Technical Paper by T.D. Stark and H.T. Eid

Shear Behavior of Reinforced Geosynthetic Clay Liners

ABSTRACT: Ring shear tests were performed to evaluate the effect of bentonite on the interface shear strength between a geomembrane and a reinforced geosynthetic clay liner (GCL), and the internal shear behavior of reinforced GCLs. The tests yielded an interface shear strength that is in agreement with the back-calculated shear strength values from two test pads that deformed at the U.S. Environmental Protection Agency GCL Test Section in Cincinnati, Ohio, USA. The ring shear tests showed that the internal shear strength of reinforced GCLs depends on the following two factors: (i) resistance against reinforcing fiber pull-out and/or tearing; and, (ii) shear strength of bentonite. The laboratory tests indicate that fiber resistance is the predominant internal shear strength factor. Recommendations are presented for the rate of shear that should be used for internal shear testing of reinforced GCLs.

KEYWORDS: Geosynthetic clay liner, Strength, Stability, Slope, Shearbox test, Shear rate, Ring shear test.

AUTHORS: T.D. Stark, Associate Professor of Civil Engineering, 2217 Newmark Civil Engineering Laboratory, University of Illinois, 205 N. Mathews Ave., Urbana, Illinois 61801, USA, Telephone: 1/217-333-7394, Telefax: 1/217-333-9464; and H.T. Eid, Post-Doctoral Research Associate, b213 Newmark Civil Engineering Laboratory, University of Illinois, 205 N. Mathews Ave., Urbana, Illinois 61801, USA, Telephone: 1/217-333-6990, Telefax: 1/217-333-9464.

PUBLICATION: *Geosynthetics International* is published by the Industrial Fabrics Association International, 345 Cedar St., Suite 800, St. Paul, Minnesota 55101-1088, USA, Telephone: 1/612-222-2508, Telefax: 1/612-222-8215. *Geosynthetics International* is registered under ISSN 1072-6349.

DATES: Original manuscript received 17 October 1996, revised version received 30 December 1996 and accepted 3 January 1997. Discussion open until 1 September 1997.

REFERENCE: Stark, T.D. and Eid, H.T., 1996, "Shear Behavior of Reinforced Geosynthetic Clay Liners", *Geosynthetics International*, Vol. 3, No. 6, pp. 771-786.

1 INTRODUCTION

In recent years, geosynthetic clay liners (GCLs) are increasingly being chosen to replace compacted clay liners (CCLs) in composite liner and cover systems for waste containment facilities. The acceptance and use of GCLs in waste containment facilities has increased annually since 1986. Market consumption of GCLs in 1995 was 5 million square meters and is estimated to increase by 8% to 5.4 million square meters in 1996 (Jagielski 1994; Anonymous 1996). Some of the advantages of GCLs over CCLs are: (i) lower and more predictable cost; (ii) prefabricated/manufactured quality; (iii) easier and faster construction; (iv) field hydraulic conductivity testing is not required; (v) availability of engineering properties; (vi) resistance to the effects of wetting/drying and freeze/thaw cycles; (vii) more airspace resulting from the smaller thickness; and, (viii) easier repair. Some of the disadvantages of GCLs over CCLs include: (i) smaller leachate attenuation capacity; (ii) shorter containment time; (iii) potential for lower external and internal shear strength; (iv) possible post-peak shear strength loss; and, (v) possible higher long-term flux due to a reduction in bentonite thickness under the applied normal stress (Anderson and Allen 1995; Anderson 1996).

The two main types of reinforced GCLs consist of a thin layer of bentonite (approximately 3 to 4 mm thick) sandwiched between two geotextiles (Figure 1). The bentonite typically used in GCLs is Wyoming bentonite, which consists of 90 to 95% montmorillonite, and 5 to 10% quartz and feldspar. Typical values of the liquid limit and plasticity index are 300 to 450, and 260 to 390, respectively (Mesri 1969; Mesri and Olson 1970).

Hydrated bentonite exhibits an extremely low shear strength, which has caused many slope stability problems in geotechnical engineering projects. This suggests that the stability of landfill liner and cover slopes constructed with a GCL (i.e. a seam of bentonite) might be susceptible to static and seismic instability. As a result, the bentonite layer in

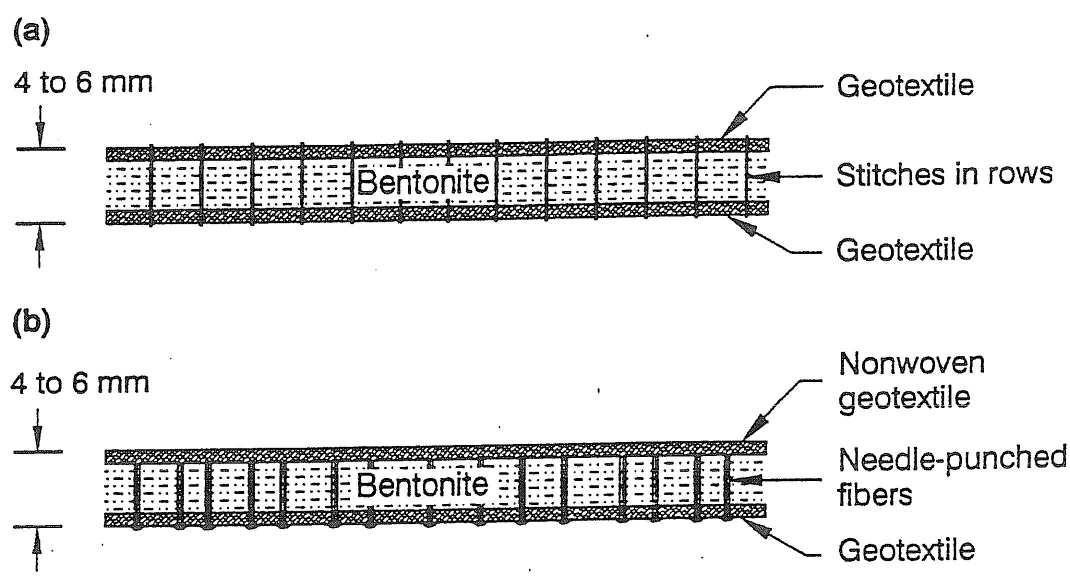

Figure 1. Cross sections of different available reinforced GCLs: (a) stitch-bonded bentonite between upper and lower geotextiles; (b) needle-punched bentonite through upper and lower geotextiles.

a GCL can be reinforced by connecting the geotextiles on either side using stitch-bonding or needle-punching to increase the internal shear strength of the GCL (Figure 1).

