when the installed bentonite area is under fully hydrated residual shear-strength conditions. This design methodology for encapsulated bentonite is described by Thiel et al. (2001) and summarized in the next section.

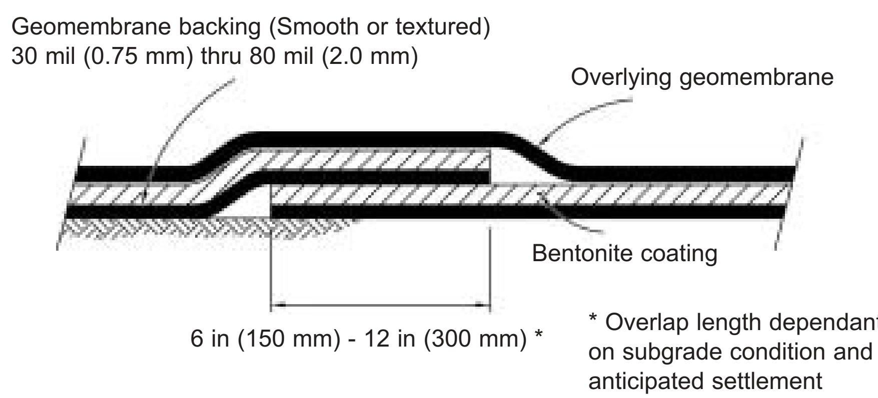

***Figure 1**: Encapsulated bentonite between geomembranes utilizing the GM-GCL with an overlying geomembrane.*

Reinforced GCLs in bottom liner systems

The critical surface is always the one that produces the lowest peak strength. If residual strengths are used in the analyses, they should reflect the surface that has the lowest peak shear strength, as this location will govern deformations. Since there are numerous potential mechanisms that could lead to localized interface deformations and progressive failure (e.g., non-uniform shear stress distribution; settlement; pore pressures; aging and creep), it is always prudent to evaluate the large-displacement shear strength of the interface that has the lowest peak strength.

A summary of this issue is presented by Gilbert (2001) and Thiel (2001) in the *Proceedings of the 15th GRI Conference*. Both these papers were discussed in a recent Designer's Forum column on slope stability (Richardson 2002). One member of the slope stability discussion panel at that conference suggested that engineers consider the interface with the *lowest residual* strength as the critical interface, regardless of the relative peak strengths. This was based on assumptions regarding long-term aging and durability of geosynthetic materials over hundreds of years.

When fabric-encased reinforced GCLs are used in a lining system, there is a presumption that the enhanced internal shear strength of the GCL will be preserved (typically due to needle punching). For this to remain true, the authors recommend that the peak internal shear strength of the hydrated GCL be greater than that of some other interface in the liner system, with the weakest interface preferably located above the GCL and as high up in the lining system as possible.

Thus, as long as the GCL fiber reinforcement remains intact, the stability of the lining system is usually governed by the interface with the lowest peak shear strength. In this case, the stability analysis should be performed using standard geo-technical engineering practices that evaluate factors such as peak vs. residual shear strength, potential pore-pressures, and potential seismic effects. It is important that the designer and CQA team verify that the materials provided meet the specified requirements as described in *GFR*'s August 2001 Designer's Forum by Richardson and Thiel (2001). In this case, the presence of a hydrated GCL with intact fiber reinforcement may not have a significant influence on slope stability.

There are at least four scenarios, however, where the presence of bentonite in a fiber-reinforced GCL may influence slope stability.

Intensity of fiber reinforcement. The internal shear strength of the reinforced GCL may be lower than the adjacent interfaces because the intensity of needle punching between the upper and lower textiles is relatively low.

Internal strength at elevated normal loads. The internal shear strength of even an aggressively needle-punched reinforced GCL may become lower than adjacent interfaces as the normal loads become higher (e.g., > 750 kPa).

