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Testing and Acceptance Criteria for Geosynthetic Clay Liners

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EFFECT OF SWELL PRESSURE ON GCL COVER STABILITY

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ABSTRACT: This paper describes the importance of bentonite swell pressure on the stability of cover systems that incorporate a geosynthetic clay liner (GCL). The results of a one-dimensional swell test indicate that the field swell pressure of a needle-punched GCL ranges from 35 to 40 kPa. An effective normal stress at or near this swell pressure may be required to maximize the contact area between the GCL and geomembrane and increase the static and seismic stability of a GCL cover. Since an effective normal stress of 35 to 40 kPa is probably not practical and a soil cover is usually not immediately placed, it is recommended that free swell conditions be assumed for GCL shear testing and the slope be designed using the resulting shear strength parameters. Suggestions for modifying existing products to increase GCL cover stability are also presented.

KEYWORDS: geosynthetic clay liners, swell pressure, slope stability

For both hazardous and municipal solid waste containment facilities, the required strategy of the U.S. Environmental Protection Agency (EPA) for environmental protection is a composite liner and cover system. This composite system usually consists of a geomembrane placed in intimate contact with a compacted clay liner (CCL). Intimate contact is necessary so that any liquid passing through a hole in the geomembrane cannot spread laterally from the hole and approach the clay over a greater wetted area than the hole itself. In recent years, geosynthetic clay liners (GCLs) are increasingly being chosen to replace compacted clay liners.

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Some of the advantages of GCLs over CCLs are: lower and more predictable cost, prefabricated/manufactured quality, easier and faster construction, field hydraulic conductivity testing is not required, engineering properties are readily available, hydraulic conductivity is more resistant to cycles of wetting and drying, freeze/thaw cycles do not significantly increase hydraulic conductivity, the smaller thickness results in more air space, and repair is easier. Some of the disadvantages of GCLs over CCLs include smaller leachate attenuation capacity, shorter containment time, lower internal and interface shear strength, larger post-peak shear strength loss, and possibly higher long-term flux because of a reduction in bentonite thickness under the applied normal stress [1].

The first documented use of a GCL in a waste containment facility occurred in 1986 at a Waste Management of North America, Inc. site in Calumet City, Illinois [2]. The product used was manufactured by enclosing bentonite mixed with an adhesive between a woven and an open weave geotextile. This product is referred to as GCL A in this paper and was manufactured by Clem Environmental Corporation. Prior to this, bentonite blankets were primarily used for foundation waterproofing. The first of these waterproofing products was introduced in 1965 and consisted of bentonite sandwiched between corrugated cardboard panels. In 1991 the term geosynthetic clay liner was applied to bentonite blankets used in landfill liner or cover systems and other containment projects. At present, there are four main types of GCL products available. Two of these products consist of bentonite sandwiched between geotextiles that are needle-punched together. In these products the bentonite is sandwiched between a woven and nonwoven geotextile or two nonwoven geotextiles. Another product involves stitch bonding two woven geotextiles together to reinforce the bentonite. The last product consists of bentonite adhered to a high density polyethylene (HDPE) geomembrane.

The acceptance and use of GCLs in waste containment facilities has increased yearly since 1986. Market consumption of GCLs in 1995 was 54 million square feet and is estimated to increase by 8 percent to 58 million square feet in 1996 [3,4]. However, there are a number of questions regarding the design and acceptance of GCLs in waste containment facility liner and cover systems. This paper primarily addresses the design and acceptance of GCLs in landfill cover systems. In particular, the effect of bentonite swell pressure and bentonite extrusion into the geomembrane/GCL interface on the stability of GCL cover systems will be investigated.

