SLOPE DESIGN USING GEOSYNTHETIC CLAY LINERS

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ABSTRACT: This paper provides guidance to design stable slopes using Geosynthetic Clay Liners (GCLs) in liners or covers for the varying combinations of conditions that may develop due to the shape of the containment facility. The determination of both short-term and long-term strength for the potential slip interfaces is reviewed. The design process for three typical applications including covers and liner systems for bowl type containment and valley type containment are shown using representative cross sections to illustrate the design concepts and considerations. Static and dynamic (seismic) conditions are reviewed. Appropriate numerical tools are discussed (i.e., limit equilibrium and numerical methods). The fundamental design philosophy of avoiding conditions that will mobilize the peak strength of the GCL are reviewed. The design and construction conditions that jeopardize this essential requirement are described.

1 INTRODUCTION

Geosynthetic clay liners (GCLs) are manufactured hydraulic barriers consisting of clay bonded to a layer or layers of geosynthetic materials (e.g., bentonite clay sandwiched between two geotextiles). The uses of GCLs on slopes for liner and cover systems in landfills, heap-leach mining, and other applications create challenges to designers because of the complexity of GCL strength behavior. It is often debated whether to design using the peak or the residual strength of the GCL. The answer to this debate must consider the type of GCL, the context of the overall system behavior, and the specific conditions under which the GCL will be used. Design must consider the internal strength of the composite GCL product, the interfaces between its outer surfaces and adjacent materials, the interfaces of other adjacent liner components considering both short-term and long-term conditions, and the internal strengths of other liner components. The application, whether a cover system, a liner system in a bowl type containment, or a liner system in a valley fill, will also influence the selection of design strength values. Each of these components is reviewed in the following sections in relation to design of stable slopes with GCLs.

2 TYPES OF GCLS

GCLs are either unreinforced or reinforced. Unreinforced GCLs are problematic due to the potential for hydration and corresponding loss in shear strength of the clay. Bentonite has a very low shear strength, which is characterized by a friction angle of 8 degrees at a normal stress of 70 kPa and decreasing to 4 degrees at 500 kPa (Olson, 1974). In order to develop higher shear strengths necessary for safely constructing steeper slopes, GCLs are often reinforced, typically by needle punching synthetic fibers through the composite, to bind the clay between the outer geotextiles and provide fiber reinforcement within the clay. The strength behavior of these needle-punched GCLs is complex. Designs using such GCLs must consider the internal strength of the composite product and the interface strength between its outer surfaces and adjacent materials.

3 SHEAR STRENGTH EVALUATION OF NEEDLE-PUNCHED GCLS

Figure 1 shows some typical test results obtained from laboratory shear box tests on materials used in liner systems. One test shows the internal strength of a needle-punched GCL where free swell of the GCL under low normal stress has been prevented and failure was forced to occur within the bentonite. Results are also shown for 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 1 mm/min. Hydration and consolidation of the materials were controlled to prevent the bentonite from squeezing out onto the interfaces.

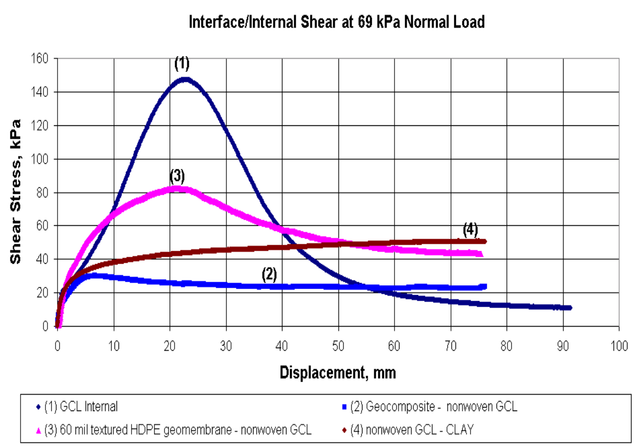

Figure 1: Typical Results for Internal and Interface Shear Tests (after Marr and Christopher, 2003)

The internal peak strength of the needle-punched GCL of about 150 kPa is the highest of all the potential failure surfaces included in Figure 1. However, 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 needle punching fibers that act like reinforcement and hold the material together provide the high internal strength. After these fibers become stretched to the point that they pull out or break, their contribution to the 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. The low shear strength of the bentonite (i.e., on the order of 8 degrees at low confining stress as previously noted) represents the lowest internal shear strength of the GCL and thus the **residual internal strength**.