There are two possible failure modes for GCLs: (i) sliding at the interface between the top or bottom of the GCL and the adjacent material (interface failure); and, (ii) sliding through the bentonite or midplane of the GCL (internal failure). Since the interface and internal shear strength of GCLs are project specific and product dependent, this paper concentrates on discussing the effect of bentonite and GCL reinforcement on the shear behavior of reinforced GCLs rather than providing GCL design values.

2 INTERFACE SHEAR STRENGTH OF HYDRATED REINFORCED GCLS

2.1 General

The U.S. Environmental Protection Agency (USEPA) recommends intimate contact between the GCL (i.e. bentonite) and the geomembrane so that any liquid passing through a hole in the geomembrane cannot spread laterally and approach the clay (i.e. underlaying barrier material) over a greater wetted area than the hole itself. However, the interface shear strength of the GCL can be affected by the extrusion of hydrated bentonite through the woven geotextile of the GCL into the adjacent geomembrane interface (Byrne 1994). Bentonite from a bentonite impregnated geotextile can also migrate into the GCL-geomembrane interface. If a geomembrane is placed in contact with the woven or bentonite impregnated geotextile of the GCL, a slope stability problem may develop along the geomembrane-geotextile interface. This problem was observed at the GCL slope stability research project in Cincinnati, Ohio, USA (Koerner et al. 1996; Scranton 1996) and in a landfill liner system at a landfill site near Hong Kong (Cowland 1997).

As part of the GCL slope stability research project in Cincinnati, Ohio, four GCL manufacturers participated in the construction of fourteen landfill cover test pads (Koerner et al. 1996; Scranton 1996). The GCL test pads were constructed at an operating waste containment facility under the supervision of the USEPA. Nine test pads were constructed on a 1 vertical:2 horizontal (1V:2H) slope, while five test pads were constructed on a 1V:3H slope. The test pads constructed on the 1V:2H slope were 7.5 to 10.5 m wide and approximately 20.5 m long. Figure 2 presents a plan view of the test pads constructed on the 1V:2H slope.

Figure 3 presents a detailed cross section of GCL Test Pad G that was constructed on a 1V:2H slope. It can be seen that the GCL was placed on the natural subgrade and overlain by a 1.5 mm (60 mil) thick textured high density polyethylene (HDPE) geomembrane. The geomembrane was manufactured by National Seal Company of Aurora, Illinois, USA. The geomembrane was textured on both sides and overlain by a geocomposite drain and 0.9 m of compacted soil.

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 toward the bottom of the 1V:2H slope (Koerner et al. 1996; Scranton 1996). The GCL installed in Test Pad H consisted of bentonite between two woven slit-film multifilament geotextiles that are stitch-bonded together. The GCL is manufactured by Colloid Environmental Technolo-

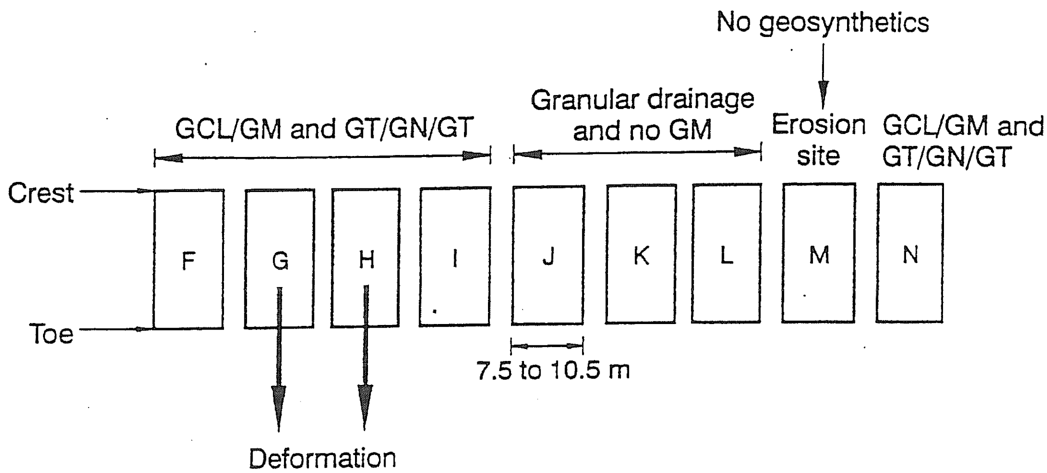

Figure 2. Plan view of GCL test pads on the 1V:2H slope (after Scranton 1996).
Notes: GM = geomembrane; GT = geotextile; GN = geonet; GT/GN/GT = geocomposite drain.

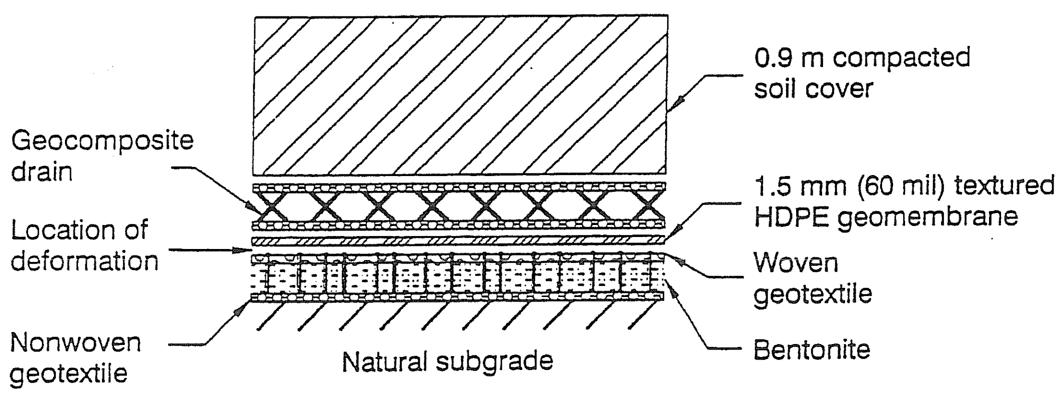

Figure 3. Detailed cross section of Test Pad G (after Scranton 1996).
Note: Drawing is not to scale.

gies Company (CETCO) of Arlington Heights, Illinois, USA and is referred to as CLAYMAX 500SP.