EPA SLOPE STABILITY RESEARCH PROJECT

A GCL slope stability research project was initiated to investigate the stability of GCL cover systems. As part of this research project, four GCL manufacturers participated in the construction of fourteen landfill cover test pads. The GCL test pads were constructed in Cincinnati, Ohio at an operating waste containment facility. Nine test pads were constructed on a 2:1 slope while five test pads were constructed on a 3:1 slope. Figure 1 presents a plan view of the nine test pads constructed on the 2:1 slope. The test pads are 8 to 9 m wide and approximately 20 m long. The instrumentation allows monitoring of the geosynthetics deformation and the moisture content of the bentonite. Figure 2 presents a cross-section of Test Pads G and H. It can be seen that the GCL was placed on the natural subgrade and overlain by a 1.5 mm (60 mil) textured HDPE geomembrane (GM). The geomembrane was then overlain by a geosynthetic drainage composite (GT/GN/GT), 1 m of compacted soil, and a geosynthetic erosion mat (Figure 2).

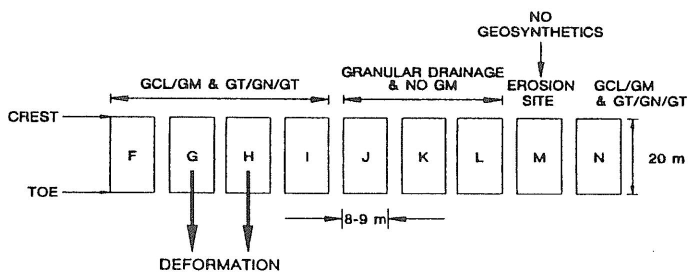

Figure 1. Plan View of GCL Test Pads on the 2:1 Slope

Approximately 20 days after installation of Test Pad H, the overlying textured geomembrane pulled out of the anchor trench and displaced 20 to 25 m to the bottom of the 2:1 slope. The GCL installed in Test Pad H consisted of two woven geotextiles stitch bonded together. The geotextiles are woven slit-film multifilament fabrics. This product is referred to as GCL B in this paper and was manufactured by Clem Environmental Corporation. In summary, deformation occurred at the interface between the woven geotextile and the overlying textured geomembrane.

Approximately 50 days after installation of Test Pad G, the overlying textured geomembrane also pulled out of the anchor trench and displaced approximately 20 meters to the bottom of the 2:1 slope. The GCL installed in Test Pad G consisted of a woven and nonwoven geotextile with needle punching providing internal support. This product is referred to as GCL C in this paper and was manufactured by Colloid Environmental Technologies Company. The deformation observed in Test Pads G and H and other field experiences, e.g., Cowland [5], clearly illustrate the importance of the GCL/geomembrane interface on the stability of landfill cover systems (Figure 2).

In Test Pads G and H, the woven geotextile of the GCL was placed in contact with the overlying textured geomembrane. A woven geotextile is used to promote intimate contact between the bentonite and geomembrane, which is required by the EPA. A woven geotextile allows bentonite to extrude through the textile during bentonite hydration and thus satisfy the requirement of intimate contact.

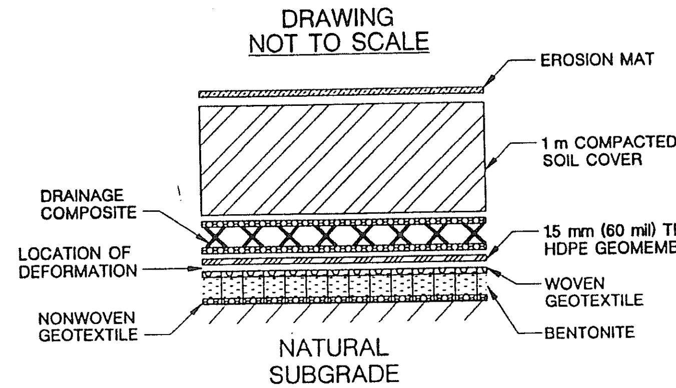

Figure 2. Cross-Section of Test Pad G

The extruded bentonite also may create a slope stability problem along the geomembrane/woven geotextile interface. The deformations observed in Test Pads G and H occurred between the woven geotextile and the textured geomembrane and are due to bentonite reducing the shear resistance of the geotextile/geomembrane interface. In summary, the requirement of intimate contact between the bentonite and geomembrane presents a dilemma for the static and seismic stability of GCL cover systems. The stability of a cover system appears to be related to the amount of bentonite that will extrude into the geomembrane/GCL interface. It is proposed herein that the amount of bentonite extrusion is controlled by the swell pressure of the bentonite or GCL. The swell pressure is defined as the normal stress required to maintain zero volume change or expansion.

