CONSTRUCTION CONSIDERATIONS FOR GEOSYNTHETIC LINER SYSTEM STABILITY

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ABSTRACT: Construction conditions experienced during the installation of a landfill geocomposite liner system lead to two slides of a 3H:1V internal slope being developed for final disposal of Flue Gas Desulfurization byproduct material (FGD). The approved geocomposite liner consisted of 18-inches of compacted clay, a 30-mil thick PVC geomembrane, 12-inches of drainage sand, and 24-inches of FGD used as protective cover material. A stability improvement program featuring the use of geosynthetics to reinforce the 3H:1V slopes was implemented and no re-occurring or additional slides have occurred at the site.

The slides occurred in two different areas of the 3H:1V slope and encompassed 2.6 and 5 acres in the Fall of 1996 and Summer of 1997, respectively. The failures developed by sliding along the PVC geomembrane/compacted clay liner interface during or shortly after placement of the protective cover material. The shear strength of this interface is characterized by a frictional resistance that is highly sensitive to the moisture content of the exposed compacted clay. Laboratory direct shear tests revealed reductions in the available shear strength of 35% and 37% for the peak and residual values as the moisture content increased 8 percentage points. In addition, shear stresses induced by the construction activities resulted in shear displacements along the interface, which in some cases, were large enough to lower the available resistance of the geomembrane/compacted clay interface to a post-peak or residual condition.

This paper will discuss the slope failures, testing and analysis used to evaluate the interface shear-strength parameters, relationship between moisture content and geomembrane/compacted clay interface strength, and the successful remedial measures. As part of the interface strength evaluation, the mechanisms that may lead to (1) increases in moisture content at the geomembrane/compacted clay interface during construction and (2) accumulation of shear displacement and strength loss that may occur along a geomembrane/compacted clay interface due to construction activities will be presented.

1 -INTRODUCTION

The General James M. Gavin Power Plant is owned by American Electric Power and is located on the Ohio River at river mile 258 just south of Cheshire, Ohio. It consists of two 1300 MW coal-fired steam electric generating units. As a means of compliance with the Clean Air Act amendments of 1990, a flue gas desulfurizaton (FGD) system was installed on each generating unit to reduce the amount of sulfur dioxide (SO_2) discharged by the plant. The FGD system uses a lime-based reagent to react with the flue gas to remove the SO_2 component of the gas prior to exit to the atmosphere. To stabilize/solidify the by-product making it suitable for placement in a residual waste landfill, fly ash and lime are mixed with it. The stabilized/solidifed FGD product consists of about 57% by-product, 40% fly ash, and 3% lime. The approved geocomposite liner system for the stabilized/solidified FGD landfill consists of 0.46 m (18-inches) of compacted clay with a hydraulic conductivity not to exceed 1.0 x 10^{-6} mm/sec, 0.75 mm (30-mil) thick poly-vinyl chloride (PVC) geomembrane, a 0.3 m (12-inches) thick layer of drainage sand, and 0.61 m (24-inches) of FGD used as a protective cover material. The FGD protective cover material was placed immediately above the drainage sand, i.e., with no separating geotextile.

The FGD landfill site is located about 2.1 km (1.25 miles) northwest of the power plant in an area that has been heavily strip-mined and largely unreclaimed. The area of the FGD placement is about 910,600 m^2 (255 acres) and consists of 3 adjacent valleys. The long-term disposal plan is to fill each valley and then fill over the filled valleys and existing topography. As a result, the landfill capacity is estimated at 37 to 44 million cubic meters (49 to 57 million cubic yards). The landfill is planned to be developed over the course of six phases (A through F). During the construction of Phase B in Valley 2, two slope failures encompassing areas of approximately 10,500 and 20,235 m^2 (2.6 and 5 acres) occurred in the Fall of 1996 and Summer of 1997, respectively. The average inclination of the failed slopes is 3H:1V. A typical cross-section through Phase B in Valley 2 is shown in Figure 1. It can be seen that Phase B fills Valley 2, Phase A fills Valley 1, Phase E "caps" or is placed over Valley 1, and Phase F caps Valleys 1 and 2. Phase C involves filling Valley 3 (not shown) and Phases D and F involve "capping" portions of Valleys 1, 2, and 3. Valley 2 has a depth of approximately 46 m (150 feet) and the subsequent Phase F filling will result in a depth of 46 m (150 feet) for a maximum waste depth of 92 m (300 feet) in Valley 2. This paper describes the two slope failures, failure mechanism, subsurface investigation, engineering properties of the stabilized/solidified FGD product and PVC geomembrane/compacted clay liner (CCL) interface, back-analysis of the slides, slope remediation, and lessons learned.

2 DESCRIPTION OF SLOPE FAILURES AND FAILURE MECHANISM

After subgrade preparation for placement of the composite liner system in Phase B, the slope inclinations ranged from 3H:1V to 50H:1V (see Figure 1). In late October, 1996, shortly after placement of the drainage sand and protective cover material, a network of cracks in the protective cover and drainage sand were observed over approximately10,500 m^2 (2.6 acres) of the slope. The cracks were located in the 3H:1V portions of the slope. The parallel cracks extended across the slope and from the crest of the slope approximately 15 m (50 feet) down the slope. The width of the cracks ranged from 12 mm to 125 mm (0.5 inches to 5 inches) but did not show any significant vertical offset. This lack of vertical offset was caused by translational sliding occurring above the stiff compacted clay liner. Investigations at the crack locations revealed that the 0.75 mm (30-mil) thick PVC geomembrane was in significant tension, which resulted in localized tearing of the geomembrane. Figure 2 illustrates the condition of the PVC geomembrane in the October 1996 slide area. During the investigation, it was observed that a thick wet, film (approximately 10 mm in thickness) was present at the surface of the compacted clay immediately underneath the PVC geomembrane. Construction records indicate that the compacted clay liner in this area was compacted at a moisture content equal to 21.5%. The Standard Proctor (ASTM D 698, 1999) optimum compaction moisture content of the compacted clay liner is 19.3%. Post-slide investigations revealed that the surface of the clay had a moisture content of 27%, whereas the bottom of the clay liner had a moisture content equal to 21%.

In June 1997, another section of 3H:1V slope in Phase B became unstable and slid. Figure 3 illustrates the condition of the PVC geomembrane in the June 1997 slide area.

Construction records indicate that the clay in this area had been compacted to an average moisture content of approximately 19.5% or about the optimum moisture content. The PVC geomembrane was placed over the compacted clay and the moisture content of the clay was monitored before the drainage sand was placed. The moisture content was monitored with the PVC geomembrane in place by cutting a square section of the PVC geomembrane at a few locations and patching it after a sample of the clay liner was obtained for moisture content testing. No significant change in the moisture content was measured prior to placement of the drainage sand. The FGD protective cover was placed following placement of the drainage sand. The liner system was completed and stable for a period of two weeks. After two weeks, three temporary haul roads were constructed on the 3H:1V slope. The three haul roads were equally spaced along the 304 ft (92.7 m) long slope with one road constructed at the toe of the 3H:1V slope and one being constructed within approximately 15.2 m (50 ft) of the slope crest. A few days after the haul roads were put into service, the June 1997 slide occurred. As a result, it is anticipated that the shear stress induced by traffic on these roads contributed to the 1997 failure. The observed network of cracking was similar to the network observed for the October 1996 slide. The parallel cracks extended across the slope and from the crest of the slope approximately 50 feet (15 m) down the slope. The width of the cracks ranged from 0.5 to 5 inches (12 mm to 125 mm) but did not show any significant vertical offset. A significant finding during the failure investigation was that the average moisture content of the compacted clay liner at the geomembrane interface was 22.5%. This corresponds to an increase in moisture content of approximately 3% during placement of both the sand drainage layer and the protective FGD cover material.