Approximately 50 days after installation of Test Pad G, the overlying textured geomembrane also pulled out of the anchor trench and displaced approximately 20 m toward the bottom of the 1V:2H slope (Koerner et al. 1996; Scranton 1996). The GCL installed in Test Pad G consisted of bentonite between a woven and nonwoven geotextile that are needle-punched together to provide internal support. This GCL is also manufactured bý CETCO and is referred to as BENTOMAT ST.

In summary, deformation occurred at the interface between the woven geotextile and the overlying textured geomembrane (Figure 3) in Test Pads G and H. After deformation, it was observed that the upper surface of the GCLs were slippery indicating that hydrated bentonite likely extruded through the woven geotextile (Koerner et al. 1996;

Scranton 1996). It is believed that the bentonite was hydrated by moisture from the natural subgrade and possible surface infiltration. The deformation was caused by the low shear resistance of the bentonite that extruded into the interface. The deformation observed in Test Pads G and H and other field experiences (Cowland 1997) clearly illustrate the importance of the GCL-geomembrane interface on the stability of landfill cover and liner systems.

2.2 Three-Dimensional Slope Stability Analysis of Test Pad G

The observed deformation in Test Pads G and H provide a unique opportunity to estimate the field or mobilized shear strength of the GCL-geomembrane interface using a regressive slope stability analysis. Figure 4 presents a cross section of Test Pads G and H at the GCL Test Section in Cincinnati, Ohio. It is important to note that the geosynthetics did not extend beyond the cover soil. Field observations (Daniel 1996) and photographs (Koerner et al. 1996) suggest that deformation occurred within the compacted cover soil on a vertical plane directly above the ends or edges of the geosynthetics. Therefore, shear resistance from the compacted cover soil was mobilized during the deformation of Test Pads G and H (Figure 4). In addition, the geosynthetics overlying the GCL pulled out of the anchor trench, providing additional shear resistance. In summary, shear resistance was developed in Test Pads G and H in the following locations: along the GCL-geomembrane interface; on two vertical planes in the compacted cover soil; and, from geosynthetics pulling out of the anchor trench at the onset of deformation.

It should be noted that the actual slope inclination of Test Pads G and H was 23.5 and 24.7°, respectively (Scranton 1996). These slope inclinations are less than the target inclination of 26.6° or 1V:2H (Koerner et al. 1996). The difference in the actual slope inclinations may provide insight into the difference in the time-to-failure for Test Pads G and H. As noted previously, Test Pads G and H deformed 50 and 20 days after construction, respectively. Therefore, it appears that 30 additional days were required for a sufficient amount of bentonite to hydrate and extrude into the GCL-geomembrane interface and cause deformation of Test Pad G due to a smaller slope inclination of 23.5°. Since a back-analysis of Test Pad G will yield a lower GCL-geomembrane interface strength, Test Pad G was used in the three-dimensional (3-D) regressive analysis to estimate the mobilized interface shear strength. The width and length of Test Pad G are 9.0 and 20.5 m, respectively (Scranton 1996).

Bishop's simplified method of slices (Bishop 1955) was used to conduct the 3-D limit equilibrium regressive analysis. Bishop's simplified method was extended to three-di-

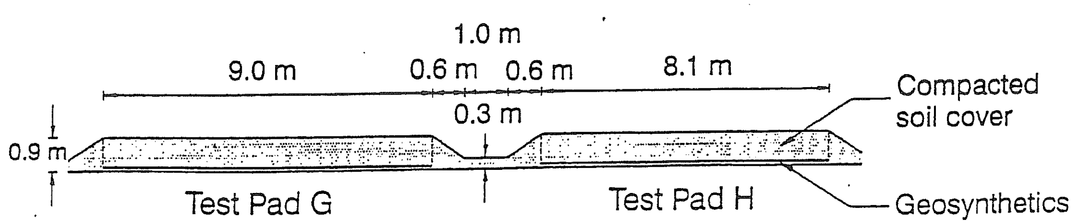

Figure 4. Cross section of Test Pads G and H in the GCL test section.

mensions by Hungr (1987) and coded into the microcomputer program CLARA 2.31 (Hungr 1988). The cover soil, a low plasticity glacial till with 15 to 20% passing the 0.075 mm sieve (U.S. Standard Sieve No. 200), was compacted into a 0.3 m lift using a bulldozer (Koerner 1996). The friction angle and total unit weight of the cover soil were estimated to be 35° and 19 kN/m^3, respectively. The friction angle of the glacial till was estimated using a relationship with liquid limit, clay-size fraction and normal stress presented by Stark and Eid (1997). An effective normal stress of 17 kPa, clay-size fraction less than 20% and low plasticity were used for this estimation. Index property and shear strength data for the glacial till were not reported by Koerner et al. (1996) or Scranton (1996). Failure surfaces consisting of the GCL-geomembrane interface and two approximately vertical planes through the cover soil (Figure 4) were analyzed.

The backfill used to cover the geosynthetics in the 0.6 m deep and 0.3 m wide anchor trench was assumed to have an average total unit weight of 16 kN/m^3, and the textured geomembrane-soil interface friction angle was 30° (Scranton 1996). The geosynthetics were only installed along the vertical face of the trench closest to the slope; therefore, a shear resistance value caused by geosynthetic pull-out was estimated for the vertical face and applied in the analysis as a line load of 1.4 kN/m along the top of the slope. The at-rest resultant earth pressure force and the geomembrane-soil and GCL-geomembrane interface friction angles were used to estimate the shear resistance developed along the vertical face of the anchor trench. The peak GCL-geomembrane interface friction angle (22.5°) was measured in the ring shear tests described below and used in the Bishop's simplified method analysis.

The 3-D back-analysis yielded a mobilized GCL-geomembrane interface friction angle of approximately 21.5° for Test Pad G. Since the slope angle for Test Pad G was 23.5°, the 3-D effects contributed approximately 2° (23.5° to 21.5°) or 9% to the mobilized friction angle. A 3-D back-analysis of Test Pad H yielded a mobilized GCL-geomembrane interface friction angle of approximately 23.0°. This difference is in agreement with the fact that Test Pads G and H have inclinations of 23.5° and 24.7°, respectively. The difference in the mobilized friction angles may be attributed to the different GCLs, i.e. different reinforcement techniques and different woven geotextiles, and possibly different hydration patterns. However, both mobilized friction angles are approximately 2° less than the slope inclination. The width and length of Test Pad H are 8.1 m and 20.5 m, respectively.