Bentonite migration. Bentonite will extrude through woven geotextiles when saturated and create a slip plane against the adjacent layer. When placed against a geomembrane, the interface shear strength is often representative of pure bentonite and independent of the internal shear strength of the GCL. This occurred at the U.S. EPA Cincinnati GCL cover demonstration project (Koerner et al. 1996). Using upper and lower nonwoven geotextiles, or at least requiring the nonwoven side of the GCL to be placed against the geomembrane, can address the issue of bentonite migration.

Long-term aging and creep of reinforcement fibers. When a liner system provides shear forces to support a slope, the reinforcing fibers in the GCL are placed in a permanent state of tension. There is currently no industry basis or validation for depending on the tensile strength of these fibers to be maintained over several hundred years. Although this issue has been addressed for geosynthetic reinforcement in retaining walls and slopes, it has not been seriously broached regarding reinforced fabric-encased GCLs. Studies presented at the recent GCL specialty conference in Nuremburg, Germany suggest that the fibers may lose their strength within a century under buried conditions, and even quicker when exposed to atmospheric oxygen levels (Thomas 2002). The Geosynthetic Research Institute has recently begun a program to evaluate this issue (Hsuan 2000). At a minimum, it might be prudent to establish maximum allowable working stress levels and geotextile polymer formulation requirements (Koerner et al. 2000; Hsuan and Koerner 2002).

Most of the issues described above can be partially or fully addressed by the designer commissioning adequate direct shear tests during the design phase to understand what level of GCL peel strength is needed to obtain the required internal shear strengths. This should be followed with GCL peel strength conformance testing of the manufactured product as part of the project CQA program. Nonetheless, there is still a compelling case that designers should consider factoring in a design scenario where the unreinforced strength of the bentonite governs slope stability, and requiring a factor of safety at least greater than unity.

The remainder of this paper discusses the situation where the unreinforced bentonite component is the critical material in the lining system. This can occur either because an unreinforced GCL is specified (as with the GM-GCL product described in Part 1 of this series in the June/July 2002 issue of *GFR*), or for the reasons described above for the fabric-encased reinforced GCL.

Slope stability of encapsulated bentonite composite liner

In the encapsulated bentonite design (**Figure 1**), the manufactured GCL bentonite layer is installed at a typical moisture content of 25%, and is sandwiched between two geomembranes. The design intent is to preserve the “dry” shear strength of the bentonite. Over time, however, encapsulated bentonite can hydrate from a relatively dry state to one that is more saturated due to defects in geomembranes and overlapped GCL seams. From the point of view of shear strength, relatively dry means that the bentonite is drier than 35% moisture. Moisture contents above 40–50% will result in reduced bentonite shear strength (Daniel et al. 1993).

By estimating the fraction of the installed bentonite area that may become hydrated, the global shear strength of the bentonite layer can be prorated (i.e., % of dry area vs. % of hydrated area). With a given relative hydrated vs. dry area of bentonite in an encapsulated bentonite installation, a design methodology can be applied to prorate the shear strength over the design life of a project. The approach of prorating shear strength for encapsulated GCLs has been successfully applied in several landfill designs since 1994 for projects in the western United States. A case history outlining the prorated shear strength design approach is presented by Thiel and Erickson (2001).

After the installation of an encapsulated bentonite liner system, there are two potential hydration mechanisms that could result in a localized increase in bentonite moisture: (1) moisture entering through defects in the upper and/or lower geomembranes, and (2) subgrade moisture seeping through overlapped seams in the case of the GM-GCL material described in Part 1 of this *GFR* GCL Design series (June/July 2002 issue of *GFR*). A brief discussion of these two hydration mechanisms is described in the following paragraphs. Note that Thiel et al. (2001) discuss potential hydration from water diffusion through the geomembranes and conclude that this hydration mechanism is insignificant.