Based on field observations, the slope failures occurred due to sliding at the PVC geomembrane/CCL interface. This caused tension in the PVC geomembrane and allowed the drainage sand layer and protective cover material to translate down slope. In some locations the geomembrane tore resulting in the failure surface migrating up through the drainage sand and protective cover material. The failures were translational in nature due to the compacted and stiff nature of the underlying compacted clay liner and existing terrain.

3 FAILURE INVESTIGATION

3.1 Laboratory Testing Program

During design of the landfill, a laboratory testing program was conducted to evaluate the shear strength of the foundation, drainage sand, and waste materials. The foundation soils in Valley 2 consist primarily of relatively deep alluvial deposits. The predominant deposits are stiff to very-stiff brown and grey lean silty clays that extend to depths of 4.6 to 17.7 m (15 to 58 feet). The alluvial soils are underlain by approximately 0.6 to 1.5 m (2 to 5 feet) of very-stiff residual brown silty clay. Bedrock was encountered at depths ranging from 4.9 to 17.7 m (16 to 58 feet). The undrained shear strength of the brown and grey silty clay was measured using unconfined compression tests on undisturbed specimens. The silty clays exhibited increasing undrained shear strength with depth as shown in Table 1 and could be separated into an upper and lower layer based on the undrained shear strength. Drained direct shear tests also were conducted and the effective stress friction angle ranged from 30 to 32 degrees. A review of these high shear strength parameters led to the conclusion that sliding did not occur in the foundation soils under the 3H:1V slope.

Table 1. Shear Strength Parameters for Foundation Soils in Phase B

Valley 2 Stratum	Undrained Shear Strength	Drained Friction Angle (degrees)
"Upper" Silty Clay	1,800 psf (86 kPa)	30
"Lower" Silty Clay	2,500 psf (120 kPa)	32

The effective stress shear strength parameters of the stabilized/solidified FGD waste product were estimated to be a cohesion of 48 kPa (1000 psf) and a friction angle of 15 degrees. These parameters were estimated from test results published by the Electric Power Research Institute (1984) on samples of the FGD sludge from the Conesville Plant in Conesville, Ohio. This power plant also is owned by American Electric Power and in general, the FGD handling is similar to the Gavin Power Plant.

The drainage sand material consists of a free-draining granular material that classifies as poorly graded gravel to well-graded sand according to the Unified Soil Classification System. The effective stress shear strength parameters of the drainage sand also were estimated to be cohesion of 0 psf (0 kPa) and a friction angle of 32 degrees. These parameters probably represent a lower bound of the shear strength of the compacted drainage sand layer.

The compacted clay liner consists of clay materials excavated from the landfill site and then compacted on the prepared subgrade. The borrow material classifies as a low- to high-plasticity clay according to the Unified Soil Classification System. The compaction specification for the clay was a standard Proctor (ASTM D 698, 1999) relative compaction of 95% and a compaction water content that ranged from 0% to +3%. Total stress shear strength parameters for the compacted clay liner were estimated to be a cohesion of 2,000 psf (96 kPa) and a friction angle of 0 degrees from the NAVFAC (1982) DM-7.1 manual. Evaluation of potential failure surfaces through the compacted clay liner indicates a high factor of safety that is not particularly sensitive to variations in the undrained shear strength. In summary, the high shear strength parameters measured or estimated for the foundation soils, drainage sand, and compacted clay liner under the 3H: 1V slope, indicates that sliding probably did not occur through these materials. As a result, the failure investigation next focused on the shear behavior of the PVC geomembrane/compacted clay interface.

3.2 Geosynthetics Testing

Following the October 1996 slide, large direct shear tests were performed in accordance with standard test method ASTM D 5321 (ASTM D 5321, 1998) to evaluate the shear strength of the PVC geomembrane/compacted clay interface. Specifically, the direct shear apparatus used in this study allows a 300-mm by 300-mm upper geosynthetic specimen to be sheared over a lower geosynthetic specimen that is 300 mm by 350 mm. The large direct shear device contains an upper and lower shear box each with a depth of

75 mm (3 inches). The normal stress is applied pneumatically and the shear displacement is limited to 50 mm (2 inches). As a result, peak and large displacement (not residual) shear strengths are reported for these tests.

The 0.75 mm (30 mil) thick PVC geomembrane installed at the site and tested herein was manufactured by Canadian General-Tower, Ltd. of Cambridge, Ontario, Canada. The geomembrane was manufactured with one side smooth and the other side embossed with a faille finish. The geomembrane was installed with the smooth side in contact with the compacted clay liner. Interface testing is being initiated at the University of Illinois to determine if the faille side may reduce the impact of moisture collection at the PVC geomembrane/CCL interface versus the smooth side. It is conceivable that the faille finish may be able to channel some of the moisture away from the interface than the smooth side.

Figure 4 illustrates the impact of compaction moisture content on the shear resistance of the PVC geomembrane/CCL interface using large direct shear test results at a normal stress of 7.2 kPa (150 psf). A normal stress of 7.2 kPa (150 psf) simulates the normal stress applied by the drainage sand and protective cover material. It can be seen that the peak friction angle (ϕ) decreases from approximately 21 degrees to about 14 degrees with an increase in compaction moisture content from 19.8 to 27.3%, respectively. As shown, the optimum moisture content for the standard Proctor compactive effort is 19.3%. It can also be seen that the interface exhibited a post-peak strength loss (approximately 10%) that appeared constant with increasing compaction moisture content. This post-peak strength loss (~10%) is significantly less than that typically observed (40-45%) for high-density polyethylene (HDPE) geomembrane/compacted clay interfaces (Stark and Poeppel 1994).

The large displacement friction angle (ϕ_{LD}) decreases from approximately 18 degrees to about 12 degrees with an increase in compaction moisture content from 19.8 to 27.3%, respectively. The small post-peak strength loss of the smooth side of the PVC geomembrane is attributed to the large and intimate contact area between the geomembrane and compacted clay and the high flexibility of the PVC geomembrane (Hillman and Stark 2000). The high interface strength and flexibility of a PVC geomembrane usually results in fewer wrinkles than HDPE geomembranes (Giroud 1995), which increases the interface contact area.

Figure 4 also presents the results on an infinite slope analysis to demonstrate the importance of compaction moisture content on the factor of safety of the PVC geomembrane/CCL interface. Limit equilibrium equations can be used to derive an expression for the factor of safety (FS) of an infinite, frictional soil slope with a slope angle of β as shown below:

$$FS = \left(1 - \frac{\gamma_w h_w}{\gamma h}\right)\frac{\tan\phi}{\tan\beta} = (1 - r_u)\frac{\tan\phi}{\tan\beta} \tag{1}$$

where γ (125 pcf; 19.6 kN/m^3) and γ_w (62.4 pcf; 9.8 kN/m^3) are the unit weights of the drainage sand and water, respectively, h_w is the water level above the failure surface, and h is the depth to the planar failure surface. If h_w is assumed to be zero, the FS of the 3H:1V (18.5 degrees) slope can be calculated as:

$$FS = \frac{\tan(\phi)}{\tan(18.5^0)} \tag{2}$$

A FS of unity corresponds to an interface friction angle of 18.5 degrees. As shown in Figure 4, a peak friction angle greater than 18.5 degrees corresponds to a compaction moisture content less than approximately 22.5%. If shear displacements are induced along the interface and a large displacement friction angle is mobilized, a compaction moisture content less than approximately 19.5% is required for a FS greater than unity. Also shown is a compaction moisture content less than 19% is required to develop a peak ϕ that would correspond to a FS of 1.2.