For the test shown in Figure 1, the internal strength of the GCL at 90 mm of displacement is 9 degrees, but is still decreasing. The flat slope of the stress-displacement curve indicates that the residual strength of the GCL is almost reached. The close agreement between the strength of the GCL at 90 mm of displacement and the 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 1 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 geocomposite is about 1/5 the peak strength of the interface. If we sandwich these materials together to form a composite liner system, or a cover system, and subject the system 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 1, 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 exceeds the peak shear strength of 30 kPa (equivalent to a friction angle of 23 degrees 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 degrees in this case). Since the GCL has an internal peak strength almost 5 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 1 indicate a design approach to use that will avoid shearing the GCL to its residual strength – i.e., *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 1 and a normal stress of 69 kPa, shear failure will occur at the interface between the GCL and the geocomposite when the mobilized shear stress reaches 30 kPa.

Design using the lowest peak strength assumes that the peak strength of the interfaces and materials do not change with time. The data in Figure 1 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. 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).

Creep and aging of polymeric materials placed in tension are handled in reinforced soil applications by applying reduction factors to the peak strength of the materials. This approach has also been suggested by the authors in a previous study on design strengths of needle- punched GCLs (Marr and Christopher, 2003). In the absence of long-term direct shear tests to determine the creep limit of the GCL polymers (i.e., the stress level above which the filaments will creep to failure within the design life of the project), a creep reduction factor of 3 has been recommended by the authors based on creep reduction factors normally used for PP fibers in tension (Koerner, 1998). This value is considered somewhat conservative due to anticipated composite soil-fiber reinforcement interaction that is not present in conventional creep tests used to obtain the reduction factor of 3.

Oxidation is the primary aging mechanism for polypropylene fibers typically used in needle-punched GCLs. A significant decrease in strength of the polymeric materials in GCLs due to oxidation aging is unlikely because the oxygen level in saturated bentonite is lower than the 8 % level usually estimated for soils in reinforced soil applications. Recent studies by Thomas (2003) confirmed longevity of polypropylene at low oxygen content. In aging tests performed on PP fibers, taken from a GCL, he found a design life in buried applications with 8% air to be more than 300 years. The actual performance life may be even longer. 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. Straw in adobe and wood beneath water last for thousands of years in a low oxygen environment. In the absence of product specific aging data and considering the buried, low oxygen condition of GCLs, an aging factor of 1.1 to 2.0 as recommended by FHWA (2001) would appear to be a conservative reduction for a 100-year to 300-year performance period, respectively.

In Figure 1 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 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, as was suggested by Marr and Christopher (2003). 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 (140/6 + 10) 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 highly 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.

4 APPLICATION

Figure 2 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 0.3g will increase this average shear stress to 32 kPa for one or more instants in time, based on results from a pseudo-static stability analysis.

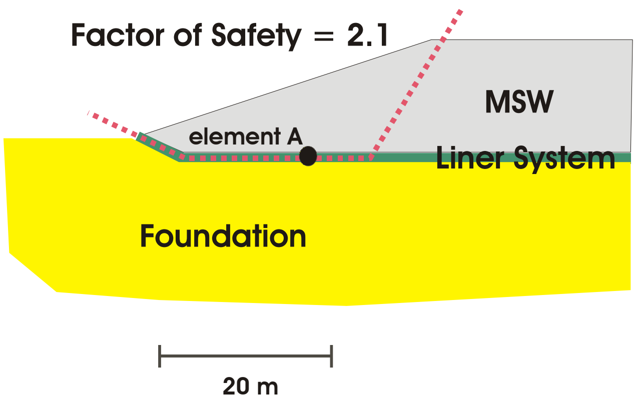

Figure 2: Typical Landfill Liner Analysis (after Marr and Christopher, 2003)

The average static shear stress in the liner system is 12.9 kPa. Figure 1 gives the results of shear tests on all components of this liner system at a normal stress of 69 kPa. All components 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 1, this slippage as well as 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 after the earthquake stops.

