# PROCEEDINGS
# 20TH INTERNATIONAL MADISON WASTE CONFERENCE

Municipal & Industrial Waste

**April 1-2, 1998**

Department of Engineering Professional Development

University of Wisconsin-Madison

432 N. Lake Street

Madison, WI 53706

(608) 262-2061

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# Design Considerations for Geosynthetic Clay Liners

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**Abstract:** This paper utilizes a landfill slope failure involving a geosynthetic clay liner (GCL) to illustrate potential stability problems that can develop from the low shear strength of hydrated bentonite. In addition, the paper discusses some of the uncertainties associated with GCLs and recommendations for determining appropriate design parameters to account for the weak nature of hydrated bentonite.

## INTRODUCTION

A growing number of regulatory environmental programs are allowing GCLs to be used in lieu of all or a portion of the compacted clay liner in composite liner and cap systems. These products are relatively new, and significant concerns remain over their ability to be incorporated in many waste containment structures. Initial concerns with GCLs focused on hydraulic conductivity, hydraulic equivalence to compacted clay liners, and internal shear strength. More recently, interface shear strength, bearing capacity, and overall long term performance have come to the forefront of concern. New information suggests that there are special considerations that should be taken into account when utilizing a GCL in certain applications. To illustrate potential stability problems associated with GCLs, a case history is initially presented.

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Presented at the 20th International Madison Waste Conference, April 1-2, 1998, Department of Engineering Professional Development, University of Wisconsin-Madison.

## CASE HISTORY

### Site Background

Mahoning Landfill, Inc. (MLI) is located on a two hundred acre parcel of land near Youngstown, Ohio. In 1962, strip-mining operations initiated at this site to mine the Middle Kittanning Coal seam on the property. By 1970, coal mining had ceased and the site began accepting municipal solid waste (MSW) from Youngstown and surrounding communities. The MSW was used to fill in the high walls and ponds left by the strip-mining. This practice continued through 1976, at which time the site fell dormant for a period of 10 years. In 1986, the site re-established waste receipt and has been accepting MSW since. The site was purchased by MLI in 1992 and as a result of regulatory requirements began the process of permitting a state of the art solid waste containment facility. As a condition of the permit, issued on March 1, 1995, MLI agreed to relocate all of the existing waste on the property (an estimated 1.7 million cubic yards) into the new fully lined facility. In 1996, MLI received 136,209 tons of MSW according to the *1997 Ohio Solid Waste Facility Data Report*.

### Cell Layout

Because of the haphazard depositional nature of the existing waste at the site, the lowest cell (Cell 2 in Figure 1) of the new landfill was not built first as is typically done in landfill construction. Instead, the next lowest cell (Cell 1) was constructed initially in an area with no existing waste. Cell 1 was designed with a temporary leachate collection system which was to be utilized until the main leachate collection sump was constructed in Cell 2. The temporary leachate collection system would be disengaged when Cell 2 and its sump became operational. The cell layout is shown in Figure 1.

The temporary leachate collection system was simply a depression of the composite liner system along the westerly edge of Cell 1 (see Figure 2). Commonly referred to as the "temporary leachate collection trench," this temporary system was sloped from north to south (left to right in Figure 2) with a pump at the southerly low end. The trench was filled with sand and gravel and included a perforated pipe to aid in collection and transport to the pump.

The GCL in use at the facility is the Gundseal® product consisting of sodium bentonite adhered to a 60 mil high density polyethylene (HDPE) geomembrane. The GCL was installed with the bentonite in direct contact with the compacted clay liner. The composite bottom liner system for the floor of Cell 1 at MLI consists of a 3 ft. thick layer of compacted clay, the Gundseal®, a geonet, and a 10 oz. nonwoven geotextile. The geotextile was not heat bonded to the geonet. It is important to note that the compacted clay liner was compacted from 0 to 2 % wet of optimum at an