2.3 Ring Shear and Direct Shear Test Results for Test Pad G

Two torsional ring shear tests were performed to simulate the field conditions in Test Pad G. A modified Bromhead ring shear apparatus, that uses an annular specimen with an inside and outside diameter of 40 and 100 mm, respectively, was used for these tests. For each test specimen, the nonwoven geotextile of the needle-punched GCL was glued to the bottom platen of the ring shear apparatus and the woven geotextile was placed against the loading platen. Gluing the nonwoven geotextile to the bottom platen simulates the lack of displacement observed at the nonwoven geotextile-natural subgrade interface in the field (Scranton 1996). The woven geotextile extended beyond the nonwoven geotextile and was cut into eight extended wedges or flaps. The flaps were glued to the lower side of the bottom platen. This reduced the amount of lateral migration of bentonite during hydration and the potential for internal failure. The edges of the GCL specimen were moistened using a spray bottle to reduce bentonite loss during specimen trimming. A 1.5 mm textured HDPE geomembrane specimen, manufactured by National Seal Company and similar to the HDPE geomembrane specimen installed in Test Pad G, was glued to the top platen to form a textured GCL-geomembrane interface.

The first GCL specimen was hydrated with distilled water under a normal stress of 17 kPa (typical normal stress in the field). Distilled water filled the water bath surrounding the specimen and hydrated the annular specimen from the inner and outer edges. The specimen was left to swell or hydrate until the vertical deformation ceased. The hydration stage required approximately two weeks to complete. The specimen was then sheared at a shear displacement rate of 0.5 mm/minute. This rate is slower than the standard rate of 1.0 mm/minute recommended in the ASTM D 5321 standard test method, and more closely simulates a drained condition. The second specimen was sheared at the same shearing rate but within two minutes after being inundated with distilled water under a normal stress of 17 kPa. This did not allow the bentonite to completely hydrate prior to or during shear. The difference in hydration is reflected in the final water contents of the bentonite. The fully and partially hydrated bentonite, under a normal stress of 17 kPa, had final water contents of 144 and 73%, respectively. The water content of the bentonite prior to hydration was approximately 10%.

Figure 5 presents the shear stress-displacement relationships from the two ring shear tests used to simulate the GCL-geomembrane interface in Test Pad G. It can be seen that the ring shear test on the fully hydrated GCL-textured geomembrane specimen interface yields peak and residual friction angles of approximately 22.5° and 15°, respectively. The partially hydrated GCL-textured geomembrane specimen interface yields peak and residual friction angles of approximately 37.5° and 24°, respectively. Therefore, the interface friction angles for the fully hydrated GCL specimen are approximately 60% of the corresponding friction angle values for the partially hydrated GCL speci-

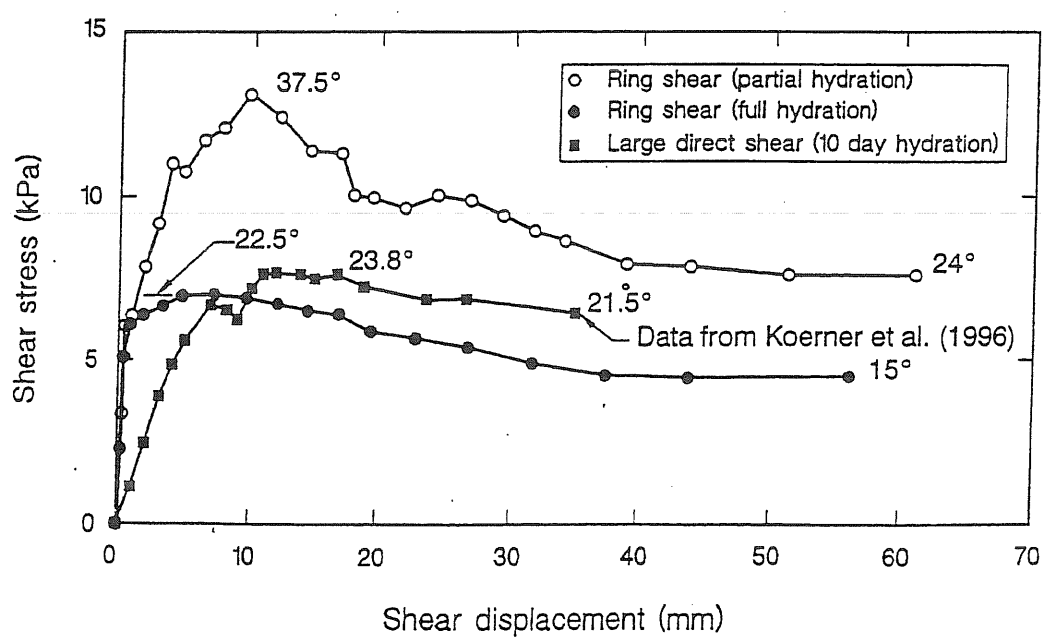

Figure 5. Ring shear and direct shear test results for the GCL-geomembrane interface in Test Pad G.

men. Visual inspection of the specimens after shearing showed that failure occurred at the GCL-geomembrane interface. This is in agreement with Byrne (1994), who states that failure will occur at the GCL-geomembrane interface at normal stresses less than 96 kPa. However, more bentonite extruded through the woven geotextile into the interface with the hydrated specimen than with the partially hydrated specimen. This likely resulted in the reduced friction angles for the hydrated specimen.

Figure 5 also presents the shear stress-displacement relationship for a 0.3 m by 0.3 m direct shear test (large direct shear test) on the same interface (Koerner et al. 1996). The normal stress and hydration time were 17.2 kPa and 10 days, respectively. In accordance with ASTM D 5321, the shearing stress was applied at a rate of 1.0 mm/minute. The measured peak and "end-of-test" friction angles were approximately 24° and 21.5°, respectively (Figure 5). The friction angle of 21.5° is termed the end-of-test friction angle because the limited shear displacement (approximately 35 mm) in the direct shear apparatus did not allow a residual condition to develop. It should be noted that the direct shear test also resulted in bentonite extrusion through the woven geotextile, which is in agreement with field observations.

The deformation of Test Pad G enables a comparison of the mobilized and laboratory measured shear strengths of the GCL-geomembrane interface. This comparison provides insight with respect to the accuracy or reliability of laboratory test results for GCL design purposes. The ring shear test on the fully hydrated GCL specimen yielded a peak friction angle (22.5°) which is in good agreement with the mobilized friction angle (21.5°). The peak friction angle from the direct shear test (24°) is slightly higher than the mobilized friction angle. The difference in the peak friction angles obtained from the direct shear and ring shear tests may be caused by the difference in the modes of shear and shear displacement rates.