Figure 5 presents large-scale direct shear test results for the tensile geotextile interfaces. These results were used to facilitate design of the successful remedial measures that included installation of a tensile geotextile above the PVC geomembrane to reduce the shear stresses induced along the geomembrane/CCL interface. The remedial measures are discussed subsequently in detail. Drainage sand was placed above the tensile geotextile and the geotextile was placed in direct contact with the faille side of the PVC geomembrane. Before placing a tensile geotextile above the PVC geomembrane, it was important to assess whether or not the geotextile would create an interface that exhibits a lower shear strength than the geomembrane/CCL interface. If the tensile geotextile created a weaker interface, sliding would occur along this interface instead of the PVC geomembrane/CCL interface. It can be seen that the drainage sand/tensile geotextile interface exhibited a peak and large displacement friction angle of 25 to 24 degrees, respectively. These friction angles are significantly greater than the geomembrane/CCL interface friction angles so this interface was not deemed problematic. However, the PVC geomembrane/tensile geotextile interface exhibited a peak and large displacement friction angle of 16 to 15 degrees, respectively. These friction angles are not significantly greater than the geomembrane/CCL interface friction angles so this interface was potentially problematic. Even if sliding did not occur on the PVC geomembrane/tensile geotextile interface, the interface friction angle probably resulted in a greater tensile strength being required for stability.

3.3 Field Investigation

Following the October 1996 slide, a field and laboratory testing program was initiated by the owner to investigate the cause of the failure and mobilized shear strength parameters for designing the repaired slope. The field investigation consisted of the following activities:

1.) Measuring the moisture content of the compacted clay liner at the geomembrane interface and at 50 mm (2 inch) depth intervals before and after placement of the drainage sand and protective cover material. Measurements were made in areas adjacent to the June 1997 slide area and before the slides were repaired. The slides were repaired in July 1997. The only data available on the moisture content of the CCL in the slide areas before the failures were measured during the Quality Assurance/Quality Control (QA/QC) program for the compaction of the CCL in that area.

2.) Measuring temperature differential values for the PVC geomembrane/compacted clay interface before and after placement of the drainage sand and protective cover material. This was accomplished by determining the surface temperature of the PVC geomembrane exposed to direct sunlight and the temperature of the geomembrane already protected by the drainage sand layer in an area adjacent to the failed slope. Temperature differentials between the geomembrane exposed to and protected from the sunlight were determined using a Gilson MA-126 surface dial thermometer. Differential air temperature values were estimated for the 1996 and 1997 slide areas based on climatic data. After the 1996 slide it was believed that the large differential temperatures were the result of the day/night cycle. It was only after the 1997 slide that the differential temperatures developed on the geomembrane surface during the day were considered a potential contributor to the instability as discussed subsequently.

3.) Measuring the accumulative strain/displacement induced on the geomembrane interface during construction of the drainage sand and protective material layers. This was accomplished by establishing survey points, on a known grid configuration, on the surface of reinforcing geosynthetics installed in the slide area during the repair (tensile geotextile) and during construction of the liner in an adjacent area where a haul road was to be built (tensile geogrid). The measurements were made by installing grid points at known locations and at a predetermined distance ($\pm$3 m or $\pm$10 ft.) from each other. This surveyed grid was established at two different sections of both the 3H:1V and the 5H:1V slopes after the geogrid and the geotextile were installed. The same points were relocated after the sand drainage layer and protective cover were completed. The strain was calculated by determining the new distance between the points, after making adjustments to account for the overall displacement of the grid of surveyed points. Based on the new distance between points, the strain on the geosynthetics was assessed as $(L_f - L_0)/L_0$ where L_0 and L_f are the initial and final lengths of the tensile geosynthetics between selected grid points.

4 RESULTS OF FIELD INVESTIGATION

4.1 Moisture Content of Compacted Clay Liner

The moisture content of the compacted clay liner was measured at 2 inch (50 mm) depth intervals before and after placement of the drainage sand and protective cover material. The measurements were made at three locations adjacent to the 1997 slide area. With this effort, it was intended to quantify the moisture content increase due to construction activities. These three locations are labeled 4555, 4572, and 4600 in Table 2. Table 2 presents the moisture content at the geomembrane/CCL interface before and after placement of the drainage sand and protective cover material and the optimum moisture content for each of these three test locations. It can be seen that there was an increase in moisture content of 4–5% at each location during placement of the geomembrane, drainage sand, and protective cover material.

Table 2. Moisture Content of Compacted Clay at PVC Geomembrane/CCL Interface

Moisture Content	Sample Location No.		
	4555	4572	4600
Optimum	19.3%	19.0%	19.8%
Before Placement of drainage blanket and protective Material	16.5%	17.1%	14.9%
After CCL liner completion	20.6%	21.3%	20.3%
Moisture Content increase During Construction	4.1%	4.2%	5.4%

The moisture content of the compacted clay liner at 2 inch (50 mm) depth intervals before and after placement of the drainage sand and protective cover material was used in an effort to determine the source of the moisture content that caused the increase in the moisture of the CCL at the geomembrane interface shown in Table 2. Table 3 presents the moisture content versus depth profile at location 4600, which is typical of the other two locations. It can be seen that the moisture content increased at most of the sampling locations within the CCL. This suggests that the increase in moisture at the geomembrane/CCL interface was not solely caused by water exiting the CCL and migrating into this interface. The exact source of the additional moisture is not known. However, this case history does suggest that the moisture content at a geomembrane/CCL interface may increase during construction and thus interface testing probably should reflect a moisture content that is several percentage points above the as-compacted moisture content.

Table 3. Moisture Content Profile of Compacted Clay Liner at Location 4600

Depth (inches, mm)	Moisture Content (%)	
	Before Construction	After Construction
1, 25	14.9	20.3
3, 75	18.1	21.6
5, 125	17.9	22.1
7, 175	21.2	18.7
9, 225	20.9	23.5
11, 275	22.0	25.9
13, 325	24.3	24.3
15, 375	21.6	21.2

Figure 6 illustrates the impact of testing the PVC geomembrane/CCL interface under a wetted condition for a compaction water content of 19.6% (approximately optimum) and 21.3%. The pre-wetted interface tested at a moisture content of 19.6% exhibited a higher peak and large displacement friction angle than the interface tested at a moisture content of 21.3%. More importantly, the peak and large displacement friction angle relationships from Figure 4 are superimposed on the data in Figure 6. It can be seen that pre-wetting the interface with a spray bottle to simulate moisture buildup under the geomembrane can reduce the interface shear resistance. As discussed subsequently, the likelihood of moisture buildup at the interface suggests that the moisture content at a geomembrane/CCL interface may increase during construction and thus interface testing probably should reflect a moisture content that is several percentage points above the as-compacted moisture content and the interface probably should be pre-wetted.

4.2 Temperature Differentials

In the October 1996 slide area, the PVC geomembrane was exposed for up to two weeks before being covered. The placement of the drainage sand and protective cover material prior to the slide occurred between September 25 and October 25, 1996. During this month, air temperature records show that the daily differential temperature, difference between the maximum and minimum values, ranged from 8 to 31°F. In the section of the slope that slid in June 1997, the PVC geomembrane also was exposed to daily differential air temperatures of 8 to 29°F.

After the June 1997 failure, it was observed that the PVC geomembrane exposed to the mid-day sunlight would become hot, whereas the PVC geomembrane became significantly cooler within a relatively short period of time after placement of the moist drainage sand. At this time, it was decided to measure the temperatures on the surface of the exposed and covered PVC geomembrane in an area of the slope adjacent to the 1997 slide. The PVC geomembrane was covered by the drainage sand in the same fashion as the slide areas. From Table 4 it can be seen that there was a large temperature differential between the exposed and covered geomembrane. The temperature differential ranged from 22 to 32°F. It is believed that the higher surface temperature of the exposed geomembrane, greater than 110°F (43°C), heated up the underlying CCL causing water to vaporize. Water vapor then accumulated under the PVC geomembrane. With placement of the moist sand layer, the geomembrane cooled rapidly and resulted in the condensation of water under the geomembrane. It is anticipated that this condensation contributed to the increase in moisture content of the compacted clay at the geomembrane interface. Condensation is another factor that cause moisture content increase at the geomembrane/CCL interface during construction and thus interface testing should reflect a moisture content that is several percentage points above the as-compacted moisture content.