FIELD SWELL PRESSURE OF GCLs

A number of researchers, e.g., [6,7], have conducted laboratory tests to estimate the swell pressure of GCLs. Leisher [6] used 0.15 m by 0.15 m square samples in dead weight and air-activated consolidometers to estimate the relationship between swell or vertical deformation and the applied normal stress. In each test the hydration fluid covered the GCL specimen and vertical deformation versus elapsed time data were recorded until equilibrium was achieved. These tests were conducted at different normal stresses to establish the relationship between vertical deformation and applied normal stress. The GCL was placed between a rigid sub-base and rigid loading platen. As a result, the full deformation or swell pressure of the GCL was measured. A typical series of swell tests for GCL A is shown in Figure 3. It can be seen that the GCL swell pressure is approximately 130 kPa. In other words, a normal stress of approximately 130 kPa is required to result in zero vertical strain. The data also suggests that more than a 150 percent increase in thickness is possible at a normal stress of approximately 7 kPa. The normal stress typically encountered in a cover system ranges from 7 to 15 kPa.

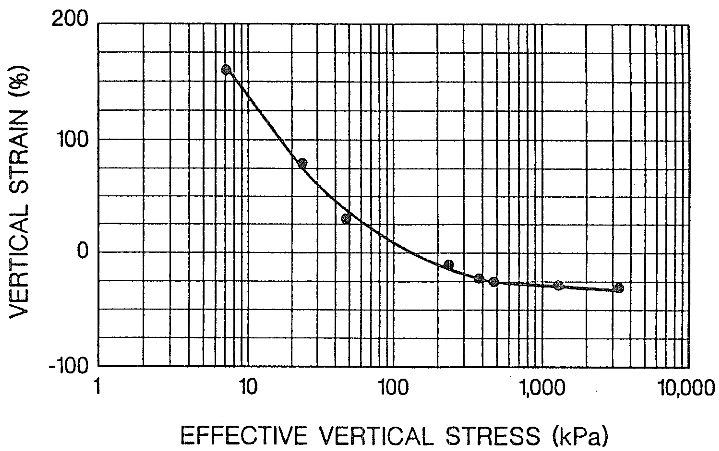

Figure 3. Swell Behavior of GCL A on a Rigid Base [7]

In a cover system, the GCL is usually placed on a lightly compacted soil layer that overlies the waste. This soil layer is usually lightly compacted because of the compressible waste underlying the compaction equipment. In addition, the overlying geomembrane is usually not in complete contact with the GCL. As a result, it was of interest to measure the swell pressure of GCLs using a flexible system to simulate field cover conditions. It was anticipated that a swell pressure less than 130 kPa would be measured because bentonite could extrude and/or deform into the nonwoven geotextile, woven geotextile/textured geomembrane interface, and granular sub-base instead of reacting against a rigid, non-porous sub-base.

To accomplish this objective, a swell pressure test was conducted in a 0.3 m by 0.3 m square consolidometer during this study. The nonwoven geotextile of the GCL was placed on a horizontal layer of sand with a unit weight of approximately 17 kN/m^3, which simulates a final interim cover layer. The consolidometer was installed in an automated INSTRON loading machine and the swell pressure was measured using a load cell. The load cell was installed between a rigid steel rod and the cross-arm of the loading machine.

The rigid steel rod was connected to the top of the loading platen and thus transmitted the swell pressure directly to the load cell. A plexiglass loading platen was placed in contact with a textured 1.5 mm (60 mil) HDPE geomembrane that was placed in contact with the upper surface (woven geotextile) of the dry, or as-received, GCL.

After placing the loading platen in contact with the geomembrane, the cross-arm of the loading machine was fixed. It should be noted that no normal stress was applied to the GCL by the loading machine prior to hydration. The cross-arm remained stationary during the swell test so that no vertical or upward movement could occur. The resulting swell pressure was measured using the load cell. The specimen was hydrated by allowing the GCL to attract water from a container that was placed along side of the consolidometer. Rubber tubing was used to connect the water container to the sand underlying the GCL. A hydraulic head was not applied to the water in the container, and thus hydration occurred due to the suction pressures induced by the bentonite.