Figure 5 shows a substantial post-peak strength loss for the GCL-textured geomembrane interface, which is particularly noticeable in the ring shear test results; yet, the peak friction angle was used for comparisons with the mobilized friction angle of Test Pad G. It is believed that progressive failure could have played a role in the observed deformation; however, after failure it was observed that the majority of the GCL top surface was slippery indicating that moisture from the bentonite had hydrated a large portion of the slope. In addition, the post-construction deformations in Test Pad G were constant with time and actually showed an upslope movement of less than 10 mm (Scranton 1996). Test Pad G failed suddenly and after showing no signs of imminent failure. Based on the lack of downslope movement, the rapid nature of the failure, and the infinite slope geometry of the sliding mass, it was concluded that the zone of progressive failure was likely limited in area. As a result, the peak friction angles obtained from laboratory measured shear strengths were used for comparison with the mobilized friction angle.

Clearly, one case history does not allow a definitive conclusion to be drawn with regard to the effectiveness of laboratory shear tests on GCLs. However, a 0.3 m by 0.3 m direct shear or torsional ring shear apparatus typically can be used to provide reasonable estimates of field GCL-geomembrane interface shear strength for landfill cover systems. The ring shear apparatus appears to be advantageous in determining the residual GCL-textured geomembrane interface strength.

2.4 GCL Design and Installation Considerations

Test Pad G mobilized a friction angle of approximately 21.5° along the GCL-textured geomembrane interface. This mobilized friction angle is greater than the peak friction angle of hydrated bentonite (approximately 16°) at an effective normal stress of 17 kPa. This peak friction angle of hydrated bentonite was estimated using a ring shear test on a GCL product comprising bentonite adhered to a textured HDPE geomembrane. Failure occurred through the hydrated bentonite, which exhibited peak and residual friction angles of 16° and 6°, respectively, at a normal stress of 17 kPa. The specimen was hydrated by filling the ring shear water bath with distilled water and allowing the bentonite to hydrate from the inner and outer edges of the annular specimen. The specimen was assumed to be hydrated when the swell or vertical deformation ceased.

Since the mobilized friction angle is greater than the peak friction angle of hydrated bentonite (approximately 16°), it may be assumed that interlocking between the textured geomembrane surface and the woven geotextile was mobilized when Test Pad G deformed. The effectiveness of this interlocking depends on the type of textured geomembrane and geotextile at the interface (Stark et al. 1996) and the amount of bentonite that extrudes into the GCL-geomembrane interface. It is recommended that the time between GCL placement and normal stress application be minimized to reduce the amount of bentonite extrusion into the interface and maximize the contact area of the GCL-geomembrane interface.

3 EFFECT OF SHEAR DISPLACEMENT RATE ON THE INTERNAL SHEAR STRENGTH OF HYDRATED REINFORCED GCLS

The shear stresses transmitted by overlaying material may cause an internal shear failure through the hydrated bentonite and reinforcement of the GCL. As a result, internal failure is typically a concern when a textured geomembrane is placed in contact with a GCL. The main objective of needle-punching or stitch-bonding GCLs is to prevent a failure from developing through the bentonite. This is intended to shift the failure surface to the GCL-geomembrane interface or another interface that exhibits a lower peak shear strength.

One of the main concerns when estimating the internal shear strength of reinforced GCLs using laboratory tests is choosing the correct shear displacement rate. The shear displacement rate chosen also has significant implications on the cost and scheduling of commercial GCL testing, and thus, the acceptance or marketability of GCLs may also be affected. To address this question, two series of ring shear tests were conducted on needle-punched GCL specimens to study the effect of shear displacement rate on internal shear strength. The needle-punched GCL selected is manufactured by the National Seal Company and is referred to as Bentofix Thermal-Lock. It consists of powdered bentonite sandwiched between nonwoven and woven geotextiles. In each series of tests, seven different specimens were sheared at displacement rates of 0.015, 0.045, 0.15, 0.5, 1.5, 18.5, and 36.5 mm/minute. All of the tests were conducted at an effective normal stress of 17 kPa to simulate a landfill cover system.

In the first series of tests, the bentonite was emptied from the GCL specimen before testing. This was accomplished by holding the specimens vertically and lightly tapping the geotextiles with one finger. The specimen was rotated and lightly tapped until all of the bentonite fell out of the GCL. In the second series of tests, the bentonite was not removed from the specimens and the specimens were sheared after hydration under a normal stress of 17 kPa. Hydration usually required two weeks and was assumed to be complete when the vertical deformation or swelling ceased. Internal shear testing of GCLs with and without bentonite helped to differentiate the effect of the shear displacement rate on the bentonite and the reinforcing fibers.

All of the test specimens were prepared by cutting the GCL into a circle with a diameter of approximately 160 mm. The diameter of the nonwoven geotextile face was then reduced to 100 mm (i.e. the outside diameter of the ring shear specimen container). The woven geotextile that extended beyond the edge of the nonwoven geotextile was cut to create eight extended wedges or flaps. A circular hole with a diameter of 40 mm was cut in the center of the specimen. For the specimens filled with bentonite, the edges were moistened using a spray bottle to reduce bentonite loss during trimming. Figure 6 shows a GCL specimen and a disassembled view of the ring shear specimen container. From the top of Figure 6 to the bottom the items are: top platen, upper plastic ring, GCL, lower plastic ring, and bottom platen. The upper plastic ring, which has inside and outside diameters of 40 and 100 mm, respectively, was then centered over the woven geotextile. The eight woven geotextile wedges were then folded over and adhered to the

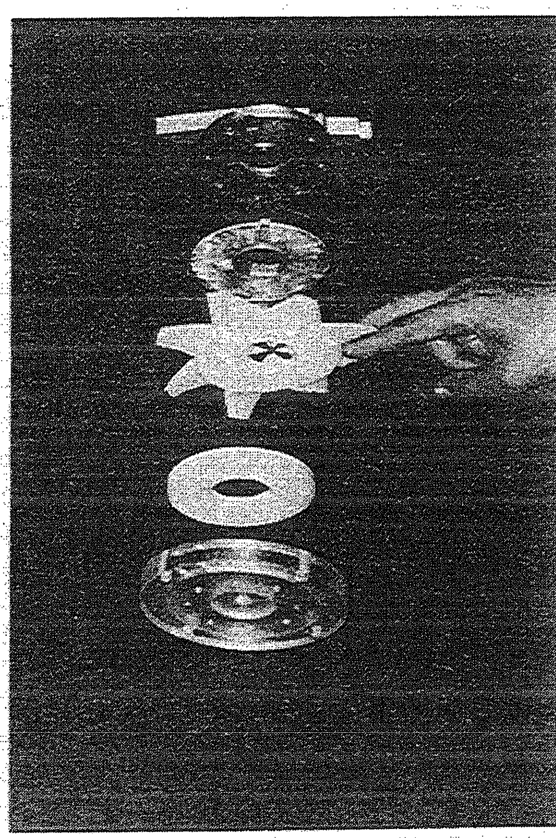

Figure 6. Photograph of the ring shear specimen container.