Table 4. Temperature of Exposed and Covered PVC Geomembrane

Measurement Location	Temperature of Exposed PVC Geomembrane (°F)	Temperature of Covered PVC Geomembrane (°F)	Temperature Differential (°F)
1	110	87	23
2	109	87	22
3	112	80	32
4	110	84	26

5 BACK-ANALYSIS OF THE SLIDES

At present, back-analyses of slope stability case histories are usually performed using a two-dimensional (2-D) slope stability method, which does not account for three-dimensional (3-D) end or shear forces. These end effects increase stability, and thus 2-D back-analyses yield unconservative estimates of the field shear strength because the end effects are incorporated in the back-calculated shear strength (Stark and Eid 1998). Since the width of the slope failures range from 15.2 to 22.9 m (50 to 75 feet) and the depth of the failure surface was only 0.90 m (3.0 feet), this was the required depth but the average depth for Phase B was 1.2 m (3.8 feet) deep, the 3-D effects were probably small. In addition, no toe buttress was installed for the drainage sand and protective cover material at the bottom of the slope. As a result, a 2-D analysis was used to back-calculate the mobilized interface strength.

The 2D slope stability analysis presented by Giroud et al. (1995) was used for the back-analysis. Equation (3) presents the limit equilibrium expression presented by Giroud et al. (1995) and it was used to back-calculate the mobilized PVC geomembrane/CCL interface friction angle. In the 2-D back-analysis, the conditions at the time of failure were estimated to include the PVC geomembrane being placed on the CCL with a moisture content of at least 24.4%, which increased from 21% at time of compaction, a slope inclination of 3H:1V, i.e., 18.4 degrees, and a maximum slope length of 23.5 m (77 feet). The PVC geomembrane was loaded with a 0.43 m (1.4 feet) average thickness of drainage sand and 0.73 m (2.4 feet) average thickness of protective cover material. The drainage sand and protective cover material exhibit total unit weights of 18.8 kN/m^3 (120 pcf) and 14.4 kN/m^3 (91.5 pcf), respectively. A weighted average total unit weight based on the thickness of the drainage sand and protective cover material of 16.0 kN/m^3 (102 pcf) was used in the analysis. As noted previously, the effective stress shear strength parameters of the drainage sand were estimated to be a cohesion of zero and a friction angle of 32 degrees. The effective stress shear strength parameters of the stabilized/solidified FGD waste product were estimated to be a cohesion of 1000 psf (48 kPa) and a friction angle of 15 degrees from EPRI (1984). A weighted average effective stress friction angle based on the thickness of the drainage sand and protective cover material was calculated to be 26 degrees and used in the analysis. Because of the granular nature of the drainage sand and the protective cover material, the effective stress cohesion was assumed to be zero. Table 5 summarizes the values used in the back analysis for each input parameter.

$$FS_B = \frac{\tan\delta_B}{\tan\beta} + \frac{a_B/\sin\beta}{\gamma_t(t-t_w)+\gamma_{sat}t_w} + \frac{\gamma_t(t-t_{w1})+\gamma_b t_{w1}}{\gamma_t(t-t_w)+\gamma_{sat}t_w}\left[\frac{t}{h}\right]\frac{\sin\phi}{2\sin\beta*\cos\beta*\cos(\beta+\phi)} +$$

$$\frac{c*t}{(\gamma_t(t-t_w)+\gamma_{sat}t_w)h}\left[\frac{\cos\phi}{2\sin\beta*\cos(\beta+\phi)}\right] + \frac{T}{(\gamma_t(t-t_w)+\gamma_{sat}t_w)h} \tag{3}$$

where:

FS_B = factor of safety if slip surface is below the geomembrane (dimensionless)

β = Slope angle (degrees)

h = height of slope (ft)

t = thickness of the soil layer above geomembrane (ft)

t_w = depth or thickness of water above geomembrane (ft)

t_{w1} = depth or thickness of water above geomembrane in toe area (ft)

δ_B = interface friction angle along a slip surface located below the geomembrane (degrees) [varied until 17.1° which yielded a factor of safety of 1.0]

a_B = interface adhesion along a slip surface located below the geomembrane (psf)

ϕ = angle of internal friction angle of soil above geomembrane (degrees) c = cohesion of soil above the geomembrane (psf)

γ_t = total unit weight of soil above geomembrane (pcf)

γ_{sat} = saturated unit weight of soil above geomembrane (pcf)

γ_b = buoyant unit weight of soil (pcf)

T = geosynthetic tension (lbs/ft)

Table 5. Summary of Input Parameters for Back-Analysis

Input Parameter	Value
FS_B	1.0
β	18.4 degrees
H	23.5 m [77 ft]
T	1.2 m [3.8 ft]
t_w	0 m [0 ft]
t_{wl}	0 m [0 ft]
δ_B	Varied
a_B	0 kPa [0 psf]
ϕ	26 degrees
C	0 kPa [0 psf]
γ_t	31.1 kN/m^3 [102 pcf]
γ_{sat}	41.5 kN/m^3 [136 pcf]
γ_b	22.5 kN/m^3 [73.6 pcf]
T	1488.5 kg/m [1000 lbs/ft]

No rain events were reported in conjunction with this failure so the depths of water on the geomembrane, i.e., t_w and t_{w1}, were assumed to be negligible. After cracking developed in the waste material and underlying drainage sand, the PVC geomembrane ruptured. Quality control testing of this material showed that the geomembrane has a tensile break strength in the machine direction of approximately 1488.5 kg/m (1000 lbs/ft). In the 2-D back-analysis the shear resistance of the PVC geomembrane/CCL interface was varied until a factor of safety of unity was obtained for the conditions presented in Table 5. This was accomplished by assigning the interface cohesion a value of zero and varying the interface friction angle. A factor of safety of unity was obtained for a PVC geomembrane/CCL interface friction angle of approximately 17 degrees.

As mentioned previously, the PVC geomembrane/CCL interface was tested at a normal stress of 7.2 kPa (150 psf) using a large-scale direct shear device after the October 1996 slide. It can be seen from Figure 4 that the back-calculated interface peak friction angle of approximately 17 degrees corresponds to a compaction moisture content of about 25%, whereas a large displacement friction angle of approximately 17 degrees corresponds to a moisture content of about 22.5%. This was initially perplexing because the compaction moisture content of the CCL was 21.5%, which corresponds to a peak interface friction angle of about 19 degrees. After the October 1996 failure it was noted that a thin layer of wet clay (approximately 10mm thick) had formed on the surface of the CCL which had a moisture content of 27%. After the June 1997 failure, it was observed that a similar thin layer of wet clay (approximately 5mm thick) had also formed and it had a moisture content of 22.5% even though the compaction moisture content in this area had been 19.5%. This comparison suggests that moisture condensed under the PVC geomembrane or was drawn to the geomembrane/CCL interface resulting in the moisture content increasing at the surface of the CCL. This increase in moisture content reduced the peak interface friction angle from 19 degrees (moisture content of 21.5%) to less than 17 degrees (moisture content of 25%) as the final moisture content at the geomembrane/clay interface rose to 27% (June 1996 failure). This analysis also suggest that an interface angle of approximately 17 degrees (moisture content 22.5%) was mobilized at the time of the June 1997 failure. Shear displacements were probably introduced during drainage sand and cover material placement and the use of the temporary haul roads. As a post-peak interface shear resistance was being mobilized, the PVC geomembrane began to stretch downslope. As the geomembrane was stretching, additional shear displacements were probably induced along the geomembrane/CCL interface and could have caused a post-peak shear strength to develop until the PVC geomembrane finally ruptured.

Based on field observations and the back-analysis, it is concluded that the October 1996 and June 1997 slides were caused by an increase in moisture content at the PVC geomembrane/CCL interface. This increase in moisture content reduced the available shear resistance of the PVC geomembrane/CCL interface. A minor factor may have been the introduction of shear displacements along the PVC geomembrane/CCL interface due to placement of the drainage sand and protective cover material, construction and use of the temporary haul roads, and the stretching of the PVC geomembrane prior to rupture.