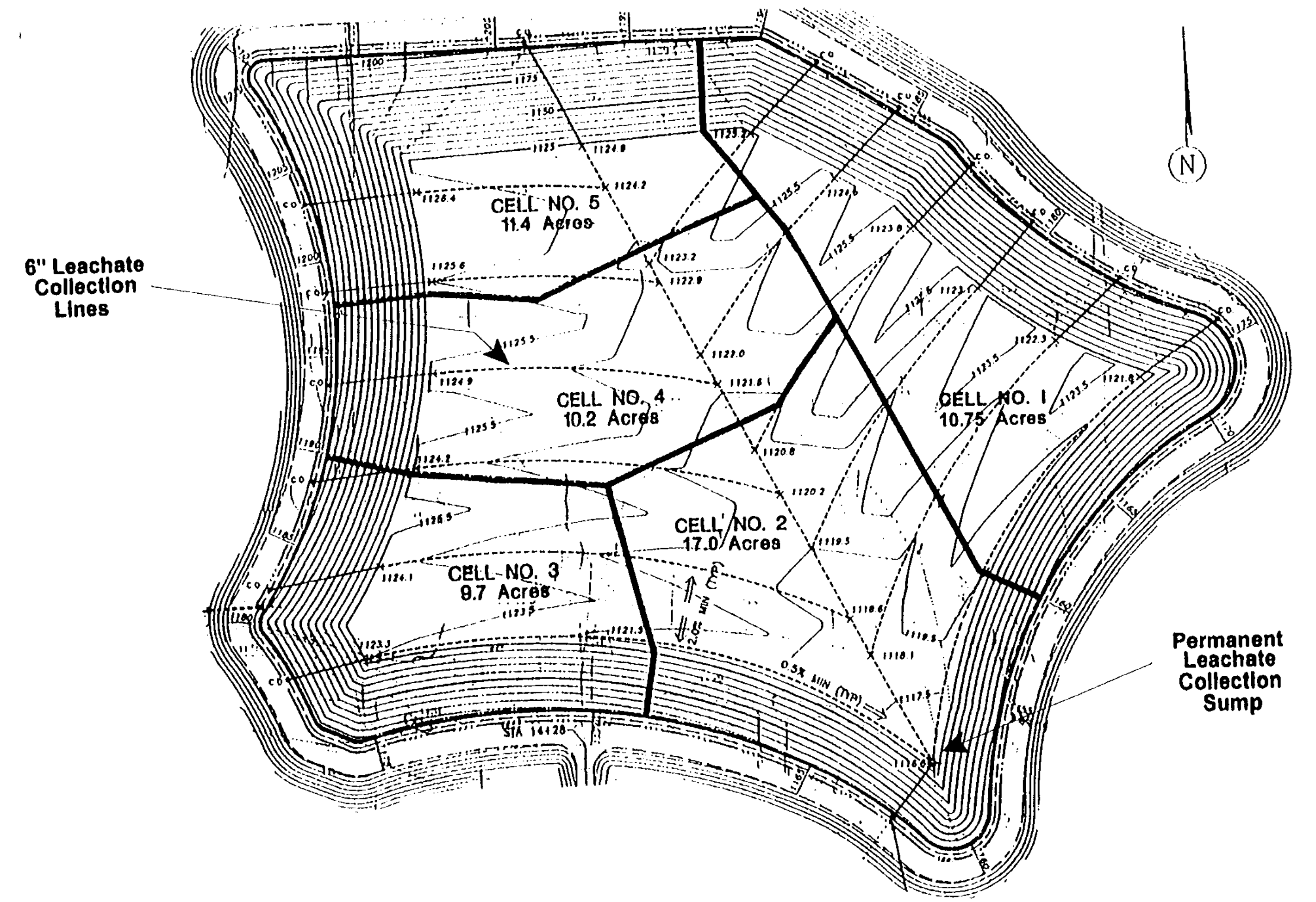


Figure 1. Cell layout.

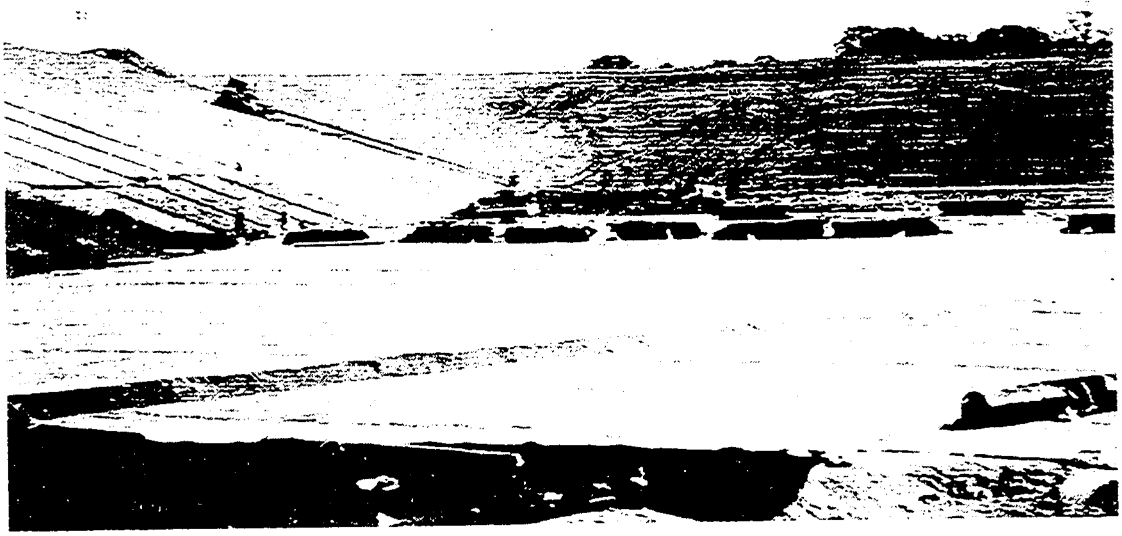

Figure 2. Temporary leachate collection system along the western edge of Cell 1B.