reverse side of the plastic ring using an adhesive. This side of the plastic ring (with the specimen attached to it) was then glued to the top platen of the ring shear apparatus. It should be noted that the woven geotextile overlying the annular specimen was not glued directly to the lower face of the upper plastic ring. This was done to prevent bonding of the GCL reinforcing fibers and slit-film fibers of the woven geotextile to the upper plastic ring. The upper plastic ring and the top platen were also connected through a tongue and groove joint to prevent shearing at the glue-geotextile interface. The nonwoven geotextile face of the specimen was then glued to the lower plastic ring that is attached to the bottom platen of the ring shear apparatus using four screws.

Figure 7 shows the assembled specimen container and an unfilled GCL specimen prior to installation in the ring shear apparatus. Just below the tongue and groove joint between the top platen and the upper plastic ring, the needle-punched reinforcing fibers can be seen due to the absence of bentonite. The specimen was then loaded in increments to a normal stress of 17 kPa. The adhesive was cured for 24 hours under a normal stress of 17 kPa before inundation with distilled water. The unfilled and filled GCL specimens were then submerged in distilled water. The unfilled specimens were sheared within two hours of submergence in the distilled water, while the filled specimens were completely hydrated prior to shearing.

Figure 8 presents a typical shear stress-shear displacement relationship for the bentonite filled and unfilled GCL specimens. The peak and residual shear strengths were reached at 10 to 15 mm, and approximately 70 mm of shear displacement, respectively. Visual inspection of the failed unfilled and filled GCL specimens revealed that shearing occurred just below the woven geotextile. The internal failure was caused by pull-out or tearing of the reinforcing fibers, and thus separation of the woven geotextile from the remainder of the GCL. In summary, a shear displacement of 10 to 15 mm was large enough to initiate pull-out or tearing of the reinforcing fibers from the woven geotextile.

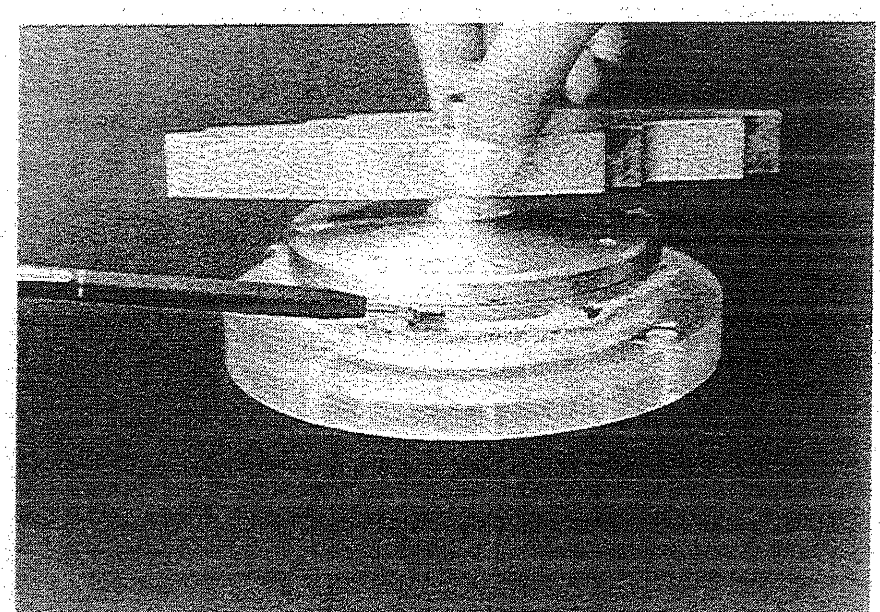

Figure 7. Ring shear GCL specimen before internal shear strength testing.

Note: Pencil is pointing at the tongue and groove joint between the top platen and upper plastic ring.

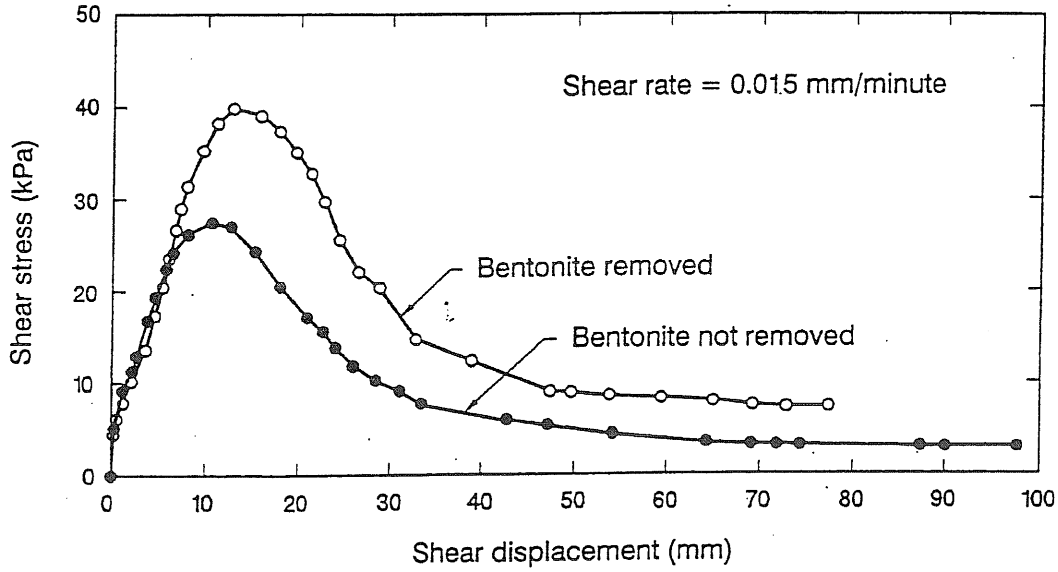

Figure 8. Ring shear test results for bentonite filled and unfilled reinforced GCL specimens at a normal stress of 17 kPa.