In summary, the bentonite was free to move into the underlying nonwoven geotextile, woven geotextile/textured geomembrane interface, and cohesionless sub-base during hydration. The pressure exerted on the fixed cross-arm during swelling was measured by the load cell and assumed to be representative of the field swell pressure.

Results of Swell Test

Figure 4 presents the results of a swell test conducted during this study on GCL C, which was used in Test Pad G. It can be seen that a swell pressure of approximately 38 kPa was measured for this GCL after 9.5 days. It should be noted that the swell pressure was still increasing after 9.5 days, and thus a field swell pressure of 35 to 40 kPa is assumed throughout this paper for this GCL. A swell pressure of 35 to 40 kPa is significantly less than 130 kPa, which was measured using a rigid sub-base and loading platen as described by Leisher [6].

The fact that more than 9.5 days is required to fully hydrate a 0.3 m by 0.3 m GCL specimen has important ramifications for the laboratory shear testing of GCLs. Conventional direct shear testing usually allows 24 to 72 hours for hydration. The results in Figure 4 suggests that the specimen may not be fully hydrated with this soaking duration. However, it should be noted that the time required for complete hydration may differ depending on the hydration conditions, e.g., suction versus a positive hydraulic head.

Examination of the GCL specimen after 9.5 days of swell revealed several interesting phenomena. First, bentonite was observed to primarily extrude at the location of the needle-punching through the woven geotextile and the interface. The needle-punching separated the filaments of the woven geotextile, which provided an opening for the bentonite to extrude through. In the non needle-punched areas of the GCL, the woven geotextile remained intact and limited the amount of bentonite extrusion.

Second, the hydrated thickness of the GCL was approximately twice the original thickness of 6 to 7 mm. Thirdly, there appeared to be only localized loss or weakening of the needle-punching due to bentonite swelling. Lastly, bentonite extruded into, but not through, the underlying nonwoven geotextile.

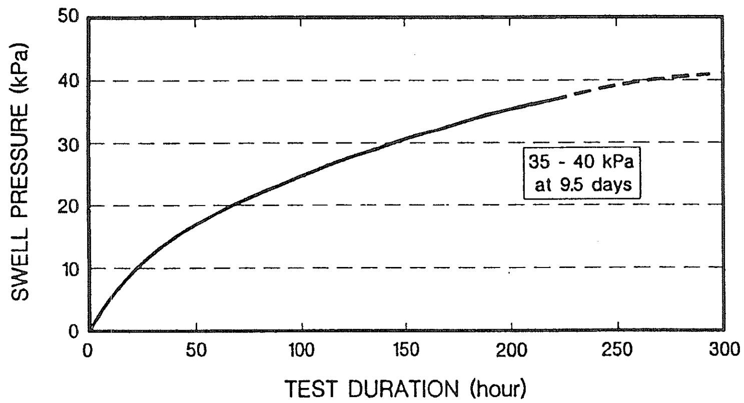

Figure 4. Swell Behavior of GCL C on a Compressible Base

In summary, the measured swell pressure for GCLs is related to the magnitude of precompression applied prior to hydration, stiffness of the test apparatus, and compliance of the simulated cover system. A swell pressure of 35 to 40 kPa appears to represent a reasonable estimate of the field swell pressure in a landfill cover that utilizes a GCL with a woven geotextile in contact with an overlying textured geomembrane. This swell pressure was measured using a deformable or flexible test configuration and no pre-compression or seating load. A field swell pressure of 35 to 40 kPa corresponds to the normal stress that is applied by 2.1 to 2.5 m of cover soil. Therefore, it can be assumed that 2.1 to 2.5 m of soil cover is needed to reduce the amount of bentonite that will extrude into the interface and increase stability. Since this soil cover thickness is usually not practical or economical, current GCL products could be modified to augment the static and seismic stability of cover systems.

