Technical Paper by T.D. Stark, D. Arellano, W.D. Evans, V.L. Wilson and J.M. Gonda

UNREINFORCED GEOSYNTHETIC CLAY LINER CASE HISTORY

ABSTRACT: This paper describes a slope failure involving an unreinforced geosynthetic clay liner (GCL) in a municipal solid waste (MSW) landfill liner system. The precise mechanism for the shear movement of the interim slope is not known. However, the significant damage observed in the upper components of the composite liner system suggests that the failure was translatory primarily along the interface between the recompacted soil liner and the overlying hydrated bentonite of the geomembrane-backed GCL. It also appears evident that the slope inclination, slope height, physical characteristics, e.g. high unit weight, of the waste, and possibly the overlying smooth geomembrane/geonet interface played a significant role in the movement. Design recommendations for interface strengths and stability analyses are also presented.

KEYWORDS: Geosynthetic clay liner, Waste containment, Strength, Stability, Shearbox.

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, E-mail: t-stark1@uiuc.edu; D. Arellano, Graduate Research Assistant, 3212 Newmark Civil Engineering Laboratory, University of Illinois, 205 N. Mathews Ave., Urbana, Illinois 61801, USA, Telephone: 1/217-333-6973, Telefax: 1/217-333-9464, E-mail: darellan@students.uiuc.edu; W.D. Evans, Environmental Engineer, Ohio Environmental Protection Agency, Division of Solid & Infectious Waste Management, Central Office, P.O. Box 1049, Columbus, Ohio 43216-1049, USA, Telephone: 1/614-728-5371, Telefax: 1/614-728-5315, E-mail: doug.evans@epa.state.oh.us; V.L. Wilson, Environmental Engineer, Ohio Environmental Protection Agency, Division of Solid & Infectious Waste, Northeast District Office, 2110 E. Aurora Rd., Twinsburg, Ohio 44087, USA, Telephone: 1/330-963-1180, Telefax: 1/330-487-0769, E-mail: virginia.wilson@epa.state.oh.us; and J.M. Gonda, Project Engineer, Earth Sciences Consultants, Inc., 190 N. Union, Suite 200, Akron, Ohio 44304, USA, Telephone: 1/330-535-6966, Telefax: 1/330-535-8112.

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1 INTRODUCTION

Geosynthetic clay liners (GCLs) have been used in recent years to replace or reduce the thickness of recompacted soil liners (RSLs) required in composite liner or cover systems for waste containment facilities. GCL products are in continuous development to improve their hydraulic performance, bearing capacity (Stark 1998), and shear strength characteristics. The two main types of unreinforced GCLs consist of a thin layer (approximately 3 to 5 mm thick) of granulated bentonite either adhered to a high density polyethylene (HDPE) geomembrane or encased between two geotextiles. Reinforced GCLs are also available and consist of geotextile-encased bentonite reinforced by connecting the backing geotextiles using stitch-bonding or needle-punching. The reinforcement is designed to increase the internal shear strength of the GCL.

The current paper describes a case history involving a geomembrane-backed GCL in a municipal solid waste (MSW) landfill liner system. Extensive laboratory testing and stability analyses were conducted to explain the slope failure that occurred mainly along the hydrated bentonite/RSL interface. Finally, recommendations for analyzing and using unreinforced GCLs in landfill liner and cover systems are presented.

2 CASE HISTORY

2.1 Overview

Mahoning Landfill, Inc. (MLI) is located on an 809,400 m^2 parcel of land near Youngstown, Ohio, USA. In 1962 strip-mining operations were 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 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 until 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 1 March 1995, MLI agreed to relocate all of the existing waste on the property (an estimated 1.3 million cubic meters) into the new fully lined facility. In 1996, MLI received 124 million kilograms of new MSW according to the 1997 Ohio Solid Waste Facility Data Report (Ohio EPA 1997).

2.2 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 that was to be utilized until the main leachate collection sump became operational in Cell 2. The temporary leachate collection system was simply a depression of the composite liner system along the boundary of Cells 1 and 2. Figure 2 shows the excavation for the temporary leachate trench after placement of the geomembrane-

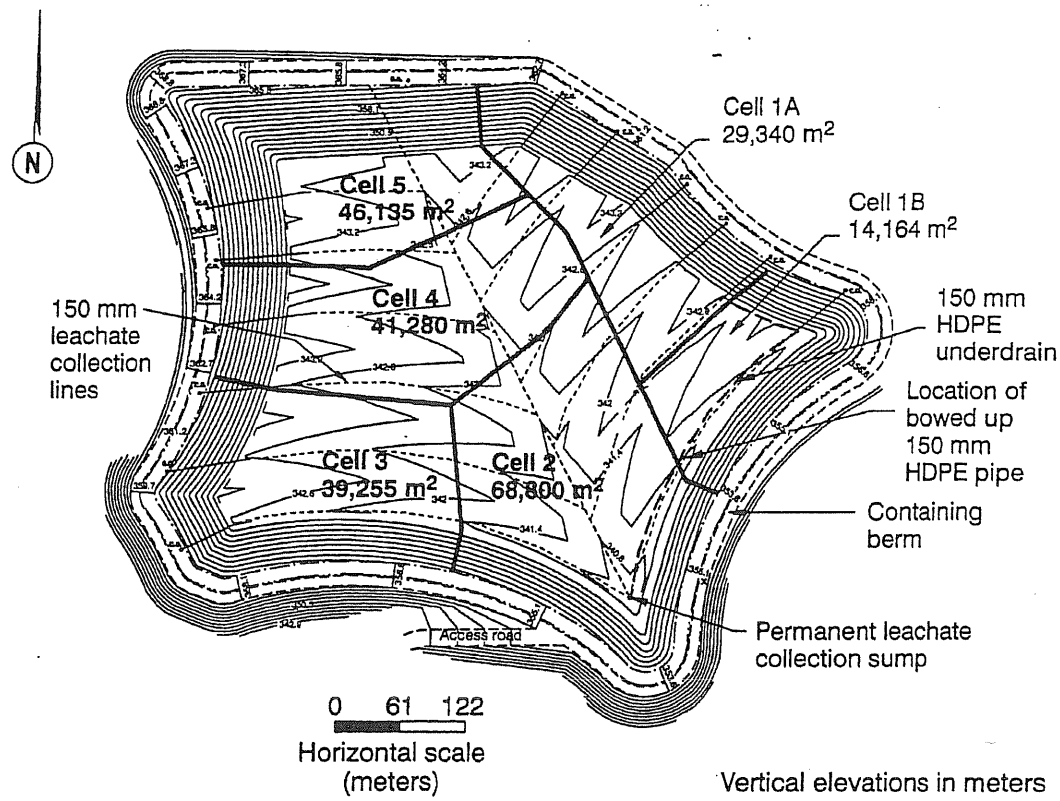

Figure 1. Plan view and cell layout.

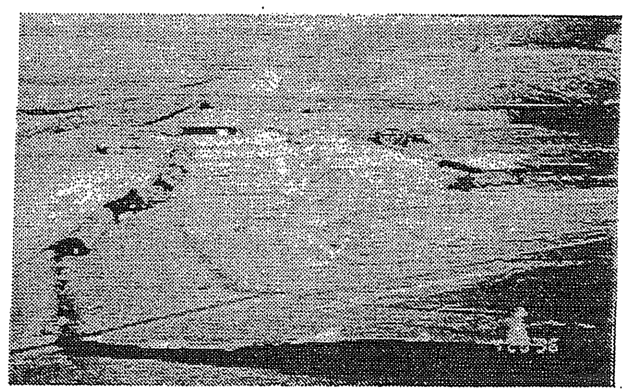

Figure 2. Temporary leachate collection system along the western edge of Cell 1B.

backed GCL. This temporary system was sloped from north to south (top to bottom in Figure 2) with a pump at the south or lowest end. The temporary leachate collection trench varied in depth and width because of the herringbone contour design of the cell floor that it bisected. The trench maintained a minimum 0.9 m thick recompacted soil liner and was lined with a geomembrane-backed GCL (Figure 3). It also maintained a minimum 0.45 m depression in the floor so that a nominal volume of leachate could be

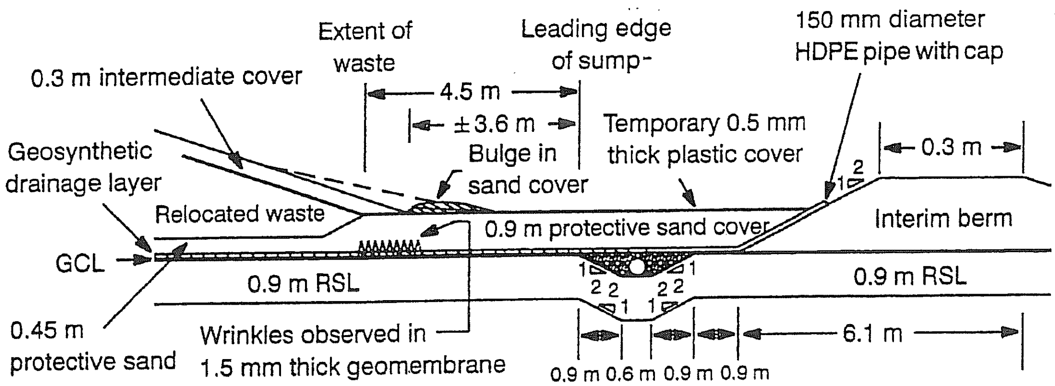

Figure 3. **Cell 1 interim berm and temporary leachate collection trench.**

collected and pumped. To accomplish this, a perforated pipe and sump pump were placed in the trench and then the entire trench was backfilled with gravel. The temporary leachate collection system would be disengaged when Cell 2 and the permanent leachate collection trench became operational.