This small shear displacement has important implications for the shear strength that should be used for the static and seismic design of GCL slopes. Continued shearing gradually oriented the pulled out or torn reinforcing fibers in the direction of shear. Fiber orientation reduced the shearing resistance until a residual strength condition was achieved. In the case of the bentonite filled GCL specimens, orientation of bentonite particles also contributed to the post-peak shear strength reduction.

Figure 9 presents the peak and residual internal shear stresses of the unfilled and filled GCL specimens tested at shear displacement rates ranging from 0.015 to 36.5 mm/minute. A shear displacement rate of 36.5 mm/minute is the fastest rate possible using the modified Bromhead ring shear apparatus. It can be seen that the peak shear stress of the unfilled GCL (represented by the dashed line) increases with increasing shear displacement rate. This increase is attributed to the faster shear rates that tear the reinforcing fibers instead of allowing a slow gradual pull-out of the fibers from the woven geotextile. This rapid tearing of the reinforcing fibers results in a higher shear resistance. Also, it should be noted that the peak shear stress of the unfilled GCL is approximately constant at shearing rates less than approximately 0.04 mm/minute.

Figure 9 also shows that the bentonite filled GCL specimens gave lower peak shear stresses than the unfilled GCL specimens. One might suspect that the shear strength of the bentonite filled GCL would yield a higher strength than the unfilled GCL because the bentonite and reinforcing fiber strengths might be additive. Due to the following reasons the test results suggest that the strengths are not additive: (i) the fibers are stretched or placed in tension prior to shear by the hydrating bentonite, which may decrease the internal strength; (ii) extrusion usually occurs at the needle-punch locations, thus, hydrated bentonite may facilitate pull-out of the reinforcing fibers by lubricating the reinforced fiber-woven geotextile junctions; (iii) the woven geotextile of the unfilled GCL specimen may be in contact with the nonwoven geotextile during shear be-

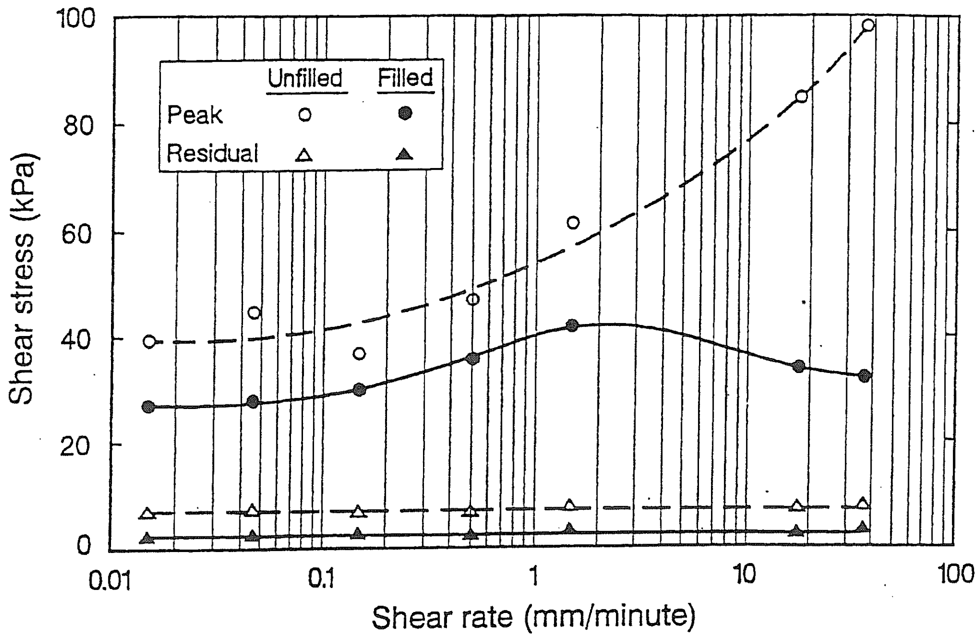

Figure 9. Effect of shear displacement rate on GCL internal shear strength at a normal stress of 17 kPa.

cause of the absence of bentonite; and (iv) creation of positive excess pore-water pressures in the bentonite filled GCL specimen.

The comparison shown in Figure 9 suggests that excess pore-water pressures do not significantly affect the internal shear strength until the shear displacement rates exceed approximately 1.5 mm/minute. This is evident from the difference between the peak shear stress trend lines for the unfilled and filled GCL specimens that is essentially constant for shearing rates less than 1.5 mm/minute. Shearing bentonite at rates greater than 1.5 mm/minute appears to create excess pore-water pressures that reduce the internal peak shear stress (Figure 9). Mesri and Olson (1970) show a similar decrease in peak principal stress difference in isotropically consolidated-undrained triaxial compression tests on sodium montmorillonite for deformation rates greater than 0.04 mm/minute. Sodium montmorillonite is the predominant bentonite clay mineral in the GCLs used for this study.

The data presented in Figure 9 suggests that a shear rate less than 0.04 mm/minute will not: (i) create excess pore-water pressures in the bentonite that will significantly affect the measured peak internal shear strength of a reinforced GCL; and, (ii) result in a strength increase due to rapid pull-out or tearing of the reinforcing fibers. Therefore, it is recommended that a shear rate of 0.04 mm/minute be used for evaluating the internal shear strength of reinforced GCLs for cover systems. A shear rate of 0.04 mm/minute was selected because it is the fastest displacement rate that can be used before the measured peak shear stress starts to increase. This shear rate is slower than the standard rate of 1.0 mm/minute (ASTM D 5321) and will likely result in an increase in the cost of laboratory GCL testing. However, it is important to note that this recommendation is based on testing of only one needle-punched GCL product at a normal stress of 17 kPa.

The data in Figure 9 also suggests that the residual internal shear stress is independent of the shear rate. In the bentonite unfilled GCL, this is attributed to the reinforcing fibers being already torn or pulled out and oriented parallel to the direction of shear. In the bentonite filled GCL, the torn or pulled out reinforcing fibers are immersed in the hydrated bentonite and oriented parallel to the direction of shear. This condition also appears to be unaffected by the rate of shear.