average water content of 11.5%. The composite bottom liner system on the 33 % internal side slopes of Cell 1 is similar, except that the geomembrane portion of the GCL is textured, and a geocomposite (a geonet sandwiched between and bonded to two nonwoven geotextiles) is used in place of the individual geonet and geotextile. An 18 in. thick layer of sand was placed on top of the uppermost geotextile as a protective material on both the floor and the internal side slopes.

**Cell 1 Filling Sequence**

Cell 1, 10.75 acres, was split into 2 sub-cells, Cell 1A and Cell 1B. Cell 1A, constituting the northern portion of Cell 1, is approximately 7.25 acres in size, and the southern Cell 1B is approximately 3.5 acres. Cell 1A was constructed first and started receiving MSW on March 1, 1996. Cell 1B was subsequently completed and began receiving waste relocated from the old landfill on June 4, 1996.

It is important to note that the physical characteristics of the "relocated waste" differed substantially from those of typical MSW. The relocated waste consisted of up to 80 % soil making it much heavier than typical MSW, and significant quantities were saturated, having been excavated from saturated zones in the old landfill. The majority of the waste placed in Cell 1B was relocated waste.

By the end of September 1996, Cell 1 was nearing capacity, and Cell 2 was not completed. It is estimated that during this time, the western slope of the relocated waste in Cell 1B achieved a grade of 40 % (2.5H:1V). Figure 3, provided by the owner/operator, shows the estimated contours of Cell 1 at the time of failure. The surface elevations are based on aerial and field surveys taken in July and September of 1996, and site personnel comments.