The composite bottom liner system for the floor of Cell 1 consists of the following from bottom to top: a 0.9 m thick layer of recompacted soil liner, a 1.5 mm (60 mil) thick smooth geomembrane-backed GCL, an HDPE biaxial geonet, a 339 g/m^2 nonwoven geotextile, and a 0.45 m thick sand protection layer. (The geotextile was not heat-bonded to the geonet.) The geomembrane-backed GCL was installed with the bentonite in direct contact with the recompacted soil liner. It is important to note that the recompacted soil liner was compacted from 0 to 2% wet of optimum, based on the standard Proctor test method (ASTM D 698), at an average water content of 11.5% (Earth Sciences Consultants, Inc. 1993). The composite bottom liner system on the 33% (1V:3H) side slopes of Cell 1 is similar, except that the geomembrane portion of the GCL is textured and a drainage geocomposite (a geonet sandwiched between and heat-bonded to two nonwoven geotextiles) was used in place of the individual geonet and geotextile. A 0.45 m thick layer of sand was placed on top of the uppermost geotextile as a protective material on both the floor and side slopes.

2.3 Cell 1 Filling Sequence

Because of time and weather constraints, Cell 1 was divided into two parts: Cell 1A, constituting the northern portion of Cell 1 is approximately 29,340 m^2 in size; and the southern section, Cell 1B, is approximately 14,164 m^2 (Figure 1). Cell 1A was constructed first and started receiving new MSW on 1 March 1996. Cell 1B was subsequently completed and began receiving waste relocated from the old landfill on 4 June 1996. To gain access to Cell 1B and not interfere with normal landfill operations, the waste relocation contractor constructed a haul road that entered Cell 1B at the western edge of the junction of Cell 1A and Cell 1B (Figure 4). This haul road went into Cell 1B from Cell 2 at an approximate slope of 10% and was built on relocated waste. Waste was relocated to Cell 1B at a rate of 11,460 to 38,200 m^3 per week.

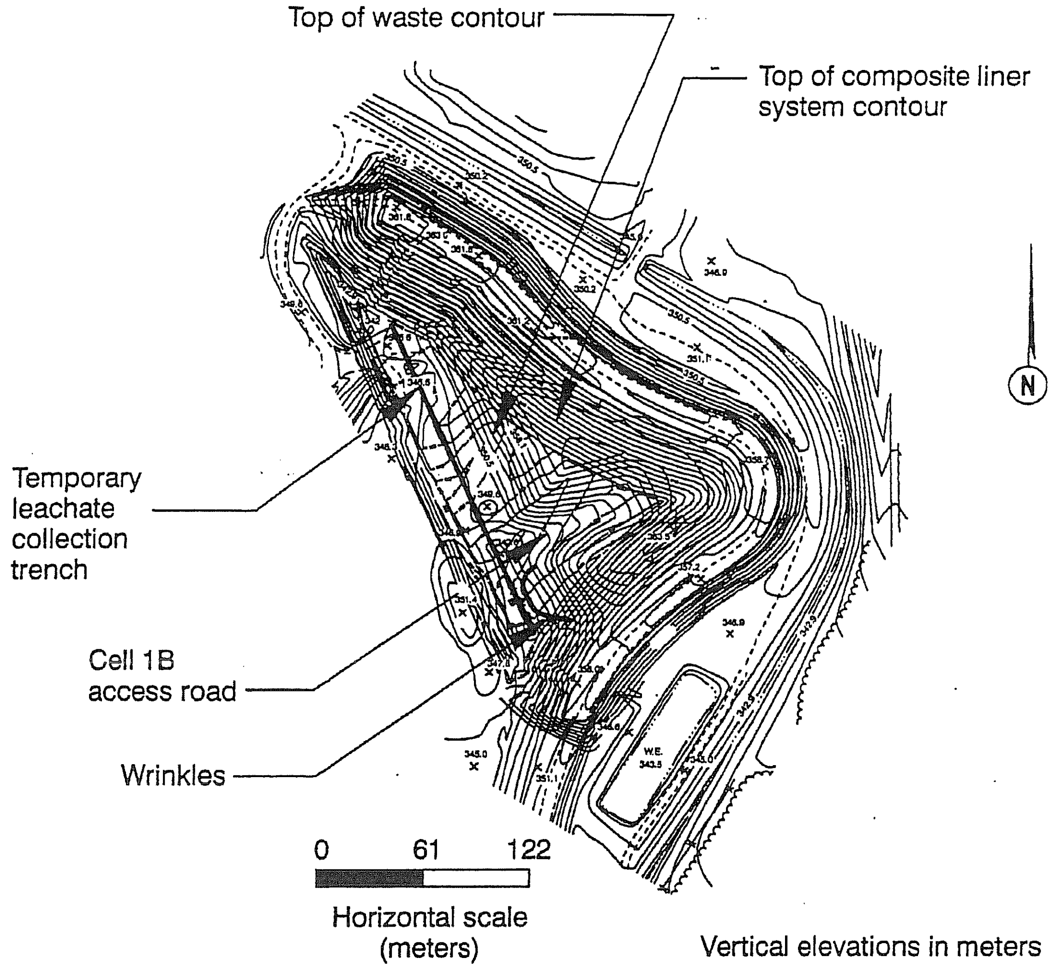

Figure 4. Access road and estimated surface elevations in Cell 1 at time of failure.

It is important to note that the physical characteristics of the "relocated waste" differed from those of the new MSW that was being placed in Cell 1A. The relocated waste consisted of up to 90% soil making it heavier than typical MSW, and significant quantities were saturated having been excavated from saturated zones in the old landfill. The average total unit weight of the relocated waste was estimated to be 18.5 kN/m^3 based on standard Proctor tests (ASTM D 698) on representative samples (Earth Sciences Consultants, Inc. 1997). The total unit weight of the new MSW was assumed to be 11.8 kN/m^3 based on Kavazanjian et al. (1995). Analyses described subsequently will show that this difference in unit weight was a contributing factor in the movement observed in Cell 1B. Of the volume of relocated waste placed in Cell 1B, the first half was relatively dry compared to the second half, which was extremely wet. This was caused by the first half of the relocated waste coming from higher elevations within the old strip-mine cut.

By 28 July 1996, the relocation of waste into Cell 1B was complete. It is estimated that during this time the western slope of the relocated waste in Cell 1B achieved a grade of approximately 40% (1V:2.5H). This steep slope was caused by the limited space available for waste storage in Cell 1B and the large quantity of waste that had to be relocated to allow for the construction of Cell 2. Figure 4 shows the estimated surface contours of Cell 1 at the time of failure (Earth Sciences Consultants, Inc. 1997). The surface elevations are based on aerial surveys taken in July and September of 1996 and then adjusted according to a field survey and site personnel observations to replicate the configuration/geometry in August 1996. The elevations for the top of the composite liner system are also depicted in Figure 4.

2.4 Discovery of Failure

On 5 August 1996, cracks in the interim soil cover were observed near the crest of the slope (see cross section A-A′ in Figure 5) and near the horizontal limit of waste along the southwestern side of Cell 1B (Figure 6). Figure 6 shows the location of cross section A-A′ in Figure 5. The observed cracks were 25 to 75 mm wide and 3 to 12 m long. The cracks exhibited no vertical offset and were characteristic of a horizontal separation. On 5 and 6 August 1996, waste was quickly pushed from the top and face of the waste slope into the void or valley where the haul road existed. This reduced the steepness of the slope and the overall height of the waste. The cracks were monitored using rudimentary methods and did not appear to lengthen, widen, or exhibit differential settlement after the slope was unloaded. It will be shown subsequently that the residual strength of the relevant geosynthetic interface is less than the peak values, which could have resulted in a larger failure due to the process of progressive failure. It is thought that a larger failure did not occur because of the rapid and significant unloading of the slope and the presence of the protective sand layer and the interim berm between Cells 1B and 2 (Figure 3) that provided some passive resistance.