The residual shear stress data in Figure 9 corresponds to residual friction angles of 22° and 8° for the unfilled and filled GCL specimens, respectively. It should also be noted that the residual friction angle of 8° for the filled GCL is slightly greater than the residual friction angle of 6° for the hydrated bentonite at an effective normal stress of 17 kPa. This may be attributed to the pulled out or torn reinforcing fibers being immersed in the hydrated bentonite which increases the residual friction angle of the filled GCL specimens.

4 CONCLUSIONS

The purpose of this study was to determine the effect of bentonite on the interface and internal shear behavior of reinforced goesynthetic clay liners (GCLs). Reinforced GCLs were sheared internally and against a textured geomembrane using a torsional ring shear apparatus to study the shear behavior. The following conclusions are based on the data and interpretations presented in this paper:

1. Extrusion of hydrated bentonite through the woven geotextile of a reinforced GCL reduces the GCL-textured geomembrane interface shear strength. However, this interface shear strength value does not reduce to the shear strength value of hydrated bentonite due to interlocking between the textured geomembrane surface and the woven geotextile.
2. Torsional ring shear test results revealed that hydration of a GCL reduces the measured peak and residual friction angles of the GCL-textured geomembrane interface by approximately 40%. This reduction is likely caused by extrusion of bentonite through the woven geotextile of the GCL and into the GCL-geomembrane interface. The peak interface friction angle measured using a 0.3 m by 0.3 m direct shear or a ring shear apparatus is in good agreement with the friction angle back-calculated using a three-dimensional analysis of Test Pad G that deformed at the U.S. Environmental Protection Agency GCL Test Section in Cincinnati, Ohio, USA.
3. The internal shear strength of reinforced GCLs depends on two factors: (i) the resistance against pull-out and/or tearing of the reinforcing fibers; and, (ii) the shear strength of bentonite. The laboratory test results presented herein indicate that fiber resistance appears to be the predominant factor at a normal stress of 17 kPa. Fiber resistance also appears to increase with an increasing rate of displacement.
4. It is recommended that a shear displacement rate of 0.04 mm/minute or less be used for laboratory testing of the internal shear strength of reinforced GCLs for cover systems. It is important to note that this recommendation is based on tests using only one needle-punched GCL at a normal stress of 17 kPa. Further study is recommended.

ACKNOWLEDGMENTS

This study was funded by the Illinois Office of Solid Waste Research, Grant No. SENR-SWRI-OSWR-10-002, and the Illinois Hazardous Waste Research and Information Center, Grant No. SENR-HWR-93-113. The first author also acknowledges the support provided by the W.J. and E.F. Hall Scholar award. National Seal Company and Colloid Environmental Technology Company supplied the materials for the laboratory testing. This support is gratefully acknowledged. The authors also appreciate the extensive and helpful review comments provided by Professor D.E. Daniel. The contents and views in this paper are the authors' and do not necessarily reflect those of any of the contributors.

REFERENCES

Anderson, J.D., 1996, "Are Geosynthetic Clay Liners (GCLs) Really Equivalent to Compacted Clay Liners", *Geotechnical News*, BiTech, Vol. 14, No. 2, pp. 20-23.

Anderson, J.D. and Allen, S.R., 1995, "What Are the Real Design Considerations When Using a Geosynthetic Clay Liner (GCL)", *Proceedings of the Ninth Annual Municipal Solid Waste Management Conference*, Texas Natural Resource Conservation Commission, Austin, Texas, USA, December 1995, pp. 15-29.

ASTM D 5321, "*Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method*", American Society for Testing and Materials, West Conshohocken, Pennsylvania, USA.

Bishop, A.W., 1955, "The Use of the Slip Circle in the Stability Analysis of Slopes", *Geotechnique*, Vol. 5, No. 1, pp. 7-17.

Byrne, R.J., 1994, "Design Issues with Strain-Softening Interfaces in Landfill Liners", *Proceedings of Waste Tech '94*, Charleston, South Carolina, USA, January 1994, Session 4, Paper 4, 26 p.

Cowland, J.W., 1997, "What is the Acceptable Shear Strength of a Geosynthetic Clay Liner?", *Testing and Acceptance Criteria for Geosynthetic Clay Liners*, Well, L.W., Editor, ASTM Special Technical Publication 1308, proceedings of a symposium held in Atlanta, Georgia, USA, January 1996, pp. 229-239.

Daniel, D.E., 1996, "Personal Communication".

Hungr, O., 1987, "An Extension of Bishop's Simplified Method of Slope Stability Analysis to Three Dimensions", *Geotechnique*, Vol. 37, No. 1, pp. 113-117.

Hungr, O., 1988, "*User's Manual: CLARA 2.31, Slope Stability Analysis in Two or Three Dimensions for IBM Compatible Microcomputers*", Oldrich Hungr Geotechnical Research, Inc., Vancouver, British Columbia, Canada, 88 p.

Anonymous, 1996, "Market Report Shows Geosynthetics Made Modest Gain in 1995", *Geotechnical Fabrics Report*, IFAI, Vol. 14, No. 4, p. 6.

Jagielski, K., 1994, "United States and Canada GCL Market Update", *Geotechnical Fabrics Report*, IFAI, Vol. 12, No. 3, p. 24.

Koerner, G.R., 1996, "Personal Communication".

Koerner, R.M., Daniel, D.E. and Bonaparte, R., 1996, "Current Status of the Cincinnati GCL Test Plots", *Proceedings of the Tenth GRI Conference on Field Performance of Geosynthetics and Geosynthetic Related Systems*, Drexel University, Philadelphia, Pennsylvania, USA, pp. 147-175.

Mesri, G., 1969, "*Engineering Properties of Montmorillonite*", Ph.D. Thesis, University of Illinois at Urbana-Champaign, Illinois, USA, 90 p.

Mesri, G. and Olson, R.E., 1970, "Shear Strength of Montmorillonite", *Geotechnique*, Vol. 20, No. 3, pp. 261-270.

Scranton, H.B., 1996, "*Field Performance of Sloping Test Plots Containing Geosynthetic Clay Liners*", M.Sc. Thesis, University of Texas at Austin, Austin, Texas, USA, 207 p.

Stark, T.D. and Eid, H.T., 1997, "Slope Stability Analyses in Stiff Fissured Clays", *Journal of Geotechnical and Environmental Engineering*, ASCE, to be published in Vol. 123, No. 4.

Stark, T.D., Williamson, T.A. and Eid, H.T., 1996, "HDPE Geomembrane/Geotextile Interface Shear Strength", *Journal of Geotechnical Engineering*, ASCE, Vol. 122, No. 3, pp. 197-203.