6 REMEDIAL MEASURES AND SLOPE RECONSTRUCTION

The measures implemented to remediate the two slides and for the construction of the remaining landfill were designed to primarily address the following two issues: 1) a moisture content at the PVC geomembrane/CCL interface that exceeds the compaction value; and, 2) construction induced stresses and displacements along the PVC geomembrane /CCL interface. Each issue is discussed in the following paragraphs.

The increase in moisture content at the PVC geomembrane/CCL interface was addressed by field QA/QC personnel by closely monitoring the compaction moisture content at the surface of the CCL so the moisture content was at or near the optimum value (19.3%). A "target" moisture content of approximately 19.3% at the surface of the CCL reduced the possibility of a large reduction in the interface shear resistance due to the anticipated increase in moisture content during placement of the drainage sand and protective cover material.

Construction induced stresses and displacements were addressed by introducing tensile geosynthetics at the PVC geomembrane/drainage sand interface. The main function of the tensile geosynthetics was to resist tensile forces that developed when the sliding forces exceeded the interface shear resistance. The tensile geosynthetics used are a woven geotextile manufactured by T.C. Mirafi (700X) and a bi-axial geogrid manufactured by Tensar (BX1200). The geotextile and geogrid were used in different locations of the 3H:1V slope depending on the construction sequence, slope length, and whether or not a temporary haul road would be constructed on the slope.

Figure 5 presents large-scale direct shear test results for the tensile geotextile interfaces. These results yield a peak interface friction angle of 24 and 15 degrees for the drainage sand/tensile geotextile and PVC geomembrane/tensile geotextile interfaces, respectively. The allowable tensile force for the woven geotextile is 3036 lbs./ft (16% strain) based on the stress/strain information provided by the manufacturer. This value reflects a reduction factor for creep, installation damage, and durability.

6.1 New Critical Interace

Factors of safety were calculated for the during construction condition using the infinite slope stability method, see Equation (1). This method does not incorporate the influence of the stiffness of each component, as does the method presented by Long et al. (1994). A summary of the factors of safety calculated for the PVC geomembrane/CCL interface using the infinite slope analysis is shown in Table 6.

Table 6. Summary of Calculated Factors of Safety at PVC geomembrane/CCL interface

Interface	Interface Friction Angle (degrees)	Factor of Safety
Drainage sand/tensile geotextile	24	1.33
Faille PVC geomembrane/tensile geotextile	15	0.75
Smooth PVC geomembrane/CCL	21 (peak)	1.15
Smooth PVC geomembrane/CCL	19 (large displacement)	1.03

These factors of safety indicate that the weakest interface is now the PVC geomembrane/tensile geotextile interface. As a result, unbalanced forces on this interface imposed during construction and use of the temporary haul roads could result in tensile stresses on the PVC geomembrane/tensile geotextile interface. It is important to note the remedial measures can result in a new critical interface and designers should be cognizant of this possibility.

To incorporate the stiffness of the drainage sand and protective cover material above the tensile geotextile, the Simple-Composite-Column (SCC) Formulation presented by Long, et al. (1995) was used to evaluate global stability. The axial stiffness in compression, but not in tension, is modeled for the soil layers while the tensile geosynthetic exhibits tensile stiffness but no compression stiffness. This method was found to be particularly applicable to this case history because the cover soils include the 0.3 m (1 ft) thick drainage sand and the 0.61 m (2.0 ft) thick FGD layer. The confined sand and compacted FGD exhibit significant compression stiffness. In fact, the stiffness of the FGD increases with time and is characterized by a minimum unconfined compression strength after 28 days of 620.5 kPa (12,960 psf). Using the SCC formulation, the minimum factor of safety against sliding along the PVC geomembrane/tensile geotextile interface after completion of construction increased from

0.75 to 2.1. The applicability of the SCC formulation, i.e., stiffness of the cover materials, was verified because the remediate slope has performed satisfactorily.

With the implementation of these two remedial measures (careful monitoring of the CCL moisture and installation of tensile geosynthetics), the two slide areas were successfully repaired. These same measures were implemented on the remaining interior slopes of the landfill that were steeper than 4H:1V in all of the landfill phases. No signs of slippage or large deformation have been observed since June 1997. During construction of Phase C in Valley 3, blasting operations were performed approximately 30 m (100 feet) from the edge of the 3H:1V slope in Phase B a few weeks after placement of the protective cover material. Peak horizontal ground surface accelerations at the edge of the slope were determined to be as high as 0.13g. No signs of distress or large strains were observed on this or any of the slopes in Phase B, Valley 2. This is significant because Stark et al. (2000) conclude that blasting can have an adverse impact on the stability of marginally stable slopes. Therefore, it may be concluded that the slope was not marginally stable because the blasting did not have an adverse impact on stability.

All of the tensile forces acting on the system are ultimately transferred to the anchor trench. For this repair, an anchor trench analysis was conducted as outlined by Koerner (1990). The factor of safety against anchor trench pullout from the imposed tensile forces was calculated to be approximately two.

6.2 Tensile Strain Along Geomembrane/CCL Interface

In an effort to determine the magnitude of stress and displacement that were being imposed on the PVC geomembrane/CCL interface, survey points were permanently marked on the tensile geotextile and geogrid reinforcement that were placed immediately over the PVC geomembrane in slope sections adjacent to the June 1997 failure. Grids of survey points were placed on the geotextile and geogrid used on 3H:1V and 5H:1V sections of the internal slope of the landfill. The points were surveyed before and after construction of the composite liner in these slope sections. A summary of the tensile strain measurements obtained is presented in Table 7. The strain in the geosynthetics was assessed using the expression $(L_f - L_o)/L_o$ presented earlier.

Table 7. Estimated Construction Induced Strains in Tensile Reinforcement

Reinforcing Material	Tensile Strain as a Function of Slope Inclination	
	5H:1V	3H:1V
Woven Geotextile - Mirafi 700 X	1.7%	2.2%
Geogrid - Tensar BX 1200	0.8% to 0.9%	1.46%

Based on the stress-strain relationships developed for each tensile geosynthetic, measured tensile strains correspond to tensile stresses of 117 to 172 kPa (2448 to 3600 psf) the 5H:1V and 3H:1V slopes, respectively. On slopes where no geogrid or tensile geotextile was used, this level of stress may induce tensile strains on the order of 9% and 26% in the PVC geomembrane on the 5H:1V and 3H:1V slopes, respectively. This level of strain is transferred into the clay/geomembrane interface and the interface resistance must accommodate the induced stress otherwise failure will occur. However, since failure has not occurred, the interface shear resistance has been sufficient to maintain slope stability.

7 CONCLUSIONS

This paper describes the two slope failures, failure mechanism, subsurface investigation, engineering properties of the stabilized/solidified FGD product and geomembrane/CCL interface, back-analysis and cause of the slides, slope reconstruction, and lessons learned. Construction conditions experienced during the installation of a landfill geocomposite liner system lead to two failures of a 3H:1V internal slope. The landfill was being developed for final disposal of Flue Gas Desulfurization byproduct material. The slides occurred in two different areas of the 3H:1V slope and encompassed 10,500 and 20,235 m^2 (2.6 and 5 acres) in the Fall of 1996 and Summer of 1997, respectively. The failures developed by sliding along the geomembrane/compacted clay interface during or shortly after placement of the protective cover material over the drainage sand layer. Laboratory direct shear tests revealed that the shear strength of this interface is sensitive to the moisture content of the exposed compacted clay. The direct shear tests revealed reductions in the available shear strength of 35% and 37% for the peak and residual values as the compacted clay moisture content increased 8 percentage points. In addition, shear stresses induced by the construction activities probably resulted in shear displacements along the interface, which in some cases, were large enough to lower the available shear resistance of the geomembrane/compacted clay interface to a post-peak value.