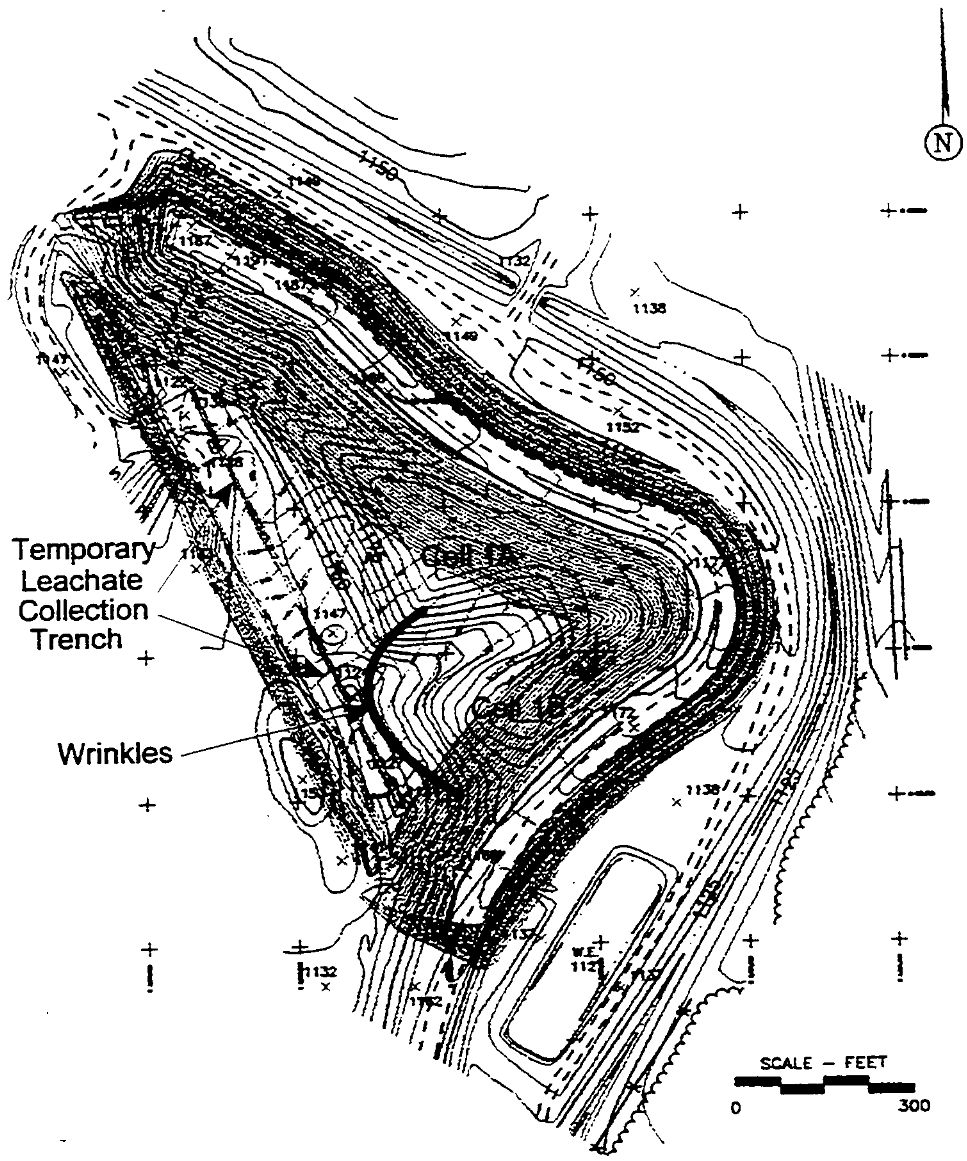


Figure 3. Estimated surface elevations in Cell 1 at time of failure.

## Discovery of Failure

On August 5, 1996, cracks were noticed on southern and western slopes of the relocated waste in Cell 1B; the cracks were 1 to 3 in. wide and 10 to 40 ft. long (see Figure 4). The cracks were monitored using rudimentary methods and did not appear to lengthen, widen, or exhibit differential settlement. At approximately the same time, the protective sand material along a portion of the toe of the relocated waste slope was discovered to have heaved upward slightly along the axis of the temporary leachate collection trench.

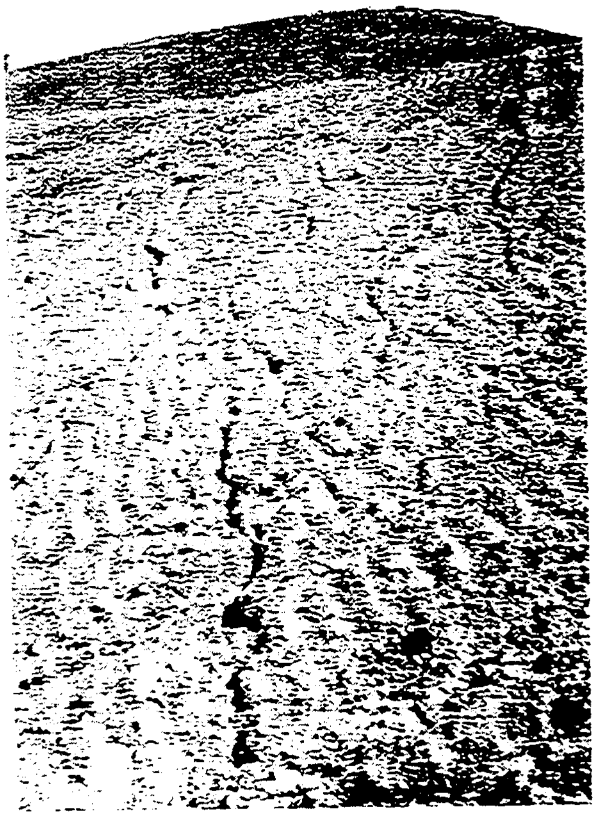

Figure 4. Cracks on western slope of Cell 1B.

Certification of the first portion of Cell 2 (Cell 2A) was obtained from Ohio EPA on August 7, 1996. With the cell containing the landfill sump now certified, the process of disengaging and sealing off the temporary leachate collection trench and sump could begin. This entailed excavating the layer of protective sand material from above the trench, draining the fluids, and welding a 60 mil HDPE geomembrane over the pervious drainage media which was left in the trench.

On September 14, 1996, while excavating the protective sand layer, accordion-like wrinkles in the geotextile, geonet, and geomembrane were discovered along the toe of the waste slope where the sand had heaved upward (see Figure 5). It was first surmised by the owner/operator that the excessive material was caused by elongation of the geosynthetics. However, samples of the GCL removed from under the waste revealed striations between the bentonite and the geomembrane and between the bentonite and compacted clay. The striations confirmed that shear displacement had occurred and that hydrated bentonite was involved. The bentonite was probably hydrated by attracting moisture from the underlying compacted clay liner. By measuring the folds of the geosynthetics, it was estimated that the toe of slope had translated laterally as much as 6 ft.

Figure 5. Wrinkles in geosynthetics at toe of slope in Cell 1B.

**Failure Investigation**

The initial investigation involved exposing the full extent of the wrinkling. After waste removal, it was readily observable that the wrinkles in the geosynthetics formed an arc as shown in Figure 3. The next step involved removing the waste from "inside" of the arc. This required that a sizable portion of the relocated waste in Cell 1B had to be moved into the newly certified Cell 2. The final phase of the investigation involved analyzing samples of the GCL and observing the compacted clay liner for damage. Waste continued to be excavated as long as damage was encountered in the composite liner system. In all, an estimated 300,000 cubic yards of waste were removed from Cell 1B.