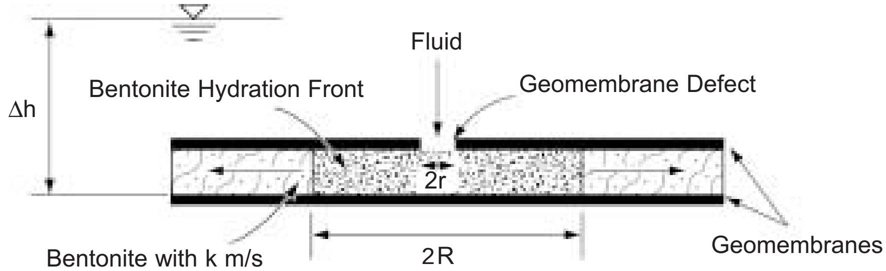

***Figure 2**: Drawing illustrating factors for calculating radial hydration of encapsulated bentonite through a geomembrane defect.*

Bentonite hydration from above and below through geomembrane defects

Equation (1) for radial hydration of encapsulated bentonite, caused by liquid entering a circular defect in the overlying geomembrane (**Figure 2**), was developed by Dr. J.P. Giroud (Thiel et al. 2001).

Equation (1)

$$\hat{t} = \frac{n R^2}{4k\,\Delta h}\left[2\ln(R/r) - 1 + \left(\frac{r}{R}\right)^2\right]$$

where:

h = total head driving moisture (suction plus liquid head) (m)
k = bentonite hydraulic conductivity (m/s)
n = available bentonite porosity (%)
r = radius of geomembrane defect (m)
R = radius of the wetting front (m)
t = time (sec)

To use **Equation (1)** requires the designer to select values of n, k and h under guidance provided by Thiel et al. (2001). Using **Equation (1)** with typical design assumptions, **Figure 3** shows the projected wetting rate around a single hole in a geomembrane for the encapsulated design.

Figure 3 shows how the radius of hydrated bentonite would be less than 600 mm over a period of 250 years in a typical bottom liner under 300 mm of liquid head. Relative to one hectare, the hydrated area beneath a single 10-mm-diameter geomembrane defect is calculated to be less than 0.008% of the total area. Thus, assuming there

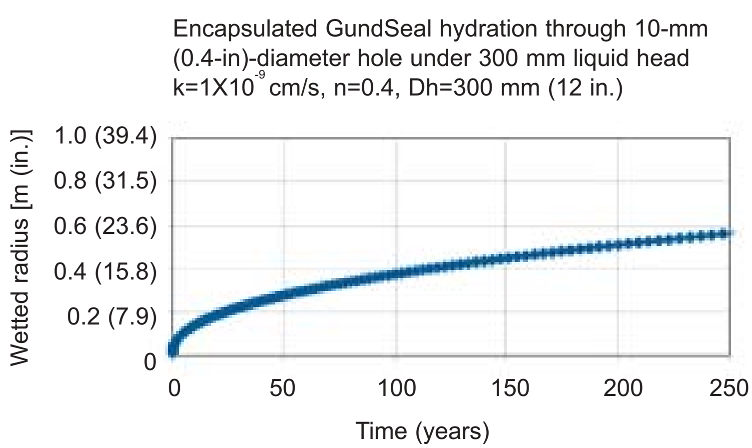

***Figure 3**: Predicted wetting of the encapsulated bentonite through a single 10-mm-diameter defect in a geomembrane.*

are 10 randomly located geomembrane defects per hectare in both the bottom and top geomembranes (a very conservative assumption), the total number of defects per hectare would be 20, the percent of the total area that becomes hydrated would be 20 x 0.008% = 0.16%. This degree of hydration beneath occasional imperfections in the geomembrane is essentially negligible assuming good contact between the geomembranes and the bentonite. Greater lateral spreading and bentonite hydration could, however, be produced by wrinkles in the overlying geomembrane. This analysis assumes a continuous head of water; if conditions were comparatively dry, the hydration would be less. Therefore, it is conservative to assume that 5% of the total installed bentonite area might become hydrated due to geomembrane defects over a 250-year design life.