Modification of Existing GCLs

One modification involves placing a nonwoven geotextile, instead of a woven geotextile, in contact with the geomembrane. This will increase the frictional resistance between the GCL and geomembrane, especially when a textured geomembrane is utilized. However, a nonwoven geotextile usually does not allow bentonite to extrude through the geotextile

and develop intimate contact with the geomembrane [1]. As a result, at least one manufacturer is inserting powered bentonite in the nonwoven geotextile that is placed adjacent to the geomembrane to promote intimate contact.

In summary, the requirement of intimate contact may not be satisfied with a nonwoven geotextile of the GCL in contact with the overlying geomembrane. In addition, if bentonite cannot extrude all of the way through the nonwoven geotextile, the swell pressure may induce larger tensile forces on the internal reinforcement. This may adversely affect the long-term internal shear resistance of the GCL.

Another possible modification of existing GCLs involves eliminating the geomembrane/GCL interface from the cover system. This can be accomplished by modifying existing GCLs to incorporate the geomembrane and GCL into a single product. One way of achieving this objective is to use a co-extruded HDPE geomembrane and geonet. To create a GCL with this co-extruded geomembrane and geonet, the geonet could be filled with bentonite. The bentonite can be placed with or without an adhesive. After bentonite placement, a nonwoven geotextile can be adhered to the top of the geonet. This results in a prefabricated composite liner (PCL) system that could be used in cover or liner systems.

Other variations of the PCL include the use of an internal configuration or structure that differs from a geonet. The internal structure simply serves to facilitate bonding of the geotextile and resists the overlying normal stress, which will be discussed subsequently. A fabric encased GCL also can be obtained by bonding two geotextiles to the internal structure instead of one geotextile. This GCL could be fabricated by bonding one geotextile to the internal structure, filling the structure with bentonite, and bonding the second geotextile.

The PCL configuration results in large interface strengths between the textured geomembrane/overlying material and the nonwoven geotextile/underlying soil interface. In addition, the geonet significantly reduces the potential for internal failure or shear through the bentonite and provides some tensile resistance to the cover system. The PCL also allows the bentonite to be in intimate contact with the geomembrane.

One of the largest benefits of the geonet or internal structure is that it protects the bentonite from the effects of handling and construction and the application of normal stress after hydration. The normal stress protection is more important in a composite liner system than a cover system. Stress concentrations in a liner system can cause the hydrated bentonite to migrate to a zone of lower stress. Stress concentrations are ubiquitous in a liner system, especially around a sump and the associated plumbing features, slope transitions and benches, and geomembrane wrinkles. Anderson and Allen [1] showed that the thickness of a GCL can be significantly reduced in the vicinity of a geomembrane wrinkle. A normal stress of 240 kPa was applied to a hydrated GCL in the presence of a geomembrane wrinkle. The one-dimensional compression test showed that the bentonite migrated toward the area or void under a geomembrane wrinkle. The thickness of the

GCL under the wrinkle was 20 to 25 mm while the thickness of the GCL farthest away from the wrinkle was less than 2.5 mm. The initial thickness of the GCL was 7.5 mm. These tests were conducted using the 0.3 m by 0.3 m square consolidometer described previously. A reduced bentonite thickness can adversely affect the calculation of hydraulic equivalence between a GCL and CCL. This is of particular importance in a sump area where leachate is designed to collect.

PCL Installation and Cost

The PCL could be installed using the scheme depicted in Figure 5. It can be seen that the internal structure is not extruded to the edge of the geomembrane sheet. This allows the geomembrane from each panel to be welded together to create a continuous geomembrane barrier. In addition, granular bentonite or a strip of the proposed fabric encased GCL (two geotextiles bonded to an internal structure) could be placed in the seam area to complete the clay barrier. Another construction or seaming technique could involve placing granular bentonite on the underlying geomembrane and simply overlapping the edge of the geomembranes. This overlapping technique is currently used for other GCL products. It should be noted again that the geomembrane and internal structure are co-extruded, and thus there is no interface or weakness between the bentonite and the geomembrane.

Other applications of the PCL involve placing the geomembrane of the PCL in contact with the subgrade material and placing a geomembrane above the PCL to encapsulate the bentonite. Another application could involve not bonding a nonwoven geotextile to the top of the geonet and placing the geonet in contact with the subgrade. Preliminary testing shows that the geonet can embed into the subgrade material, which results in a large interface strength. The thickness of the geonet not embedded in the subgrade reduces the potential for hydrated bentonite to migrate due to the overlying normal stress.