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, in valley fills and above ground sideslope fills, large settlements such as in waste materials that result in downdrag on the liner, construction induced deformations, migration of bentonite from the GCL to the interface, and sudden increases in pore pressure) as discussed by Thiel and von Maubeuge (2002) and Gilbert (2001).

A special case may also develop where the weakest interface is below the GCL and the liner components are anchored. This condition may induce tensile forces in the GCL and cause it to rupture.

5 DESIGN TOOLS

Slope stability is determined using methods of analysis that compare the shear stress necessary to maintain force equilibrium with the shear strength of the materials. Factor of safety is defined as the ratio of available shear strength to the mobilized shear stress required to maintain equilibrium. Typical conditions and standard practice usually require a minimum factor of safety of at least 1.5. This means that the available shear strength must be at least 50% more than the mobilized shear stress necessary to maintain force equilibrium of the slope. Mobilized shear stress is what is necessary to resist the force of gravity that tries to pull all slopes to a flat position.

Most slope designs are done with the method of limiting equilibrium. The equations of equilibrium are used to compute the forces acting within the slope. For shallow failures, such as those applicable to cover systems, the so-called infinite slope condition is analyzed. For uniform slope conditions, the minimum factor of safety for the cover system can be calculated (e.g., see Koerner, 1998).

Liner systems involve deeper failure surfaces with more complex geometries and multiple materials. Stability analyses for liners are typically done by dividing the slope into vertical slices and determining force equilibrium for each slice. Simplifying assumptions must be made for the analysis to be made determinate. All materials are treated as rigid-plastic materials and factor of safety is assumed to be constant for all slices. Multiple potential failure surfaces must be analyzed to find the one with the lowest factor of safety. These analyses are done with computer programs using methods like the Ordinary, Simplified Bishop, Simplified Janbu, Morgenstern-Price, and Spencer methods of slices (Johnson, 1974 and Siegal, 1975). Computers complete thousands of analyses per minute but the results are only as good as the input data, particularly the strength of the materials. In addition to the factors described in this paper, factor of safety is particularly sensitive to the magnitudes of pore water and pore air pressures acting on the failure surface. Todays practice typically uses the Simplified Bishop method for failure surfaces of a circular shape and Simplified Janbu for wedge-shaped failures. Both methods make simplifying assumptions that underestimate the actual factor of safety for slopes involving geosynthetics materials. The methods of Spencer and Morgenstern-Price give higher and more accurate values of factor of safety.

Some finite element methods, such as PLAXIS, have been developed to compute factor of safety by reducing the available shear strength of all materials by the same factor until the calculated plastic strains become large. With PLAXIS, we have obtained factors of safety for slopes similar to those obtained with limiting equilibrium methods. The main advantages of using a finite element method over limiting equilibrium are that stresses and displacements can be obtained for conditions other than failure and the failure surface is not artificially contrained to a circle or sliding blocks. The results may also be helpful to identify whether progressive failure is a likely mechanism and whether geosynthetics are subjected to excessive tensile forces.

6 CONCLUSIONS

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 1.0 mm/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 3 for creep times a factor of 1.1 for 100 years of aging or 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 degrees C, will affect creep results and should be considered in selecting appropriate reduction factors for temperature effects.)
- 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 minimum 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 deformation 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 needle-punched 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.

Design methods should use limiting equilibrium methods of analysis with careful attention paid to using the appropriate analysis method for the geometry, the appropriate strength parameters for all materials, and pore water/gas pressures if they exist. Special cases, such as those involving progressive failure or potential sliding beneath the GCL may warrant a more advanced analysis using a good finite element computer program.

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