After waste removal, several large tears (1 to 6 ft.) and numerous small tears (less than 1 ft.) were found in the geomembrane portion of the GCL, and two ruptured geomembrane seams were discovered. The bentonite portion of the GCL also fared poorly. Bentonite had randomly stayed affixed to the geomembrane or adhered to the compacted clay liner and in other areas had oozed up into wrinkles or flowed away from stress concentrations. This resulted in an inconsistent layer of bentonite in the affected area. Figures 6, 7, and 8 illustrate some of the damage to the composite liner system.

Figure 6. Damage to the geomembrane portion of the GCL as a result of the failure.

Figure 7. Damage to the bentonite portion of GCL as a result of the failure.

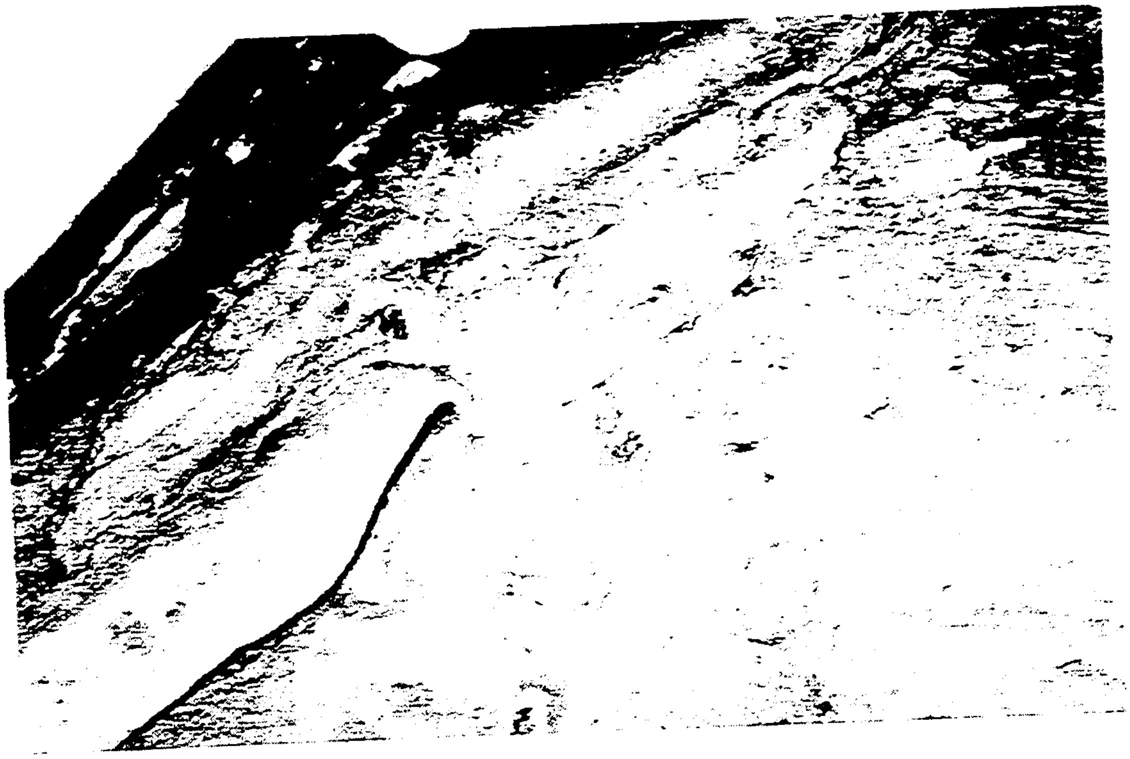

Figure 8. Migration of hydrated bentonite in wrinkles of geomembrane.

## Conclusions

The precise failure mechanism is still under investigation and is the subject of current research. However, the significant damage observed in the upper components of the composite liner system suggest that the failure was translatory in nature and involved lateral movement of a large mass primarily along the bentonite portion of the GCL. It also appears evident that the slope inclination, slope height, and physical characteristics of the "relocated waste" in Cell 1B played a major role in mobilizing the slide block.

The remedy consisted of welding another layer of the Gundseal® over the damaged area. However, in many instances the folds of the damaged GCL had to be flattened to obtain intimate contact between the components of the composite liner system. This was accomplished by cutting the folds open and laying the existing GCL flat onto the compacted clay liner.

It is extremely important to note that the failure at this facility could have gone undetected. Had it not been for the owner/operator disengaging the temporary leachate collection system, the failure may not have been discovered. The cracks on the slope face and the slightly heaved sand at the slope toe probably would not have been sufficient to conclude that a failure had occurred nor have warranted further investigation.

## GCL DESIGN ISSUES

As a result of this failure and uncertainties over the ability of GCLs to replace compacted soils in all instances of waste containment, the Ohio Environmental Protection Agency issued an advisory on structural considerations for incorporating GCLs in waste containment design (Evans, 1997). The remainder of the paper discusses portions of the advisory.