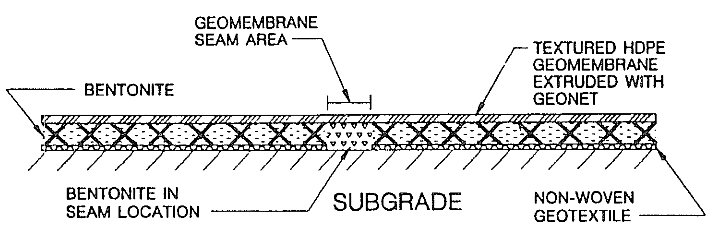

Figure 5. Possible Installation of Prefabricated Composite Liner

The main disadvantage of the PCL is cost. The manufactured cost of this GCL has not been established but it will be greater than existing GCLs. However, the PCL incorporates a geomembrane and saves a step in the installation process. As a result, the overall cost may be similar or less than an existing composite system consisting of a geomembrane and GCL. One manufacturer is currently working on fabricating prototypes of the PCL, which will aid in cost estimating.

GCL Cover Installation/Design Considerations

The laboratory data presented previously and the field performance of GCL Test Pads G and H were used to develop recommendations for the installation/design of GCLs. Clearly, the time between GCL placement and normal stress application should be minimized. If the GCL hydrates without the applied normal stress, more bentonite will likely extrude into the GCL/geomembrane interface and reduce the contact area between the GCL and overlying geomembrane.

For stability purposes, the contact area between the bentonite and geomembrane should be minimized. To minimize this contact area, a normal stress greater than or equal to the swell pressure should be applied. The swell tests conducted using a flexible system suggest that a normal stress of approximately 35 to 40 kPa needs to be applied, which corresponds to a soil thickness of 2.1 to 2.5 m. Clearly this thickness of soil cover is not practical. As a result, it is recommended that designers assume that the GCL will hydrate under free swell conditions and the slope be designed using the resulting shear strength parameters. This involves conducting laboratory shear tests that allow soaking with zero confinement until vertical expansion is completed or until the actual time between GCL placement and soil cover completion has elapsed. After vertical expansion or swell has been completed or the estimated time between GCL placement and soil cover completion has elapsed, the normal stress at which shearing will occur should be applied. Shearing can begin as soon as the GCL specimen consolidates to the applied normal stress. The resulting shear strength parameters should be used to evaluate cover stability.

Conducting GCL shear tests after completion of free swell, or the actual time that will elapse between GCL placement and soil cover construction, will provide shear strength parameters that are representative of field hydration conditions. The basis for this recommendation is that a typical GCL cover construction procedure involves placing one to two acres of GCL and geomembrane per day. As a result, it may require several days to a couple of weeks for completion of the soil cover due to geomembrane seaming, seam testing and repair, drainage layer placement, and cover soil compaction. In the interim the bentonite may hydrate due to (1) high suction pressures in the bentonite attracting moisture, (2) heat absorption by the overlying geomembrane causing subgrade moisture to migrate to the GCL, and/or (3) the GCL retaining saturated landfill gas.

REPORTING GCL FRICTION ANGLES

Since this symposium also addresses the laboratory testing of GCLs, the remainder of this paper provides recommendations for the reporting of GCL shear test results. Geosynthetic/geosynthetic and soil/geosynthetic interfaces usually exhibit stress dependent behavior. Previous testing of geosynthetic interfaces typically encountered in waste containment facilities [8,9] has shown that the stress dependent behavior results in nonlinear failure envelopes (Figure 6). The entire nonlinear failure envelope cannot be represented by a single value of friction angle. More importantly, this suggests that a friction angle applicable to a cover stability analysis is probably not applicable to a liner stability analysis.