At approximately the same time that cracks were observed in the cover soil, the protective sand material along a portion of the relocated waste slope toe was discovered to have slightly bulged upward along the toe of the relocated waste slope (Figure 6). The upward heave of 0.2 to 0.4 m was not immediately noticed because a 0.5 mm thick sheet of plastic material (Figure 3) covered the protective sand. The plastic sheet, which is also used as alternate daily cover at the site, was wrinkled due to temperature effects and exposure to the sun and, thus, the slight upheaval was not detected until the slope was carefully inspected by landfill personnel. Because the slope cracks did not change after 5 and 6 August, the only action taken at this time was to reduce the slope inclina-

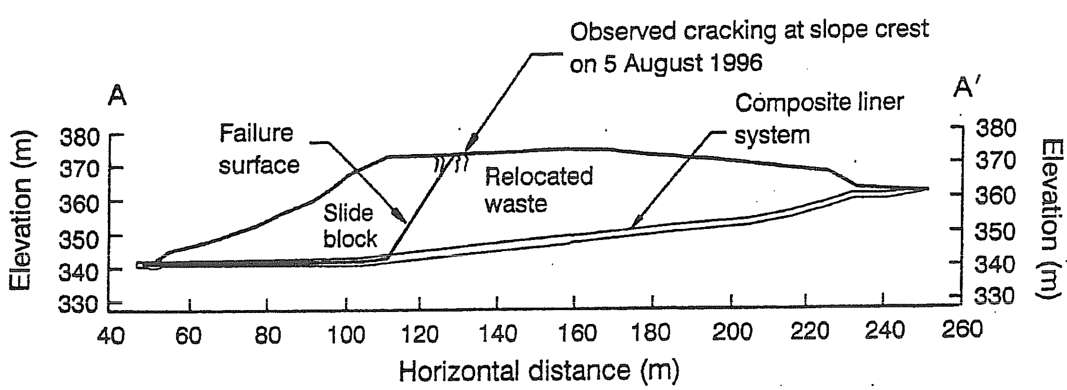

Figure 5. Cross section A-A′ in Cell 1B.

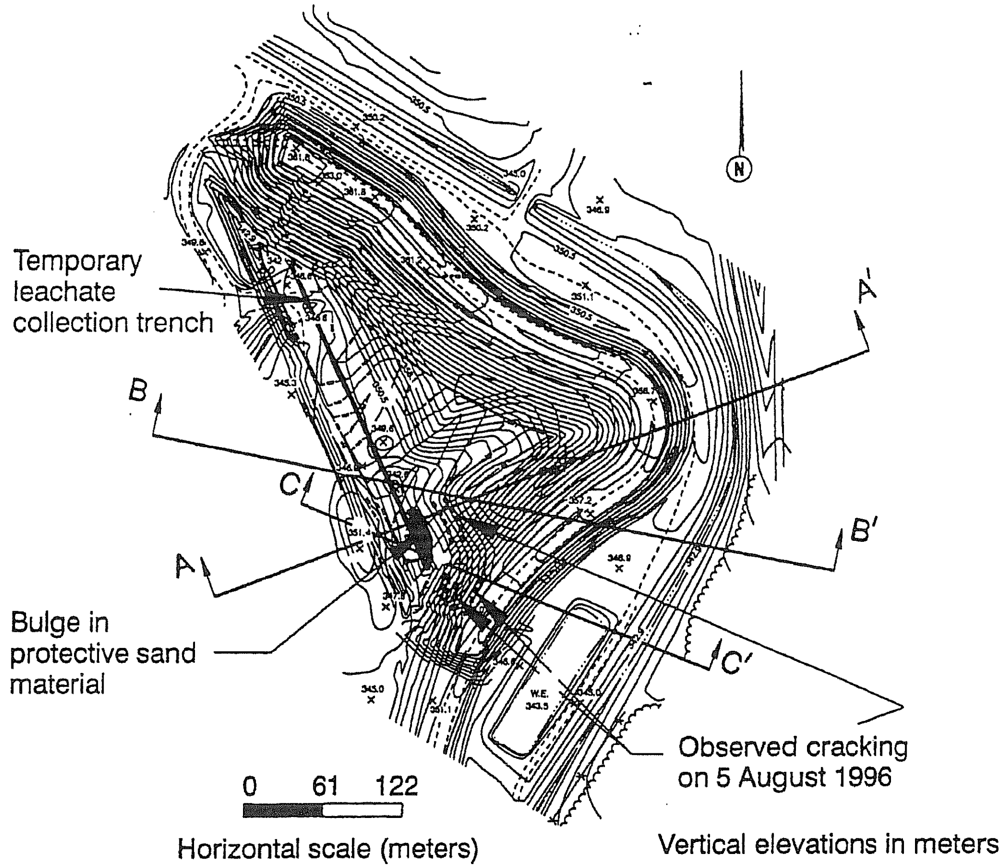

Figure 6. Location of slope stability cross sections.

tion to 1V:3H by filling the void for the haul road. This was the only remedial option available because Cell 2 was not certified.

Certification of the first portion of Cell 2 was obtained from the Ohio Environmental Protection Agency (OEPA) on 7 August 1996. With the cell containing the landfill sump now certified, the process of disengaging and sealing the temporary leachate collection trench and sump between Cells 1 and 2 could begin. This entailed excavating the layer of protective sand material from above the trench, draining the fluids, pulling back the geonet and geotextile, and welding a 1.5 mm (60 mil) HDPE geomembrane over the pervious drainage medium (No. 57 AASHTO stone), which was to be left in the trench. Once the HDPE geomembrane was welded in-place, the geonet, geotextile, and protective sand cover material would be replaced. Closure of the temporary leachate collection trench proceeded from Cell 1A towards Cell 1B. On 14 September 1996, near the junction of Cells 1A and 1B accordion-like wrinkles in the geotextile, geonet, and geomembrane were discovered along the toe of the relocated waste slope where the sand had heaved upward (Figure 4). Figure 7 presents a photograph of some of the wrinkles in the geosynthetics near the northern end of Cell 1B. Of course, some of the wrinkling could have been caused by temperature effects after installation. However, it can be seen from Figure 2 that the geomembrane did not exhibit this magnitude of wrinkling

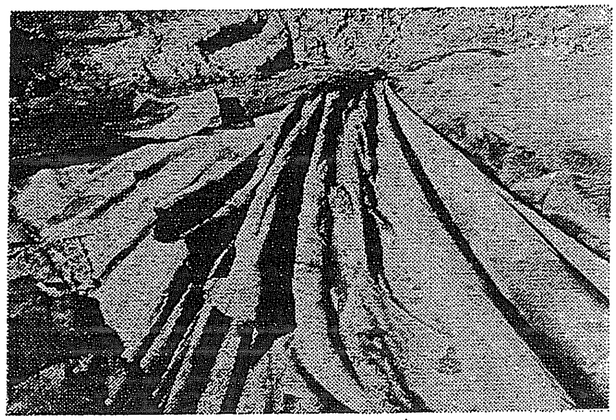

Figure 7. Wrinkles in the geomembrane at the toe of the slope in Cell 1B.
Note: The width of the bottom edge of the photograph is approximately 4 m.

shortly after installation. The size of the wrinkles suggested that other mechanism(s) must have contributed. The wrinkles formed an arc shape (Figure 4) and were parallel to the toe of the waste slope along the western edge of Cell 1B. The wrinkles then were found to be parallel to the Cell 1A/Cell 1B boundary. The exposed wrinkles were approximately 200 to 300 mm high and covered an area approximately 45 m long and 1.2 m wide. It was estimated that approximately 3 to 4 m of geomembrane was wrinkled into the 1.2 m wide strip. After discovering the wrinkles, the temporary collection trench was closed and the protective sand was placed on the geosynthetics.

On 16 September 1996, 6 m of the temporary leachate collection trench at the southern end of Cell 1B was exposed to determine if wrinkles were present. This resulted in exposure of the leachate feeder pipe along the southern end of Cell 1B that is connected to the temporary leachate collection trench (Figure 1). Shortly after exposing the pipe, the 150 mm diameter HDPE pipe started to slowly bow upwards. This indicated that the slide mass was possibly moving toward the temporary leachate collection trench. The bowed pipe was removed, the temporary leachate collection trench and sump were covered with the protective sand, and the slope was quickly reduced to 1V:4H by pushing waste from Cell 1B into Cell 2. It is thought that a larger slope failure did not occur, even though the geosynthetic interfaces exhibited a post-peak strength loss, because the temporary leachate collection trench was quickly filled with sand to mobilize the passive resistance of the interim berm and the slope was rapidly unloaded.