Hydration from below through GM-GCL overlapped seams

This consideration is only applicable to GM-GCL products whose seams are overlapped and not welded (**Figure 1**). This situation was described in detail in Part 1 of this GCL Design Series. This scenario would not apply to encapsulated GCLs where the geomembrane seams on both sides of the GCL were completely welded. In the encapsulated mode, where a GM-GCL material is deployed with the geomembrane face-down with overlapped seams against a soil subgrade (**Figure 1**), moisture will be absorbed into the exposed bentonite edge of the overlap seam due to the difference in matric suction between the bentonite and the subgrade soils. The extent and rate of wetting along the exposed bentonite edge of the overlap depend upon the water content of the soil in contact with the bentonite. The method for calculating the hydration rates along an exposed overlapped GM-GCL seam edge was developed by Dr. J.P. Giroud and described by Thiel et al. (2001).

For wet subgrade conditions, it is estimated that after 250 years the bentonite hydration distance may approach 1 m from the edge of the overlapped panel. The calculation for percent of total installed bentonite area that may hydrate is plotted in **Figure 4**. The results indicate that after 250 years, approximately 33% of the area would become hydrated using a 150-mm overlap, and approximately 25% of the area would be hydrated using a 300-mms overlap.

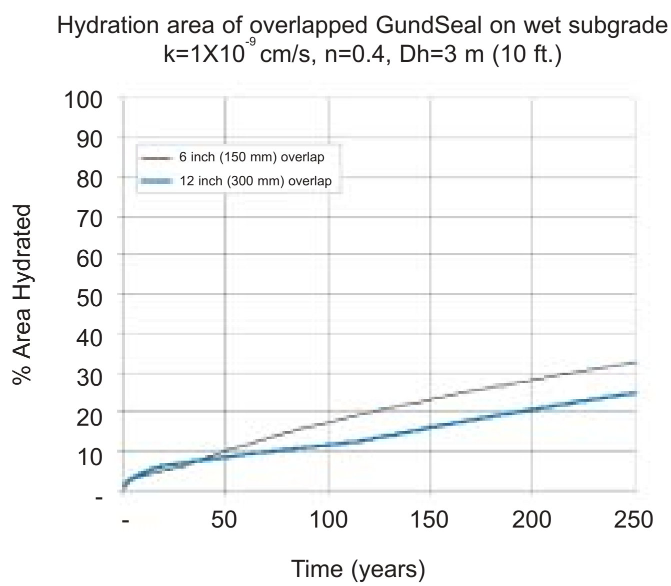

***Figure 4**: Predicted percent area hydrated of the encapsulated bentonite coating of the GM-GCL through overlaps deployed on a wet subgrade.*

Total hydration of an encapsulated bentonite lining system

By combining the bentonite hydration due to geomembrane defects (above all GCLs) and hydration at overlapped GCL seams (GM-GCL product only), total hydration of an encapsulated GCL liner system can be estimated for a given set of design criteria over the life of the project. Examples are provided by Thiel et al. (2001) showing how 5–34% of the bentonite's area might become hydrated over a period of 250 years, depending on the moisture content of the subgrade on which the liner was placed. Therefore, a conservative estimate of the total long-term (250 years) hydrated bentonite area resulting from geomembrane defects and GM-GCL overlap seams would range from 5+5 = 10%, to 5+34 = 39%.

Slope stability analysis for encapsulated GCL

The shear strength along a slip plane within the bentonite of the encapsulated GCL is a proration of both the hydrated and dry shear-strength properties of bentonite. Given the random location of geomembrane defects, and mostly even spacing of overlapped GM-GCL seams, one can assume the hydration pattern in the bentonite area to be relatively uniformly distributed over a project area. Therefore, a weighted average for the global shear strength can be defined based on the hydrated and dry shear strengths of bentonite, and the corresponding assumed fractions of hydrated and dry areas.