There are two main areas of concern with incorporating a GCL in a landfill:

- Identifying performance standards that can account for uncertainties associated with the use of a product that does not have a proven long-term performance record; and

- Determining accurate and appropriate design parameters to account for the weak nature of hydrated bentonite.

These two main areas of concern have a number of specific issues that are discussed subsequently.

### Long-Term Performance

The first use of a GCL in a waste containment facility occurred in 1986. Since that time, GCLs have gained in popularity, but have been surrounded by technical uncertainties resulting in modification of existing products. Little is known about the long-term performance of GCLs. This issue is discussed at length in U.S. EPA's *Report of 1995 Workshop on Geosynthetic Clay Liners*, dated June 1996, and also in the American Society for Testing and Materials (ASTM) Special Testing Publication No. 1308, *Testing and Acceptance Criteria for Geosynthetic Clay Liners*, (ASTM, 1997). Additionally, there appears to be a growing opinion among researchers in the GCL arena that it may be prudent to utilize a post-peak instead of peak shear strength for slope design. This is due to uncertainties surrounding the processes that may initiate deformations in composite lining systems during construction, waste placement, and subsequent waste consolidation (Stark and Poeppel, 1994). These processes may result in the development of a post-peak or residual shear strength condition that is significantly smaller than the peak strength value. Therefore, due to a lack of long-term performance data, uncertainties in the processes that might initiate deformations, and the long-term durability of needle-punched or stitch-bonded fibers, designing for post-peak conditions is recommended in the advisory (Evans, 1997).

**Shear Strength of Hydrated Bentonite**

The bentonite component of the GCL usually controls the shear strength characteristics of the composite bottom liner and cap system. Hydrated bentonite exhibits one of the lowest, if not the lowest, peak and residual shear strengths of any soil (Mesri and Olson, 1970). Bentonitic soils also have a high affinity for moisture and will attract significant amounts of moisture from even the driest subgrade. As a result, it should be assumed that GCLs will undergo widespread hydration unless they are encapsulated by two geomembranes.

Some designers have suggested encapsulating the GCL between two geomembranes to reduce the extent of hydration. While this will minimize widespread hydration, localized zones of hydrated bentonite and the ensuing weakened conditions are still a possibility because of potential imperfections in geomembrane installation. A U.S. EPA sponsored test section of an encapsulated GCL recently failed due to such localized zones of hydration (Koerner et al., 1996a).

GCL manufacturers generally supply typical shear strength data for their products. While this data may be useful in preliminary design evaluations, it is not recommended for slope stability calculations. This data is usually accompanied by disclaimers which state that the information should not be relied upon to determine final design parameters and that project-specific shear testing should be conducted for this purpose. The authors also recommend that project-specific materials should be tested under appropriate conditions, including normal stress, moisture content, and shearing procedure.

Currently, no established or otherwise universally accepted test method exists for determining the internal and interface shear strength of GCLs. "Appropriate" shear testing has proven to be a highly subjective and controversial issue. This is to be expected when one considers the array of products, each with distinctively different characteristics, and the reality that any inaccuracies inadvertently introduced into sample selection, sample preparation, or actual shearing can alter the measured shear resistance.

Outlined below are some of the more pertinent aspects of shear testing a GCL that should be considered.

A. Sample Selection

Ideally the shear specimens should be selected from rolls of GCL that are delivered to the site. However, this is often impractical. The next best alternative is to obtain identical product samples from another site. If either of the preceding options are unavailable, samples from the manufacturer may be used, if the manufacturer will certify that the samples are representative of materials that will be shipped to the field. This is important because the amount of needle-punching reinforcement can vary significantly in the manufacturing process.

B. Hydration

According to U.S. EPA (1996), GCLs will hydrate when placed in contact with typical construction subgrade soils and will probably hydrate significantly within the first few days. U.S. EPA (1996) reports that moisture contents as high as 50 % have been measured after 10 days. Stark (1997) reports that this hydration typically occurs under a free swell condition and that the swell pressure of a reinforced GCL can be on the order of 35 to 40 kPa (730 - 835 psf). A confining stress of this magnitude, equivalent 2.1 to 2.5 m (7 - 8 ft) of soil, is typically not applied to a cap system, and it is usually a number of weeks, if not months, before a confining stress capable of preventing GCL swell is applied to the composite bottom liner system. This swell pressure is capable of weakening the reinforcement of GCLs and/or forcing hydrated bentonite into the interfaces, thereby greatly decreasing the integrity of the bottom liner or cap system. Consequently, it is recommended that project-specific GCLs and adjacent materials be allowed to fully hydrate, as a single unit, in a free swell condition until vertical expansion has essentially ceased. The vertical expansion can be determined by monitoring vertical displacement until swelling has reached the end of primary swell as determined by ASTM 4546. The moisture content of the shear specimens should be verified after the shear test to verify the degree of hydration.