2.5 Bentonite Hydration

Samples of the GCL, removed from under the waste near the toe of the relocated waste slope, revealed partially hydrated to hydrated bentonite and striations between the bentonite and recompacted soil liner, and between the bentonite and geomembrane. The striations confirmed that shear displacement had occurred and was probably a contributing factor to the formation of the geomembrane wrinkles. However, some of the displacement might also have occurred along the smooth geomembrane/geonet interface because striations were observed at the smooth geomembrane/geonet interface in a few locations. This possibility is discussed in more detail in Section 3.3.

The GCL was installed between October and November 1995. Cell 1B started to receive waste on 4 June 1996, which is approximately seven months after the bentonite was placed on the wet of optimum RSL. This seven-month period also included the moist spring months and, thus, it was concluded that the bentonite hydrated at a low normal stress. The water content of bentonite samples obtained after removal of the waste ranged from 64 to 86%. The water content of the bentonite after manufacturing ranges from 10 to 15%. It is anticipated that the bentonite hydrated by attracting moisture from the underlying RSL along the base of the cell. This attraction of moisture was also observed in the partially hydrated laboratory tests described in Section 3.2. In these tests, the bentonite of the GCL was placed on top of the RSL and allowed to absorb moisture until equilibrium was obtained. At low normal stresses, the vertical dial gage showed a vertical expansion, thus confirming the absorption of moisture from the RSL.

Other factors that may have contributed to the bentonite hydration were surface water infiltration from the Cell 2 side of the interim berm and/or groundwater migrating upward into the RSL. This caused the RSL to become soaked and soft near the northeast corner of Cell 1B. After initial installation, the GCL and RSL had to be removed and an underdrain system installed to drain the water acting on the RSL. After installation of the underdrain system, the hydrated and damaged GCL was removed until "dry" bentonite was encountered. However, it is possible that some of the remaining bentonite may have been partially hydrated. In addition, the new GCL was probably able to hydrate from the repaired RSL, which was also placed wet of optimum. These factors probably resulted in the presence of a layer of hydrated bentonite along the base of the cell. Along the 33% (1V:3H) side slope, it is believed that the bentonite was at least partially hydrated by also attracting moisture from the underlying RSL. However, bentonite samples were not obtained from the side slope and, thus, the magnitude of bentonite hydration is not known.

The underdrain system was installed between December 1995 and January 1996 and consists of a 150 mm diameter HDPE perforated pipe placed in a collection trench. The perforated pipe is located approximately 0.75 m below the bottom of the RSL. The trench is approximately 0.6 m deep, 1.2 m wide, and starts 0.15 m below the bottom of the RSL. After excavation, the trench was lined with a geotextile filter fabric and then backfilled with drainage stone after placement of the pipe. Figure 1 shows the location of the underdrain system as a heavy dashed line at the southern end of Cells 1 and 2.

2.6 Failure Investigation

The initial failure investigation involved determining the full extent of the wrinkling in the 1.5 mm thick geomembrane. After waste removal, it was readily observed that the wrinkles in the geosynthetics formed an arc as shown in Figure 4. The next step involved removing the waste from "inside" the arc. This required that a sizable portion of the relocated waste in Cell 1B be moved into the newly certified Cell 2. The final phase of the investigation involved analyzing samples of the GCL and observing the RSL for damage. Waste continued to be excavated as long as damage was encountered in the composite liner system. In all, an estimated 229,200 m^3 of waste was removed from Cells 1A and 1B. The total amount of waste in Cell 1B at the time of the failure was approximately 305,600 m^3. The failure investigation was limited to the base of Cell 1B because the majority of the observed damage to the composite liner system was limited to this area. However, the area that was wrinkled on the 33% (1V:3H) side slope on the southern end of Cell 1B was also repaired.

After waste removal, several large tears (0.3 to 2 m wide) and numerous small tears (less than 0.3 m wide) were found in the geomembrane portion of the GCL, and two ruptured geomembrane seams were discovered. The large tears were located approximately below where the cracks were observed in the cover soil at the crest of the slope. As a result, an initial failure surface was assumed to extend from the cracks at the crest of the slope, through the relocated waste and geomembrane to the hydrated bentonite, and along the hydrated bentonite/RSL interface to the temporary leachate collection trench (Figure 5).

After waste excavation, it was found that the bentonite portion of the GCL also fared poorly. Bentonite had randomly stayed affixed to the geomembrane or adhered to the RSL and, in other areas, had migrated into wrinkles or flowed away from stress concentrations. This resulted in an inconsistent layer of bentonite in the affected area.

To further investigate the failure, the anchor trench along the edge of Cell 1B was opened using five test pits, approximately 1 m deep and spaced 60 m apart. The cover soil was removed to expose the underlying geotextile and geonet. The geotextile and geonet were then cut to observe the underlying geomembrane. All of the test pits showed that the geomembrane was present, undisturbed, and still anchored in the anchor trench. Therefore, it was assumed that the geomembrane probably tore or ripped along the bottom of Cell 1B, which resulted in no damage or tension being developed in the anchor trench.

2.7 Remedial Measures

The remedial measures consisted of welding another layer of the geomembrane-backed GCL over the damaged area. However, in many instances the folds or wrinkles of the damaged GCL had to be flattened to obtain intimate contact between the bentonite of the newly placed GCL and the underlying damaged liner. This was accomplished by cutting the folds open and laying the existing GCL flat onto the recompacted soil liner. This reduced the time that the intact recompacted soil liner would be exposed and possibly damaged by rain water and/or leachate. Approximately 6,070 m^2 of the 14,164 m^2 liner system in Cell 1B was damaged.

It is extremely important to note that the failure at this facility could have gone undetected. Had the owner/operator not been disengaging the temporary leachate collection system, the failure probably would not have been discovered. The cracks on the slope face would not have been sufficient to conclude that a failure had occurred, nor have warranted further investigation. Further investigation then revealed the slightly heaved protective sand at the slope toe, and the excavation of the temporary leachate collection system provided additional evidence of the failure. In summary, small surface cracking, especially in a concentrated area, can be the manifestation of significant displacement and damage to the underlying composite system and in particular the geomembrane. These features should be carefully monitored and not covered up or dismissed as settlement induced.

3 LABORATORY TESTING

3.1 GCL Specimen Preparation and Test Method

A modified Bromhead ring shear apparatus that utilizes an annular specimen with an inside and outside diameter of 40 and 100 mm, respectively, was used for the testing described herein. For each GCL test, a 1.5 mm (60 mil) thick high density polyethylene (HDPE) geomembrane-backed GCL was glued to the top platen. The smooth geomembrane-backed GCL was manufactured by GSE Lining Technology, Inc. of Houston, Texas, USA. The bentonite typically used in this GCL is Wyoming bentonite with a liquid limit and plasticity index of 600 to 650 and 560 to 610, respectively (Gleason et al. 1997).

The recompacted soil liner consists of an off-site gray lean clay. The clay liner sample used in the ring shear tests was hand excavated from the RSL after removal of the waste and geomembrane-backed GCL. The clay classifies as low plasticity clay (CL) according to the Unified Soil Classification System with a liquid limit of 31% and a plasticity index of 14%. Hydrometer tests conducted during the current study revealed that 91 to 98% of the soil liner passes the 0.075 mm sieve (US Standard Sieve No. 200) and the clay size fraction (percent by weight finer than 0.002 mm) is 27 to 35%. Standard Proctor compaction tests (ASTM D 698) revealed that the optimum water content of the RSL is 9 to 11% and the maximum dry unit weight is 19.3 to 20.1 kN/m^3. Quality assurance records show that the RSL was placed at water contents between 0 and 2% wet of optimum at an average water content of 11.5% and a dry unit weight of 19.2 to 19.5 kN/m^3.

The ring shear RSL specimens were obtained by air drying a portion of the hand excavated sample from Cell 1B. The air dry soil was crushed with a ceramic pestle and processed through a 0.425 mm sieve (US Standard Sieve No. 40). The clay was mixed to the desired water content using distilled water. The ring shear specimens were compacted directly into the ring shear specimen container using a Harvard miniature compactor. The desired dry unit weight was obtained by compacting the appropriate weight of moist soil into the specimen container using two lifts. After compaction, the top platen, with a secured geomembrane-backed GCL with the bentonite exposed, was placed on the compacted soil and the specimen container was installed in the ring shear apparatus.