The prorated strength [τ(design)] is calculated as:

Equation (2)

$$\tau(design) = \tau(dry) - \frac{Area(hydrated)}{Area(total)}\left(\tau(dry) - \tau(hydrated)\right)$$

For example, if the hydrated fraction [$Area(hydrated)/Area(total)$] of the installed area of encapsulated GM-GCL over the design life of a project is assumed to be 10%, **Figure 5** illustrates the prorated design shear-strength envelope for a high normal-load residual strength. These shear-strength data were used for a landfill bottom-liner system designed and constructed in 1994 with a subsequent expansion in 1997 (Thiel and Erickson 2001).

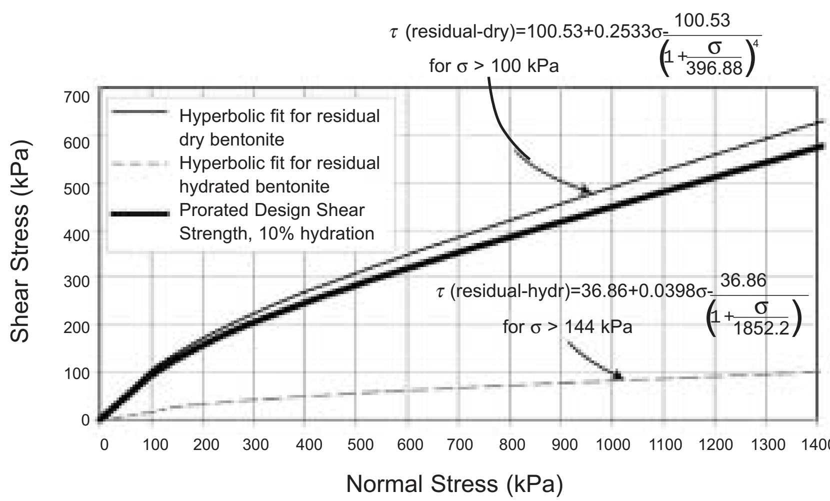

Figure 5: *Prorated shear-strength curve for encapsulated, partially hydrated bentonite.*

The factor of safety for a stability analysis using this approach is typically on the order of 1.5 or greater. Although this may satisfy the basic design requirements for slope stability, the authors recommend one additional requirement, which is that the factor of safety be greater than 1.0 assuming the bentonite is fully hydrated and has only residual shear strength. This latter condition is often the more critical.

Summary

For designs utilizing GCLs, the bentonite layer and interfaces adjacent to the GCL are obvious locations to evaluate for the critical slip plane. When evaluating a design for stability, testing and utilizing the appropriate GCL shear strength parameters in conjunction with project specific design criteria is crucial to effective long-term performance. To this end, the authors provide the following conclusions and recommendations regarding bottom-liner system designs:

• Designers and CQA firms should conduct material-specific testing of GCL internal and interface shear strengths to verify that the materials specified and/or supplied for a project are realistic and meet the design requirements. Whoever commissions the testing should possess a skilled familiarity with the design objectives as well as direct shear testing techniques.

• Designers should attempt to position the critical slip plane above the primary geomembrane to the extent feasible for a given project.

• Designers should consider evaluating all facilities for stability using the residual shear strength along the geosynthetic interface that has the lowest peak strength. This would be an advisable risk-management practice for designers, and verify the *FS* under these conditions is at least greater than unity.

• For evaluating fabric-encased reinforced GCLs, there are several reasons that designers may wish to verify that the long-term factor of safety is greater than unity using the shear strength of unreinforced bentonite.

• The shear-strength properties of sodium bentonite are well known and understood and can be taken from the literature.

• Encapsulating dry bentonite between two geomembranes is a valid technique for preserving a higher shear strength in the bentonite for at least some amount of time (ranging from decades to centuries depending on the design assumptions, site conditions, and level of care during construction). Regardless, the authors recommend that the slope stability be evaluated and verified to be greater than unity assuming the hydrated residual shear strength of unreinforced bentonite.

Part 3 of this series (September 2002 issue of *GFR*) focuses on installation, durability and construction quality assurance of GCL lining systems.

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