Reconstruction of the slopes was performed by including tensile geosynthetics immediately above the PVC geomembrane to resist construction related tensile forces. Based upon the understanding of the mechanisms that affect the shear resistance of the PVC geomembrane/compacted clay interface, tensile geosynthetics were incorporated in the construction of the remaining slopes of the landfill that have slope inclinations steeper than 4H:1V to minimize their potential for sliding. Since the successful completion of the remedial measures, Phases B and C of the landfill were completed without any signs of instability even when blasting operations induced peak accelerations as high as 0.13g on the slopes of Valley B.

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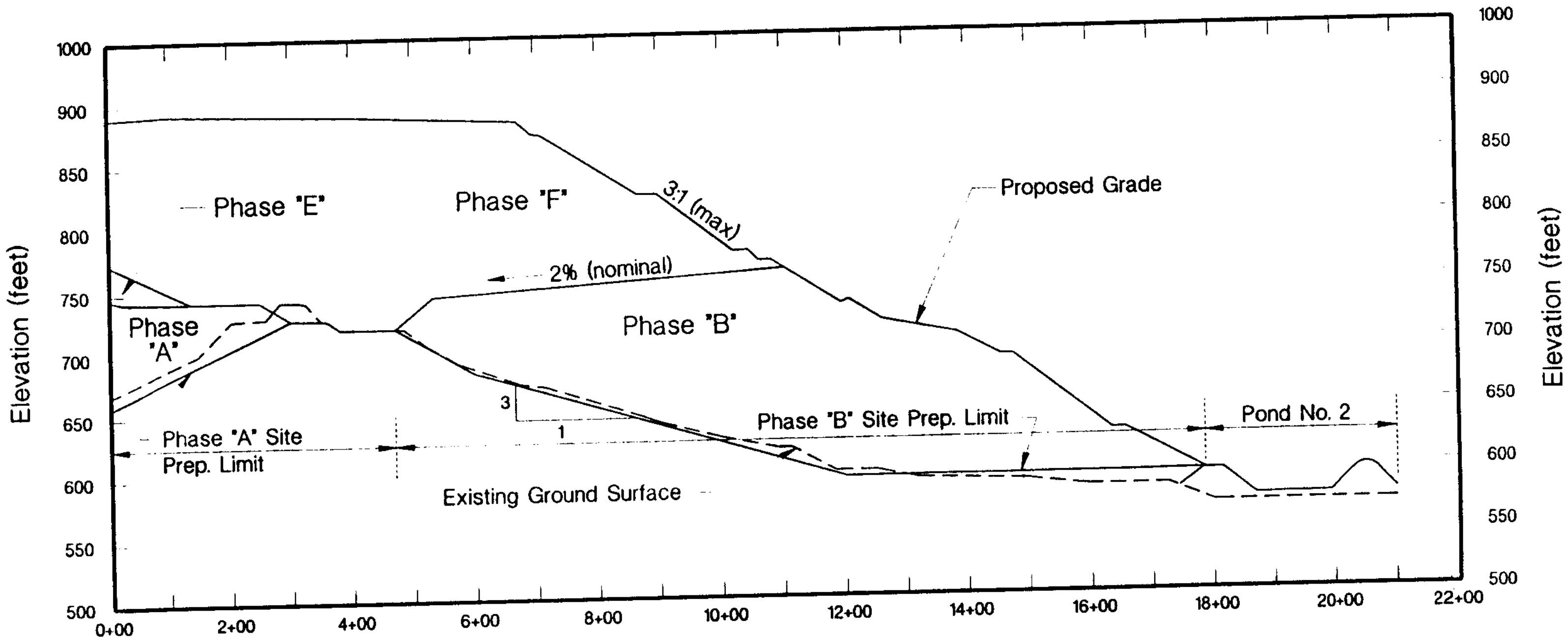

Figure 1. Typical Cross-Section Through Phase B in Valley 2

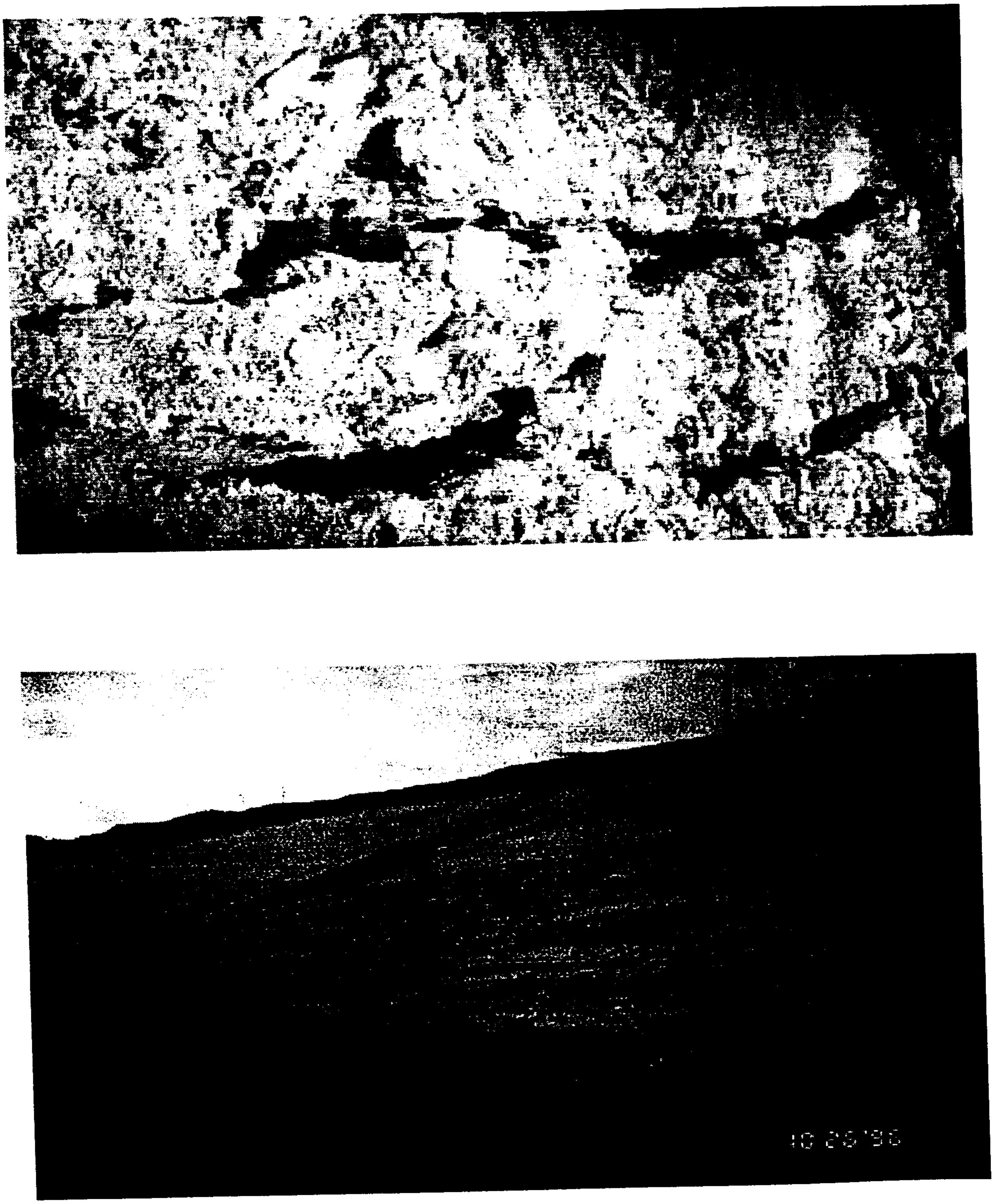

Figure 2. (a) Slope cracking and (b) tear observed in PVC geomembrane s after October 1996 slide