The geomembrane-backed GCL was tested with the bentonite in contact with the RSL under two different bentonite hydration conditions: (i) partially hydrated; and (ii) hydrated. The partially hydrated condition corresponds to the bentonite being placed on top of the RSL and being able to absorb moisture until equilibrium was obtained. This was accomplished by modifying the existing ring shear specimen container. Thirty-two horizontal holes were drilled along the outer edge of the specimen container so that distilled water, which was added to the specimen water bath, could only enter the bottom of the RSL specimen. This allowed the RSL to access the water in order to model the presence of groundwater and/or surface infiltration without inundating the bentonite. Water was added to the specimen container such that the level in the water bath never exceeded the upper surface of the RSL. It is thought that this simulated the condition of placing bentonite in direct contact with the RSL. A standard ASTM test method (ASTM D 4546) was used to estimate the end of primary swelling of the bentonite. The bentonite absorbed water immediately when it was placed on top of the RSL in the ring shear apparatus. This hydration phase occurred at a normal stress of 17 kPa and lasted three to four days. A normal stress of only 17 kPa was used for the hydration phase because waste placement started seven months after the GCL was placed on the RSL. The average water content of the bentonite after testing was 58%, which suggests a partially hydrated condition as discussed in Section 3.2.

Eid and Stark (1997) showed that the shear strength of this unreinforced GCL is sensitive to the normal stress applied during hydration. Eid and Stark showed that a reduction of 30 and 25% occurred in the measured peak and residual friction angles, respectively, due to hydration at 17 kPa versus hydration at the shearing normal stress. Therefore, selection of the hydration normal stress is important. Field events suggest a hydration normal stress of 17 kPa was justified to simulate no waste placement for seven months.

For the hydrated condition, the RSL specimen was inundated with distilled water and the bentonite was allowed to hydrate until the end of primary swelling or vertical deformation ceased at a normal stress of 17 kPa. This was accomplished by adding distilled water to the specimen water bath so that the water level was at the RSL/bentonite interface and allowing approximately 15 days for hydration. The hydration phase was continued until primary swelling ceased. As a result, the bentonite could absorb as much water as desired and achieve a fully hydrated condition. The water content of the bentonite after testing was 178%, which indicates a hydrated condition.

3.2 GCL Interface Test Results

Figure 8 shows that the peak shear strength of bentonite decreases as the water content increases from the manufactured value of 10 to 15%. More importantly, at a water content between 60 and 80% the majority of the strength loss has occurred. Therefore, the partially hydrated tests resulted in an average water content at or below the transition water content of 60 to 80% while the hydrated tests yielded an average water con-

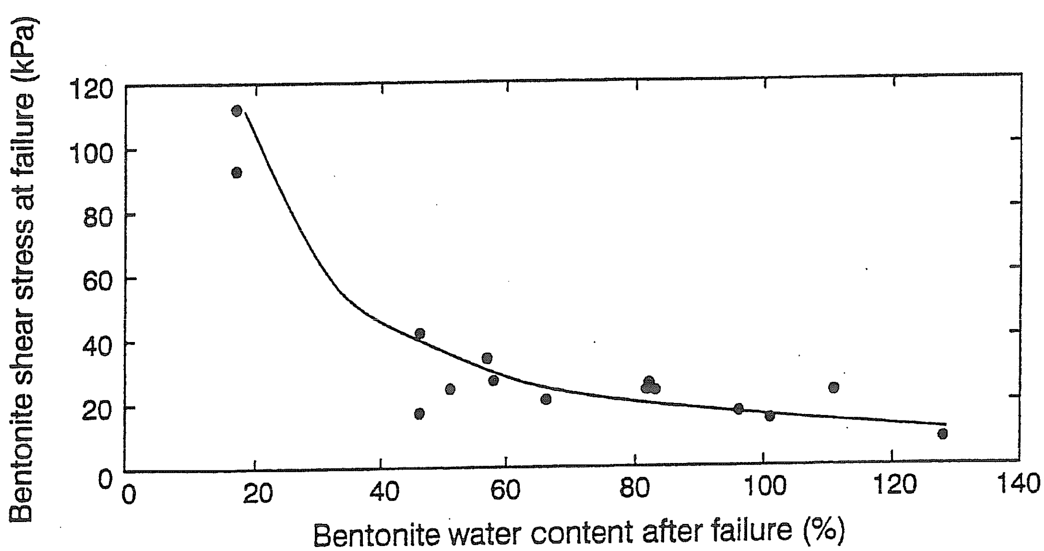

Figure 8. Relationship between bentonite shear strength and water content (data from Daniel et al. 1993).

tent above the transition water content. The field water contents range from 64 to 86%, which suggests that the bentonite had undergone a significant strength loss in the field due to hydration. The field bentonite water content may have been even higher if the bentonite consolidated under the weight of the relocated waste. However, it is anticipated that the amount of bentonite consolidation was small, if any, due to the presence of a geomembrane and RSL.

A shear displacement rate of 0.015 mm/minute was used for all of the RSL/GCL ring shear tests. This is the slowest displacement rate available for the modified Bromhead ring shear apparatus and probably corresponds to a drained condition for this interface. Stark and Eid (1997) showed that a displacement rate of 0.018 mm/minute is slow enough to create a drained condition for high plasticity clays (liquid limit of about 300). In each test, the RSL/GCL specimen was sheared until the residual strength was reached, which typically required 70 and 40 mm of displacement for the partially hydrated and hydrated conditions, respectively.

Figure 9 presents the peak and residual failure envelopes from the partially hydrated smooth geomembrane-backed GCL/RSL interface shear testing. The partially hydrated bentonite/smooth geomembrane interface exhibits a peak and residual friction angle of approximately 15 and 9°, respectively. Superimposed in Figure 9 are the peak and residual failure envelopes from the hydrated smooth geomembrane-backed GCL/RSL interface tests. It can be seen that the hydrated tests yield lower peak and residual failure envelopes than the partially hydrated tests. The hydrated bentonite/smooth geomembrane interface exhibits a peak and residual friction angle of approximately 8 and 5°, respectively.

The peak shear strength values measured in a ring shear device can be less than comparable direct shear values (Stark and Poeppel 1995). This is caused by the difference in shear displacement between the inner and outer edges of the specimen. The difference in the shear displacement is caused by the outer specimen edge being slightly past the peak shear resistance while the inner specimen edge is mobilizing the peak strength. This can result in a slightly lower peak shear strength than that measured in a direct shear apparatus. However, Hvorslev (1937, 1939) showed mathematically and experimentally that the maximum shear resistance measured in a ring shear device is in agreement with the peak shear strength measured in a direct shear apparatus when an annular specimen with a ratio of inner, R_1, to outer, R_2, radii greater than or equal to 0.5 is used.

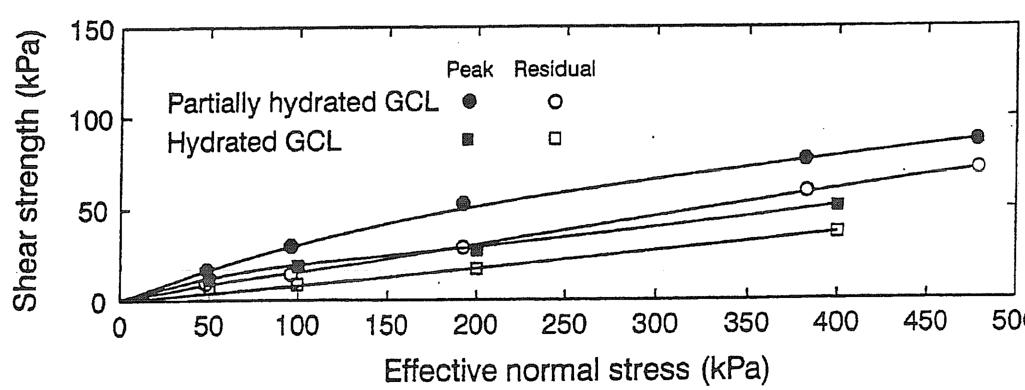

Figure 9. Drained peak and residual shear strength envelopes for GCL/RSL interfaces.

A R_1/R_2 ratio of approximately 0.6 was used for the interface tests described herein and, thus, the peak interface shear strength should be in agreement with the direct shear values. This has been confirmed using a number of different geosynthetic interfaces and a direct shear apparatus that satisfies the standard test method, ASTM D 5321.