Therefore, it has been recommended that the entire nonlinear failure envelope or a friction angle that corresponds to the average effective normal stress on the critical slip surface be used in a stability analysis [8,10]. For simplicity, most laboratories and designers report or utilize a single value of friction angle to represent the shear resistance. As a result, it is recommended that the reported friction angle utilizes a subscript to identify the range of normal stress over which the friction angle should be used in stability analyses and/or the range of normal stress at which the shear testing was conducted. The proposed notation for the peak friction angle is:

$$\phi_{1000\text{ - }6000\text{ psf}} \cong \text{x degrees}$$

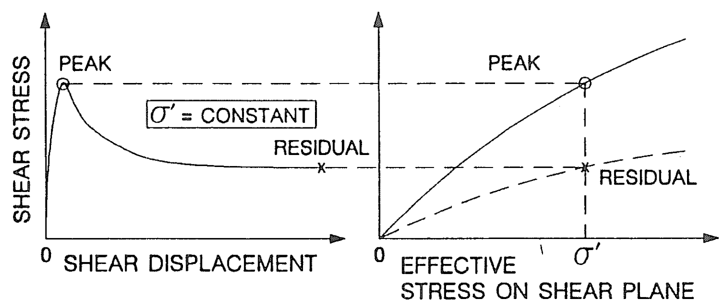

Figure 6. Shear Characteristics of Geosynthetic Interfaces

Post-Peak Friction Angles

In general, geosynthetic interfaces exhibit a peak shear strength at a shear displacement of 2 to 8 mm [8,9]. Continued displacement after the peak interface strength results in a decrease in the measured shear resistance (Figure 6). After considerable continuous shear displacement in one direction a constant minimum, or residual, shear strength is achieved. This usually results in a stress dependent or nonlinear residual failure envelope as shown in Figure 6. The magnitude of shear displacement required to reach a residual strength condition is dependent upon the interface. However, continuous shear displacement of 500 to 750 mm has been reported for textured geomembrane/nonwoven geotextile interfaces [9]. This magnitude of shear displacement may be larger than the displacement that can be achieved in one travel of a 0.3 m by 0.3 m direct shear box. If this is the case, the friction angle calculated at the end of a 0.3 m by 0.3 m direct shear test may overestimate the residual interface strength and should not be reported as a residual value.

It should be noted that reversal of a direct shear box does not result in continuous shear displacement in one direction along an interface, and thus a reversal direct shear test may also overestimate the residual interface strength. The residual strength condition is only achieved when the shear stress-displacement relationship becomes horizontal, i.e., there is no change in the measured shear stress. La Gatta [11] recommends plotting the shear stress-horizontal displacement relationship from shear tests using the logarithm of horizontal displacement to determine if a residual strength condition is achieved. This plotting technique accentuates the slope of the shear stress-displacement curve at large deformations, allowing the horizontal portion of the curve to be clearly defined. Therefore, to ensure that a residual strength condition is reached before a ring shear or direct shear test is terminated, it is recommended that the shear stress be plotted using the logarithm of horizontal displacement. Once the shear stress becomes essentially constant on a semilogarithmic plot, the test can be stopped.

Since 0.3 m by 0.3 m direct shear tests are usually terminated at a shear displacement ranging from 25 to 100 mm, it is recommended that the magnitude of shear displacement and the applicable normal stress be reported as a subscript to the post-peak friction angle. The displacement subscript, instead of a residual subscript, should be used because direct shear tests may be terminated before a residual strength condition is achieved. The proposed notation for the post-peak friction angle is presented below:

$$\phi_{50\text{ mm, }2000\text{ - }8000\text{ psf}} \cong \text{y degrees}$$

If a residual interface condition is achieved in the shear test, a subscript denoting a residual strength condition can be used. One such notation is shown below:

$$\phi_{r,\ 2000\text{ - }8000\text{ psf}} \cong \text{z degrees}$$

Stark and Poeppel [8] and Cowland [5] showed that a residual interface strength is usually mobilized along a sideslope and should be used in estimating the static stability of waste containment facilities. The use of seismic deformation analyses in waste containment facility design has also accentuated the need to estimate the residual interface strength or relate the measured shear strength to the level of deformation. Clearly, the laboratory shear resistance should reflect the level of seismically induced displacement that is anticipated. Therefore, adding a displacement notation to the reported friction angle will aid engineers in determining the appropriate shear strength for static and seismic stability analyses. Another reason to specify the displacement is that the shear stress-displacement relationships may not be included in the laboratory report or may not be incorporated into the stability report. Therefore, the magnitude of shear displacement may not be conveyed to the engineer, client, and/or regulatory agency. This lack of information is especially problematic when the data is incorporated into a database and subsequently published.