C. Normal Stress

Project-specific materials, including soils and geosynthetics, should be tested over the entire range of normal stresses that will be encountered in the particular design because the shear strength values are usually stress dependent.

- For cap systems, this includes the low normal stresses associated with these applications and any additional stresses that may be induced by surface water diversion benches, roads, equipment, or other structures constructed above the composite cap system.

- For composite bottom liner systems, the range of normal stresses that need to be evaluated can be extensive because of low normal stresses at the perimeter of the fill and high values under the deepest areas of the fill.

D. Shear Displacement Rate

Gilbert et al. (1997), and Stark and Eid (1997) show that the rate of shear displacement can affect the measured shear strength of GCLs. Shear strength values from tests using a displacement rate of 1 mm/min, the ASTM D-5321 recommended value, have been shown to be greater than those values using slower displacement rates. Stark and Eid (1997) report that rates equal to or less than 0.04 mm/min (.0016 in/min) do not seem to have a large impact on measured shear strength values of one reinforced GCL. Gilbert et al. (1997) and U.S. EPA (1996) recommend ASTM D-3080 for determining the appropriate direct shear rate.

E. Test Method

Currently the most common method used for determining internal shear strengths and interface shear strengths of GCLs is ASTM D-5321, which utilizes a 300 mm square shear box. However, other techniques are being utilized for GCL testing, such as the torsional ring shear device (Stark and Eid 1997) and a pullout device (Fox et. al., 1997a).

**Avoiding GCL Thinning**

After GCLs have hydrated and stresses have been applied, the bentonite has been observed to migrate away from high stress concentrations, resulting in localized thinning of the GCL. This phenomenon is especially likely to occur in areas of a composite bottom lining system, where non-uniform stress concentrations typically develop. This includes areas in the immediate proximity of wrinkles, in and around sumps, and beneath leachate collection piping.

One-dimensional compression tests show that the thickness of a hydrated GCL can decrease significantly due to bentonite migration. This phenomenon has been noted by numerous authors including Fox et al. (1997b), Richardson (1997), Anderson (1996), Koerner and Narejo (1995), and Anderson and Allen (1995). According to Fox et al. (1997b), bentonite migration seems to be more pronounced in unreinforced GCLs than in reinforced GCLs. Anderson and Allen (1995), and Anderson (1996) also show that the thickness of a GCL can be significantly reduced in the vicinity of a wrinkle in the overlying geomembrane due to hydrated bentonite flowing up into the air space of the wrinkle. Geomembrane wrinkles may change shape but do not necessarily disappear according to Koerner (1996b).

Thinning of the GCL has serious implications for meeting the regulatory requirements, which often include criteria for specific mass of bentonite per unit area, as well as hydraulic performance. GCLs are allowed to replace all or a portion of the compacted clay liner based on their manufactured hydraulic performance. However, the hydraulic performance or fluid flux through a GCL is directly related to the thickness or specific mass of bentonite per unit area. Thus, if the bentonite thins, the fluid flux through the GCL will increase, and the requirements for hydraulic performance and specific mass of bentonite per unit area may no longer be satisfied. It is therefore recommended that the sump areas and areas directly beneath leachate collection piping not incorporate GCLs without taking additional precautions and that wrinkling of the geomembrane be kept to an absolute minimum. Some additional measures which have been proposed include multiple GCL layers or encapsulating the GCL between two geomembranes at these locations. Unfortunately, substantial information on these configurations is lacking and more research needs to be conducted.

## Concerns and Recommendations Unique to Unreinforced GCLs

Unreinforced GCLs lack any added reinforcement to resist shear stresses. As a result, these products have internal shear strength and bearing capacity characteristics approximately equivalent to hydrated bentonite. U.S. EPA (1996) presents shear strength data on unreinforced GCLs that show friction angles of about 10 degrees. Richardson (1997) estimates the bearing capacity of a hydrated unreinforced GCL to be 40 kPa (825 psf) and the internal shear strength to be less than 5 kPa (100 psf) for low normal stresses.

For low normal stresses such as those in cap systems, unreinforced GCLs will hydrate fully under confining stresses significantly less than the swell pressure of the GCL. These products may also undergo significant creep due to the time-dependent deformational characteristics of hydrated bentonite, resulting in mobilization of a post-peak or residual strength condition. Additionally, the extremely low bearing capacity of unreinforced GCLs may result in thinning of the GCL from bentonite migration due to non-uniform stress concentrations, such as wheel loads, that may be applied to a cap during closure and post closure. For these reasons, it is recommended that composite cap system designs do not incorporate unreinforced GCLs and that unreinforced GCLs be restricted to use on bottom lining slopes of less than 10%.