3.3 Smooth Geomembrane/Geonet Interface Testing

The smooth geomembrane/geonet interface was also tested using the modified Bromhead ring shear apparatus because striations were observed at this interface after waste removal. For each test, a 1.5 mm (60 mil) thick HDPE geomembrane, manufactured by GSE Lining Technology, Inc. of Houston, Texas, USA, was glued to the bottom platen. The HDPE drainage geonet was also manufactured by GSE Lining Technology, Inc. and consists of a two-strand geonet with a thickness of 5 mm. The geosynthetics were glued to the Lucite ring using a thin coat of epoxy cement and allowed to cure separately for 24 hours under a normal stress of 300 kPa. The curing time aided bonding of the geosynthetics and minimized vertical displacements due to the glue during testing. The specimen container and geosynthetic were marked to ensure that the geosynthetics did not slip during shear. The surface of the geomembrane was also wiped, if necessary, using a paper towel to minimize the effect of fingerprints and perspiration on the interface strengths (Yegian and Lahlaf 1992). The ring shear tests were conducted at a laboratory temperature of 20°C. A shear displacement rate of 0.015 mm/minute was used for the smooth geomembrane/geonet interface tests.

Figure 10 presents the peak and residual failure envelopes from the smooth geomembrane (GM)/geonet (GN) interface shear testing. The smooth GM/GN failure envelopes are clearly stress dependent. At normal stresses greater than approximately 300 kPa, the slope of the peak and residual failure envelopes increases. This is attributed to the geonet embedding into the geomembrane and providing some additional shear resistance. Superimposed in Figure 10 are the peak and residual failure envelopes from the hydrated smooth geomembrane-backed GCL/RSL interface tests. It can be seen from Figure 10 that the peak smooth GM/GN failure envelope is below the peak GCL/RSL failure envelope for effective normal stresses ranging from 0 to approximately 225 kPa. The effective normal stress acting on the base of the failure surface in Figure 5

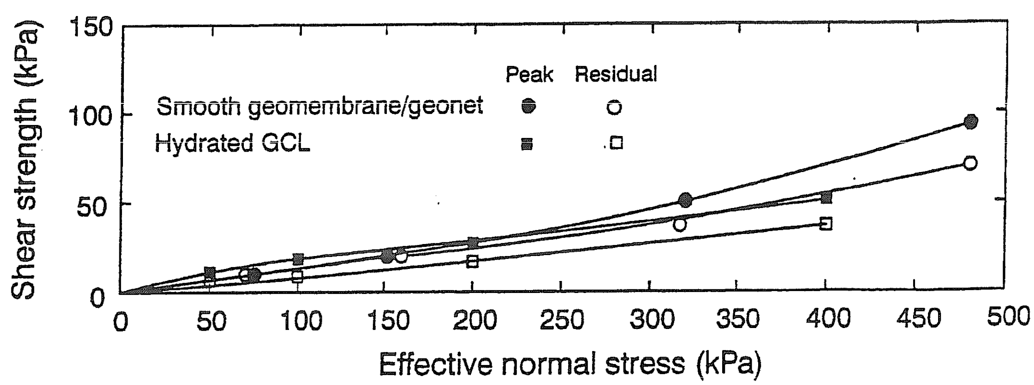

Figure 10. Peak and residual shear strength envelopes for hydrated GCL/RSL and smooth GM/GN interfaces.

ranges from 10 kPa at the toe of the slope to 440 kPa where the failure surface enters the relocated waste. More importantly, after waste excavation, striations were observed near the toe of the slope on the GM/GN interface whereas the striations on the hydrated bentonite/RSL interface were concentrated behind the slope toe.

The following conclusions can be drawn from this comparison: (i) the critical interface can change depending on the effective normal stress; and (ii) it is possible that sliding could have occurred at the smooth GM/GN interface as well as the bentonite/RSL interface depending on the effective normal stress, which helps explain the presence of displacement/striations at the smooth GM/GN interface as well as the bentonite/RSL interface.

4 STABILITY ANALYSES

4.1 Overview

All of the cross sections were analyzed using a two-dimensional (2-D) stability method that satisfies all equilibrium conditions (Spencer 1967). The microcomputer program XSTABL Version 5 (Sharma 1996) was used for the analyses. The input parameters, i.e. the unit weight and shear strength, used in the limit equilibrium analyses are presented in Table 1. It can be seen that the entire peak failure envelope was input directly into XSTABL instead of using a cohesion and friction angle for the geosynthetic interfaces. The failure envelopes are shown in Figures 9 and 10 and were inputted using combinations of shear and normal stress. The effect of waste unit weight and bentonite shear strength on the factor of safety, *FS*, was investigated and will be discussed subsequently. The waste cover material consists of sandy clay and was approximately 0.3 m thick (Earth Sciences Consultants, Inc. 1997).

Table 1. Material properties used in the stability analysis.

Material description	Moist unit weight (kN/m^3)	Shear strength parameters		Reference
		Cohesion (kPa)	Friction angle (°)	
Waste cover soil	18.9	0	33	Earth Sciences Consultants, Inc. (1997)
Relocated waste material	18.5	5	33	Earth Sciences Consultants, Inc. (1997) and the current study
Sand protective cover	17.3	0	33	Earth Sciences Consultants, Inc. (1997)
Bentonite/RSL interface	6.8	Peak envelope	Peak envelope	The current study
Smooth GM/GN interface	6.8	Peak envelope	Peak envelope	The current study

The shear strength parameters for the relocated waste were measured by Earth Sciences Consultants, Inc. (1997). Direct shear tests were conducted on reconstituted specimens of the relocated waste. The relocated waste was placed in a 0.3 m by 0.3 m square direct shear box (ASTM D 5321) at a unit weight of 18.5 kN/m^3 and a water content of 17.7% to simulate the field placement conditions. The relocated waste specimens were obtained from field samples that consisted of brown sandy clay with approximately 10% waste. It was concluded that a 0.3 m by 0.3 m square specimen was representative of field conditions because the relocated waste contained approximately 10% waste and 90% soil. Three direct shear tests were conducted on the relocated waste at effective normal stresses of 96, 191, and 373 kPa to cover the range of normal stresses acting on the liner system. The normal stresses were applied using pneumatic cylinders and a displacement rate of 0.25 mm/minute was used for the shear testing. The peak failure envelope for the relocated waste was found to correspond to a cohesion and friction angle of 5 kPa and 33°, respectively (Table 1). The shear stress-displacement relationships from these direct shear tests did not exhibit a post-peak behavior after approximately 60 mm of displacement as shown in Figure 11.

4.2 Failure Surface

The large tears observed in the geomembrane after waste removal were located approximately below the cracks observed at the crest of the slope (Figure 5). As described previously, the failure surface was assumed to extend from the cracks at the crest of the slope, through the relocated waste and geomembrane to the hydrated bentonite,

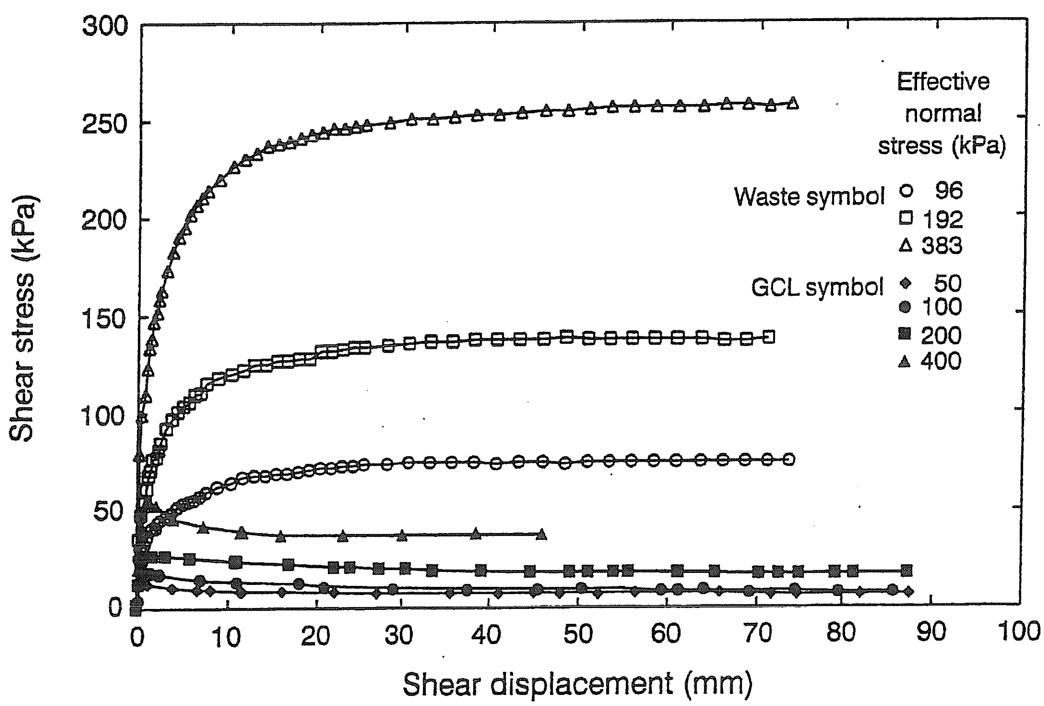

Figure 11. Shear stress-displacement relationships for a waste and hydrated GCL/RSL interface.

and along the hydrated bentonite and/or smooth GM/GN interface to the toe of the slope. Therefore, it was concluded that the toe of the slope moved towards the temporary leachate collection trench. This sliding occurred prior to excavation of the protective sand cover (Figure 3) because a 0.2 to 0.4 m high bulge was observed in the protective sand cover. After the slide block moved, the geomembrane tore and cracking probably developed at the crest of the slope and along the southwest portion of the slide mass (Figure 6).