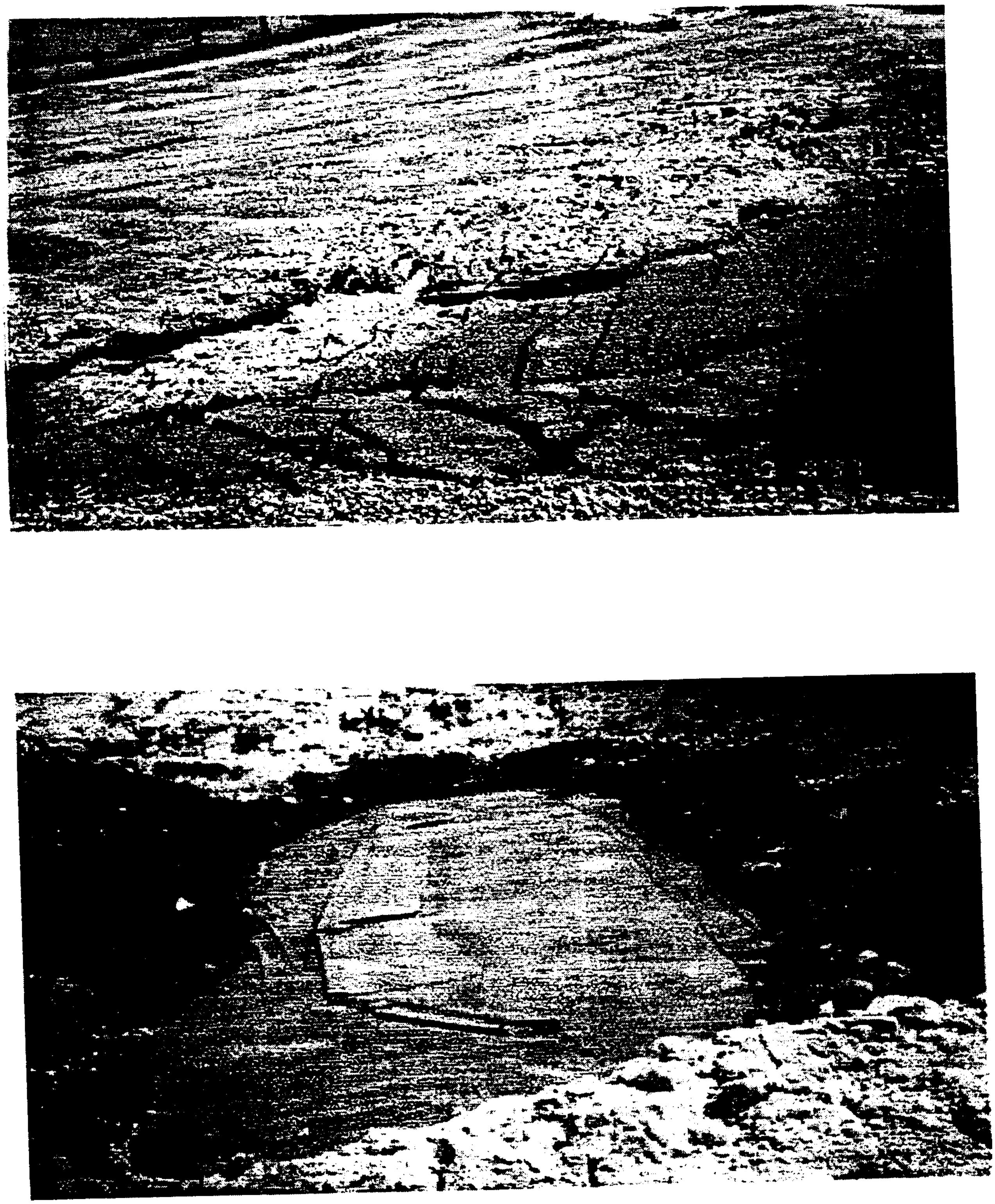

Figure 3. (a) Slope cracking and (b) tear observed in PVC geomembrane after June 1997 slide

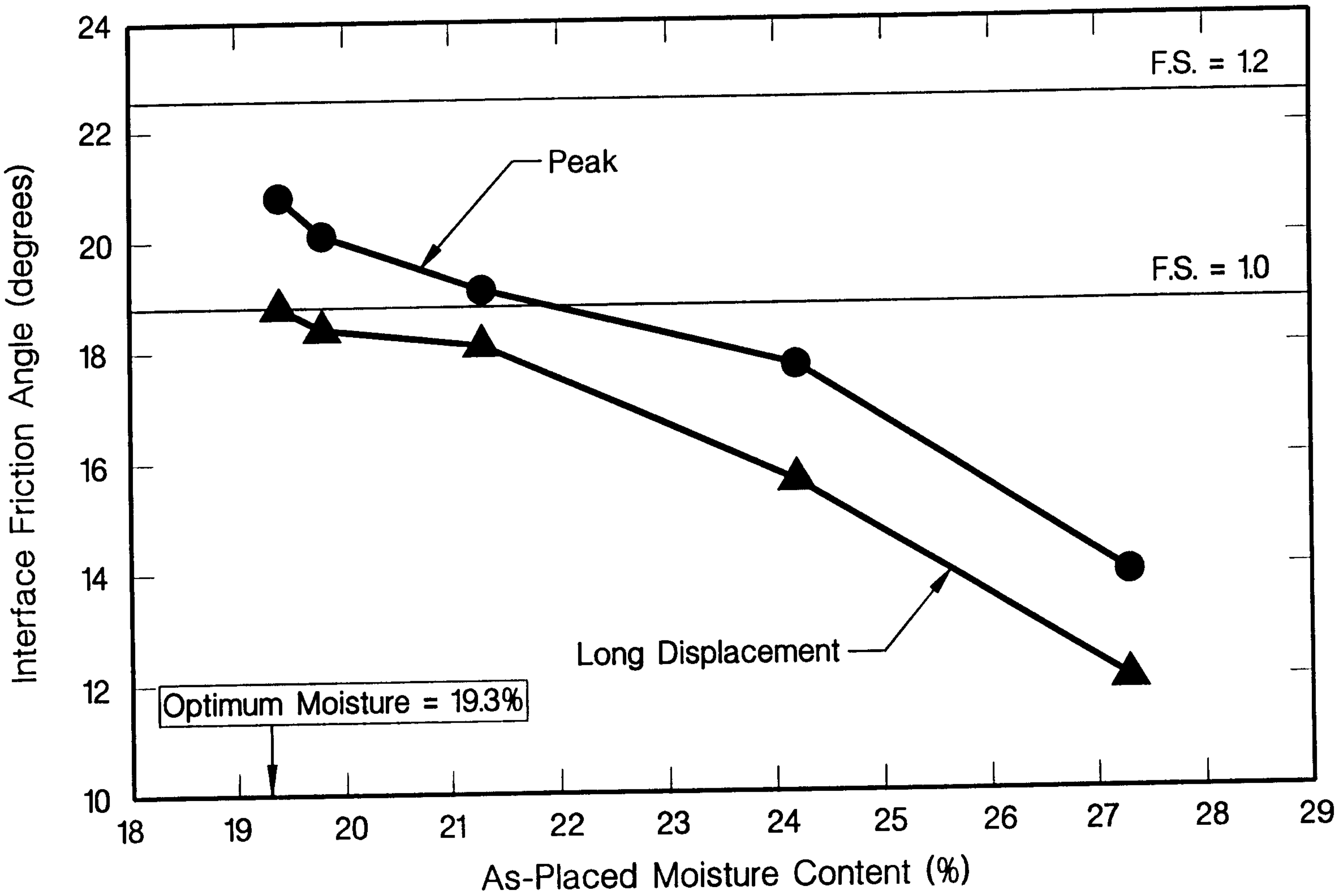

Figure 4. Effect of compaction moisture content on PVC geomembrane/GCL interface friction angle at a normal stress of 7.2 kPa