SUMMARY AND CONCLUSIONS

Recent field observations demonstrate the importance of the geomembrane/GCL interface strength on the stability of landfill covers. The greater the swell pressure or the smaller the applied normal stress, the more likely that bentonite will extrude into the geomembrane/GCL interface when a woven geotextile is used as the top layer of the GCL. Swell tests conducted during this study using a flexible system indicate that the field swell pressure of a needle-punched GCL probably ranges from 35 to 40 kPa. It is recommended that a normal stress or soil cover be applied as soon as possible after GCL installation to reduce the amount of bentonite that extrudes into the interface, and thus minimize the contact area between the bentonite and geomembrane.

To reduce the amount of bentonite that extrudes into the interface, the applied normal stress should be at or above the swell pressure. Since a normal stress of 35 to 40 kPa, i.e., soil cover thickness of 2.1 to 2.5 m, may not be practical and usually several days to a couple of weeks elapse between GCL placement and completion of the soil cover, it is recommended that designers assume that the GCL will hydrate under a free swell condition. This can be simulated by conducting laboratory shear tests that allow soaking with zero confinement until vertical expansion is completed or until the actual time between GCL placement and soil cover completion has elapsed. After swelling has ceased, the desired normal stress should be applied. Shearing could begin after the specimen consolidates to the applied normal stress. The resulting shear strength parameters should be used to evaluate slope stability.

Geosynthetic/geosynthetic and soil/geosynthetic interfaces usually exhibit a stress dependent or nonlinear failure envelope. Therefore, it is recommended that the reported peak friction angle utilize a subscript to identify the range of normal stress over which the friction angle is applicable and/or the range of normal stress at which the shear testing was conducted. It is also recommended that the magnitude of shear displacement and applicable normal stress be reported as a subscript to the post-peak friction angle. The displacement subscript, instead of a residual subscript, should be used because direct shear tests may be terminated before a residual strength condition is achieved.

ACKNOWLEDGMENTS

Sam Allen of TRI/Environmental in Austin, Texas supervised the 0.3 m by 0.3 m swell test on the needle-punched GCL. The writer also acknowledges Richard Thiel for his suggestions on the laboratory shear test procedure.

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A COMPARISON OF SAMPLE PREPARATION METHODOLOGY IN THE EVALUATION OF GEOSYNTHETIC CLAY LINER (GCL) HYDRAULIC CONDUCTIVITY

Reference: Siebken, J. R. and Lucas, S., "**A Comparison of Sample Preparation Methodology in the Evaluation of Geosynthetic Clay Liner (GCL) Hydraulic Conductivity,**" *Testing and Acceptance Criteria for Geosynthetic Clay Liners, ASTM STP 1308,* Larry W. Well, Ed., American Society for Testing and Materials, 1997.

Abstract: The method of preparing a single needle-punched GCL product for evaluation of hydraulic conductivity in a flexible wall permeameter was examined. The test protocol utilized for this evaluation was GRI Test Method GCL-2 Permeability of GCLs. The GCL product consisted of bentonite clay material supported by a woven and a non-woven geotextile on either side. The method preparation focused on the procedure for separating the test specimen from the larger sample and whether these methods produced difficulty in generating reliable test data. The methods examined included cutting with a razor knife, scissors, and a circular die with the perimeter of the test area under wet and dry conditions. In order to generate as much data as possible, tests were kept brief. Flow was monitored only long enough to determine whether or not preferential flow paths appeared to be present. The results appear to indicate that any of the methods involved will work. Difficulties arose not from the development of preferential flow paths around the edges of the specimens, but from the loss of bentonite from the edges during handling.

Keywords: Geosynthetic Clay Liner, Flexible Wall Permeameter, Permeability, Hydraulic Conductivity

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