It is anticipated that the slide mass at the toe of the slope moved first and, thus, the analysis should yield $FS \approx 1$ for a failure surface similar to the one shown in Figure 5. A search was conducted to locate the critical failure surface through the waste (Figure 5). This was accomplished by specifying a failure surface along the base of the slide mass that corresponded to the bentonite/RSL interface and performing a search for the critical orientation of the failure surface through the waste using XSTABL.

Using the peak hydrated bentonite/RSL interface failure envelope shown in Figure 9 and the input parameter values presented in Table 1, the failure surface in cross section A-A′ (Figure 5) yielded a 2-D FS value of 1.01. The analysis was repeated with the peak smooth GM/GN interface failure envelope instead of the failure envelope for the hydrated bentonite/RSL interface. This analysis yielded $FS = 1.11$, thus, confirming field observations that showed sliding primarily occurred at the RSL/hydrated bentonite interface.

A composite failure envelope was used in the analysis because striations were also observed at the smooth GM/GN interface. As mentioned previously, the composite failure envelope consists of the peak smooth GM/GN interface failure envelope at effective normal stresses less than 225 kPa and the peak hydrated bentonite/RSL interface for an effective normal stress greater than or equal to 225 kPa. The composite failure envelope yielded $FS = 0.98$, which is in agreement with field observations of striations on the GM/GN interface near the toe of the slope and striations concentrated on the hydrated bentonite/RSL interface behind the slope toe.

The analysis was also repeated using the partially hydrated bentonite/RSL interface failure envelope (Figure 9). This analysis yielded $FS = 1.36$ (Table 2). As a result, it was concluded that the bentonite on the base of the slide mass was probably hydrated, allowing the slide mass to move toward the temporary leachate collection trench. A hydrated condition was also in agreement with field observations and bentonite water contents measured after waste excavation.

Table 2. Factors of safety, *FS*, for various cross sections and peak shear strengths.

Cross section	2-D *FS* value			
	Fully hydrated bentonite/ RSL interface	Partially hydrated bentonite/ RSL interface	Smooth GM/ GN interface	Composite of fully hydrated bentonite/RSL and smooth GM/GN interface
A-A′	1.01	1.36	1.11	0.98
B-B′	1.33	1.67	1.30	1.29
C-C′	1.47	2.50	1.32	1.29

A number of other cross sections were analyzed. The results from two of these cross sections are shown in Table 2 and Figure 6 presents the location of these two cross sections. These cross sections were chosen because they correspond to the cracking that was observed along the southern edge of Cell 1B. It can be seen from Table 2 that the $FS > 1$ for cross sections B-B' and C-C'. As a result, these cross sections were deemed not critical. In addition, the direction of sliding was perpendicular to the temporary leachate collection trench.

In summary, cross section A-A′ appears to be the most representative 2-D cross section for the following reasons: (i) it yielded $FS \approx 1$; (ii) the leachate collection pipe shown in Figure 1 moved vertically upward after the protective sand material was excavated from the southern end of the temporary leachate collection trench indicating that sliding was possibly occurring parallel to the pipe; (iii) the geomembrane wrinkles were parallel to the toe of the waste slope indicating that sliding was perpendicular to the slope toe; and (iv) the surficial cracking occurred along the southern portion of the cell indicating movement away from or to the west of the southern containing berm (Figure 1).

A three-dimensional (3-D) back-analysis was also conducted to investigate the importance of end effects on the slide mass. Stark and Eid (1998) showed that the difference in 2-D and 3-D back-calculated friction angles can be as large as 30% when the material along the vertical sides, e.g. municipal solid waste, has a higher shear strength than the material along the base of the slide mass, e.g. a weak geosynthetic interface. A computer model, CLARA 2.31 (Hungr 1988), which utilizes either an extension of Bishop's simplified method (Bishop 1955) or Janbu's simplified method (Janbu et al. 1956) to three dimensions (Hungr 1987) was used for the analysis. The assumptions required to render the 3-D analysis statically determinate are the same as for the 2-D methods. The 3-D analysis yielded a *FS* value approximately 6% greater than the 2-D value. The slight difference between the 2-D and 3-D *FS* values was caused by the haul road along the northern side of the slide mass creating a void next to the slide mass. As a result, the end effects mobilized along the north side of the slide mass were small. On the southern side of the slide mass the geosynthetic liner system probably provided a limited amount of end resistance. As a result, the 2-D and 3-D *FS* values differed by a small amount.

4.3 Effect of Waste Unit Weight

As mentioned previously, the unit weight of the relocated waste was greater than the new MSW that was being placed in Cell 1A because it consisted of up to 90% soil and was at or near saturation. The effect of unit weight of the relocated waste on the *FS* value was investigated using cross section A-A′, the input parameters shown in Table 1, and the peak failure envelope for the hydrated bentonite/RSL interface. The results of the analysis are shown in Figure 12. It can be seen that even at a unit weight of 9.4 kN/m^3, $FS < 1.15$ for a waste friction angle and cohesion of 33° and 5 kPa, respectively. Kavazanjian et al. (1995) suggest that new municipal solid waste exhibits a friction angle of 33° and a cohesion of 24 kPa for normal stresses less than 30 kPa. The *FS* value for a unit weight of 9.4 kN/m^3 and a cohesion and friction angle of 24 kPa and 33°, respectively, is approximately 1.16. Therefore, the slope probably would have been marginally stable even if new MSW, and not relocated waste, was placed in Cell 1B. In summary, it appears

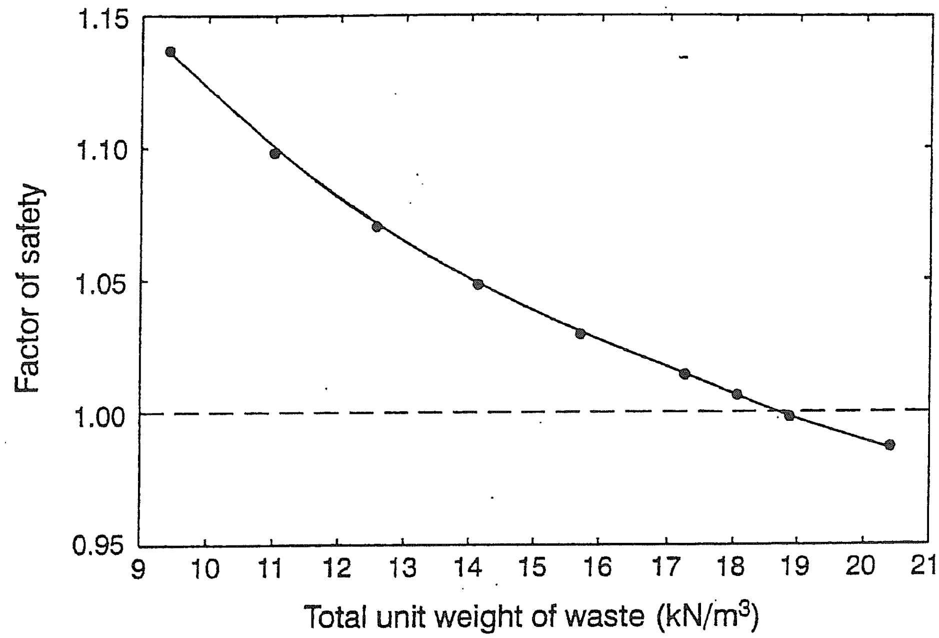

Figure 12. Effect of waste unit weight on factor of safety in Cell 1B.

that the steep interim slope (1V:2.5H) and the heavy relocated waste induced shear stresses that exceeded the shear resistance along the critical geosynthetic interface.