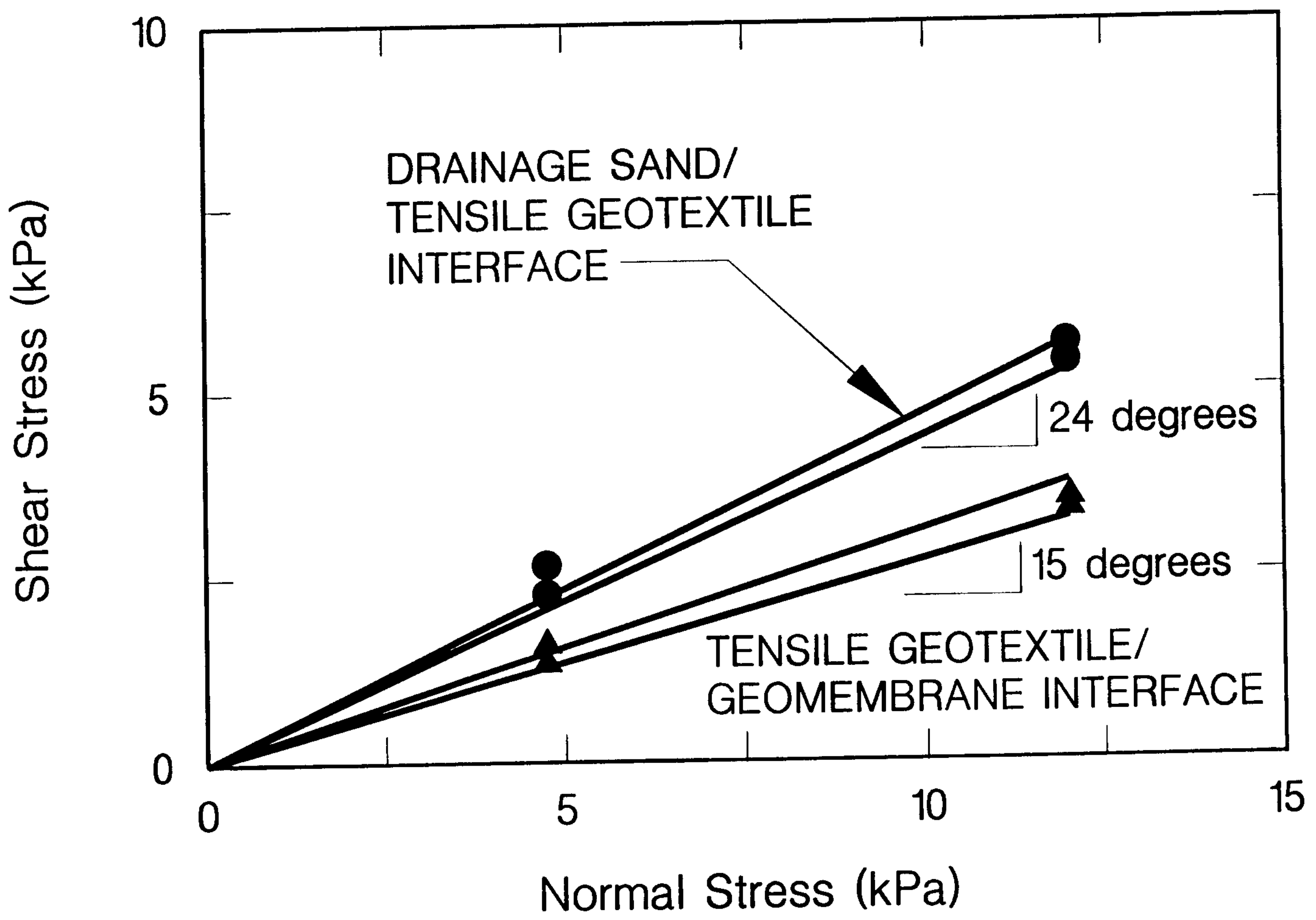

Figure 5. Shear resistance of tensile geotextile interfaces

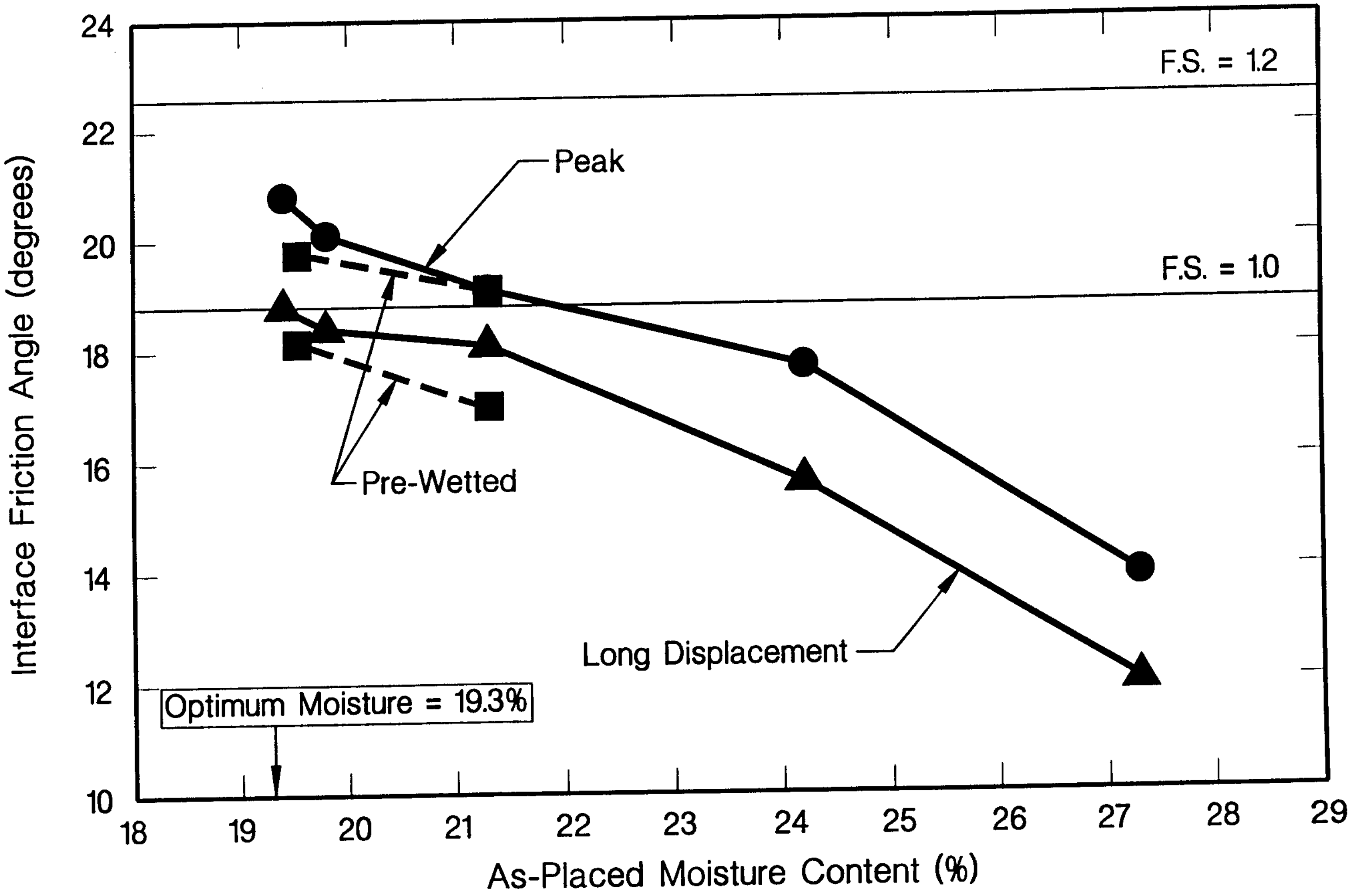

Figure 6. Effect of pre-wetting PVC geomembrane/GCL interface on friction angle at a normal stress of 7.2 kPa