## Summary

In summary, significant concerns remain regarding the ability of GCLs to perform as safely and durably as compacted soils in some applications. The concerns are due to a lack of long-term performance data on these products and the inherently low shear strength of bentonitic soils. The use of post-peak shear strength conditions can account for many of the long-term performance uncertainties, and the shearing procedures presented herein can help quantify the low shear strength hydrated bentonite.

The contents and views in this paper are the writers' and do not necessarily reflect those of MLI or the Ohio Environmental Protection Agency.

# References

Anderson, J. D., (1996), "Are Geosynthetic Clay Liners Really Equivalent to Compacted Clay Liners," *Geotechnical News*, BiTech Publishing, Ltd., Richmond, British Columbia, Canada, Vol. 14, No. 2, June, pp. 20-23.

Anderson, J. D., S. R. Allen, (1995), "What Are The Real Considerations When Using a Geosynthetic Clay Liner," *Proceedings of the 9th Annual Municipal Solid Waste Management Conference*, Austin, TX.

ASTM (1997), "Testing and Acceptance Criteria for Geosynthetic Clay Liners, American Society for Testing and Materials, Special Test Publication No.1308," American Society for Testing and Materials, Well, L. W., Editor, 268 p.

Evans, W. D. (1997), "Advisory on Structural Integrity Considerations for Incorporating Geosynthetic Clay Liners in Solid Waste Landfill Design," Division of Solid & Infectious Waste Management, Ohio Environmental Protection Agency, Columbus, OH, September, 12 p.

Fox, P. J., M. G. Rowland, J. R. Scheithe, K. L. Davis, M. R. Supple, C. C. Crow (1997a), "Design and Evaluation of a Large Direct Shear Machine for Geosynthetic Clay Liners," *Geotechnical Testing Journal*, American Society for Testing and Materials, Vol. 20, No. 3, September, pp. 279-288.

Fox, P. J., D. J. Battista, S.-H. Chen (1997b), "A study of the CBR bearing capacity Test for Hydrated Geosynthetic Clay Liners," *Testing and Acceptance Criteria for Geosynthetic Clay Liners, American Society for Testing and Materials, Special Test Publication No.1308*, Larry W. Well, Ed., American Society for Testing and Materials, pp. 251-264.

Gilbert, R. B., H. B. Scranton, D. E. Daniel (1997), "Shear Strength Testing for Geosynthetic Clay Liners," *Testing and Acceptance Criteria for Geosynthetic Clay Liners, American Society for Testing and Materials, Special Test Publication No.1308*, Larry W. Well, Ed., American Society for Testing and Materials, pp. 121-135.

Ohio EPA (1997), "1997 Ohio Solid Waste Facility Data Report," Ohio Environmental Protection Agency, Division of Solid & Infectious Waste Management, 127 p.

Koerner, R. M. and D. Narejo (1995), "Bearing Capacity of Hydrated Geosynthetic Clay Liners," *Journal of Geotechnical Engineering*, American Society of Civil Engineers, Vol. 121, No. 1, pp.82-85.

Koerner, R. M., D. E. Daniel, R. Bonaparte (1996a), "Current status of the Cincinnati GCL Test Plots," *Proceedings of the 10th GRI Conference on Field Performance of Geosynthetics and Geosynthetic related Systems*, Drexel University, Philadelphia, PA, pp. 147-175.

Koerner, R. M. (1996b), "The GRI Newsletter/Report," Geosynthetic Research Institute, Drexel University, Philadelphia, PA, Vol. 10, No. 2, June.

Mesri, G., R. E. Olson (1970), "Shear Strength of Montmorillonite," *Geotechnique*, Vol. 2, No. 3, pp. 261-270.

Richardson, G. N. (1997), "GCL Internal Shear Strength Requirements," *Geosynthetics Fabrics Report*, March, pp. 20-25.

Stark, T.D., A.R. Poeppel, (1994), "Landfill Liner Interface Strengths from Torsional Ring Shear Tests," *Journal of Geotechnical Engineering*, ASCE, Vol. 120, No. 3, March, pp. 597-615.

Stark, T. D. (1997), "Effect of Swell Pressure on GCL Cover Stability," *Testing and Acceptance Criteria for Geosynthetic Clay Liners, American Society for Testing and Materials, Special Test Publication No. 1308*, Larry W. Well, Ed., American Society for Testing and Materials, pp. 30-44.

Stark, T. D., H. T. Eid (1997), "Shear Behavior of Reinforced Geosynthetic Clay Liners," *Geosynthetics International*, Vol.3, No. 6, pp. 771-786.

U. S. EPA (1996), "Report of 1995 Workshop on Geosynthetic Clay Liners," United States Environmental Protection Agency, Cincinnati, Ohio, 96 p.