4.4 Post-Failure Analysis

The previous stability analyses were performed using peak interface strengths (Tables 1 and 2). Given the magnitude of movement indicated by the wrinkling of the GCL, there may have been sufficient deformation to reach a post-peak or residual strength condition. The interface test results shown in Figure 11 indicate that a residual strength condition was obtained at a shear displacement of approximately 40 mm for the hydrated GCL/RSL interface. An analysis using the final configuration of the deformed slope and the residual failure envelope for the hydrated bentonite/RSL interface should yield *FS* = 1 because the field deformations were greater than 40 mm. However, one difficulty in conducting this analysis was that the topography of the slope was not obtained immediately after the slope was unloaded. As a result, the surface topography was estimated using photographs and eyewitness accounts.

Because cross section A-A′ was deemed critical, Table 3 presents the *FS* values corresponding to a residual strength condition and the deformed slope geometry (Figure 13). It can be seen that the hydrated bentonite/RSL interface yields *FS* =1, which is in agreement with field observations. Momentum effects were not considered in this analysis due to the limited movement and, thus, probable slow rate of movement. As expected, the partially hydrated bentonite/RSL and smooth GM/GN interfaces yielded *FS* > 1, which is in agreement with the *FS* values in Table 2. A composite residual failure envelope was not applicable because the hydrated bentonite/RSL interface failure envelope is lower for the full range of normal stresses (Figure 10).

Table 3. Factors of safety, *FS*, for cross section A-A′ after sliding and residual shear strengths.

Cross section	2-D *FS* value			
	Fully hydrated bentonite/ RSL interface	Partially hydrated bentonite/ RSL interface	Smooth GM/ GN interface	Composite of fully hydrated bentonite/RSL and smooth GM/GN interface
A-A′	0.99	1.31	1.07	Not applicable

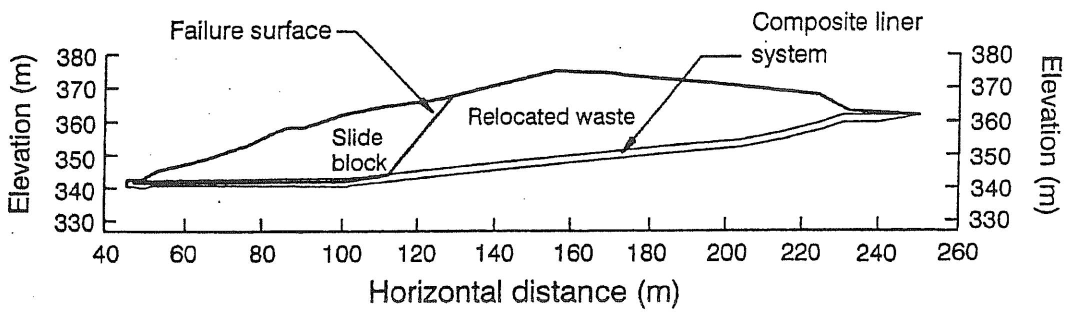

Figure 13. Cross section A-A′ after slope unloading.

Therefore, it may be concluded that a post-peak or residual interface strength condition can develop in the field and can lead to additional movement or sliding unless the slope is buttressed and/or unloaded. This is evident because the *FS* values for cross section A-A′ before unloading and using a residual hydrated bentonite/RSL interface strength yielded *FS* = 0.82. Fortunately, the relocated waste slope was unloaded quickly, which probably prevented larger movements from occurring.

The results of this analysis also can be used to confirm the design analysis or procedure presented by Stark and Poeppel (1994). A *FS* value greater than unity with a residual strength from ring shear testing assigned to all potential slip surfaces should be satisfied because field shear displacements and the effect(s) of progressive failure are not known. This will ensure that a large slope failure does not develop even though a post-peak or residual strength condition develops due to shear displacement and/or strain incompatibility, which is described in Section 4.5. The results of the post-failure analysis can also be used to conclude that the torsional ring shear device provides a reasonable estimate of field peak and residual interface strengths. The main advantage of the ring shear apparatus is that large continuous shear displacement can be applied in one direction in order to achieve a residual strength condition.

4.5 Strain Incompatibility

It is anticipated that another cause of the development of a post-peak or residual shear strength along a geosynthetic interface is the strain incompatibility between the waste and underlying interfaces. For example, the relocated waste mobilizes a peak shear strength at a displacement 10 to 15 times larger than the displacement at which the hydrated bentonite/RSL interface mobilizes a peak shear strength (Figure 11). As a result, progressive failure can cause the interface to mobilize a post-peak shear strength before the peak strength of the relocated waste has been mobilized. This is evident from Figure 11 where the peak shear strength of the interface is mobilized at a shear displacement of 2 to 3 mm while the peak strength of the waste is mobilized at a displacement of 30 to 40 mm. At a shear displacement of 40 mm the GCL/RSL interface has mobilized a residual shear strength.

In summary, this strain incompatibility can facilitate the development of slope instability because the geosynthetic interface may mobilize a post-peak or residual strength while the waste is mobilizing a strength that is significantly below the peak strength. This can be incorporated into a design by assigning a residual strength to the critical interface or slip surface and requiring a factor of safety, $FS > 1$.

5 CONCLUSIONS

The precise failure mechanism at the Mahoning Landfill is not known. However, the significant damage observed in the geosynthetic components of the composite liner system suggests that the failure was translatory. Sliding was observed to have primarily occurred at the interface between the hydrated bentonite of the geomembrane-backed geosynthetic clay liner (GCL) and the underlying recompacted soil liner (RSL). It also appears evident that the slope inclination, slope height, physical characteristics, e.g. high unit weight, of the "relocated waste" in Cell 1B, and possibly the smooth geomembrane/geonet interface, played a significant role in mobilizing the slide block. The following conclusions can be drawn from this case history:

1. The analyses described herein suggest that the failure could have been predicted using state-of-practice limit equilibrium stability analyses and interface test results. For cost considerations, designers could assume a hydrated condition for the bentonite and test the other problematic interfaces to determine the critical interface instead of testing the bentonite/RSL interface.
2. Small surface cracking, especially in a concentrated area, can be the manifestation of significant shear displacement and damage to the underlying composite liner system and, in particular, the geomembrane. Site personnel should frequently observe the ground surface for cracks. These features should be carefully monitored and not covered up or dismissed as settlement induced.
3. Careful planning should be conducted to ensure that construction activities proceed as planned such that slopes are not overbuilt or oversteepened. If there are construction delays, waste may need to be diverted to another area or a slower waste relocation process may be required to ensure that the slope is not overbuilt. The unavailability of Cell 2 probably facilitated the construction of a steep "relocated waste" slope in Cell 1B.
4. Changes in waste properties should be observed and documented and their impact on slope stability quickly assessed. Relocated waste may exhibit a higher unit weight and lower shear strength parameters than new municipal solid waste and the stability analyses should incorporate these differences. The increase in unit weight is usually caused by the inclusion of soil or other material during the excavation process.

5. Unreinforced GCLs should not be placed in direct contact with a RSL that is placed wet of optimum. If it is, a shear strength value that corresponds to a fully hydrated condition should be used for stability analyses for interim and final slopes. This is recommended because the bentonite will probably become hydrated by attracting moisture from the underlying soil liner and/or other sources. The hydrated, unreinforced bentonite will exhibit extremely low shear strength and bearing capacity, which may result in slope instability and/or bentonite migration due to stress concentrations, respectively. One technique for increasing the shear strength and decreasing the potential for bentonite migration is to encapsulate the unreinforced bentonite using two geomembranes.
6. The critical interface can be stress dependent and may change along the length of the failure surface. This can be incorporated in a stability analysis by using a composite failure envelope. For example, a composite failure envelope for this case history would consist of the peak smooth GM/GN interface failure envelope for effective normal stresses less than 225 kPa and the peak hydrated bentonite/RSL interface failure envelope for effective normal stresses greater than 225 kPa. This will ensure that the stress dependent nature of the geosynthetic interfaces is incorporated in the analysis.
7. Because field interface displacements and the effect(s) of progressive failure are not known, a factor of safety, $FS > 1$ with a ring shear residual interface strength assigned to all potential slip surfaces should be satisfied in addition to meeting regulatory requirements.
8. Engineers should investigate the stability of interim slopes, even though it may not be required by regulations, to ensure slope stability during construction. This is recommended because a slope can be overbuilt without being detected, underlying geomembranes can be easily damaged and/or torn by typical driving forces, and, most importantly, people are usually required to work below the exposed slope.

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ABBREVIATIONS

AASHTO:	American Association of State and Highway Transportation Officials
ASTM:	American Society for Testing and Materials
EPA:	Environmental Protection Agency
GCL:	geosynthetic clay liner
GM:	geomembrane
GN:	geonet
HDPE:	high density polyethylene
MLI:	Mahoning Landfill, Inc.
MSW:	municipal solid waste
RSL:	recompacted soil liner