Case history of liner veneer instability

P. Amaya[1], B. Queen[2], T. D. Stark[3] and H. Choi[4]

[1]*Civil Engineer, Geotechnical Engineering Section, American Electric Power Service Corporation, 1 Riverside Plaza Columbus, OH 43215–2373, USA, Telephone: +1 614 716 2926, Telefax: +1 614 716 2963, E-mail: pjamaya@aep.com*
[2]*Environmental Specialist, South East District Office, Division of Solid and Infectious Waste Management, Ohio EPA, 2195 Front Street, Logan, OH 43138, USA, Telephone: +1 740 385 8501, Telefax: +1 740 385 6490, E-mail: brian.queen@epa.state.oh.us*
[3]*Professor, Department of Civil and Environmental Engineering, The University of Illinois, 2217 NCEL, 205 N. Mathews Ave., Urbana, IL 61801-2352, USA, Telephone: +1 217 333 7394, Telefax: +1 217 333 9464, E-mail: tstark@uiuc.edu*
[4]*Assistant Professor, Department of Civil and Environmental Engineering., Korea University, Anam-Dong, Seongbuk-Gu, Seoul 136-713, South Korea, Telephone: +82 2 3290 3326, Telefax: +82 2 928 7565, E-mail: hchoi2@korea.ac.kr*

Received 15 March 2005, revised 15 September 2005, accepted 31 October 2005

ABSTRACT: Construction conditions experienced during installation of a landfill geocomposite liner system led to two slides on a 3H:1V slope during construction. The landfill was being developed for final disposal of flue gas desulfurization by-product material (FGD). The slides occurred in two different areas of the 3H:1V slope and encompass 10,500 and 20,235 m^2 in the Fall of 1996 and Summer of 1997, respectively. The slides developed by movement along the PVC geomembrane/compacted clay liner interface during or shortly after placement of the protective FGD 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 liner. The direct shear tests revealed reductions in the available shear strength of 35% and 37% for the peak and large displacement values as the compacted clay liner moisture content increased by 8 percentage points. In addition, shear stresses induced by surface traffic activities might result 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 or large displacement value. This paper discusses the slides, direct shear testing, slope stability analyses used to evaluate the mobilized interface shear strength parameters, the relationship between moisture content and geomembrane/compacted clay liner interface strength, and the effect of shear displacement on the mobilized shear strength.

KEYWORDS: Geosynthetics, PVC geomembrane, Compacted clay liner, Direct shear test, Shear strength, Slope stability

REFERENCE: Amaya, P., Queen, B., Stark, T. D. & Choi, H. (2006). Case history of liner veneer instability. *Geosynthetics International*, **13**, No. 1, 36–46

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 desulfurization (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/solidified 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 of compacted clay liner (CCL) with a hydraulic conductivity not to exceed 1.0×10^{-9} m/s, 0.75 mm (0.03 in) thick poly(vinyl chloride) (PVC) geomembrane, a 0.3 m thick layer of drainage sand, and 0.61 m of FGD used as a protective cover material. The FGD protective cover material was placed immediately above the drainage sand with no separating geotextile.

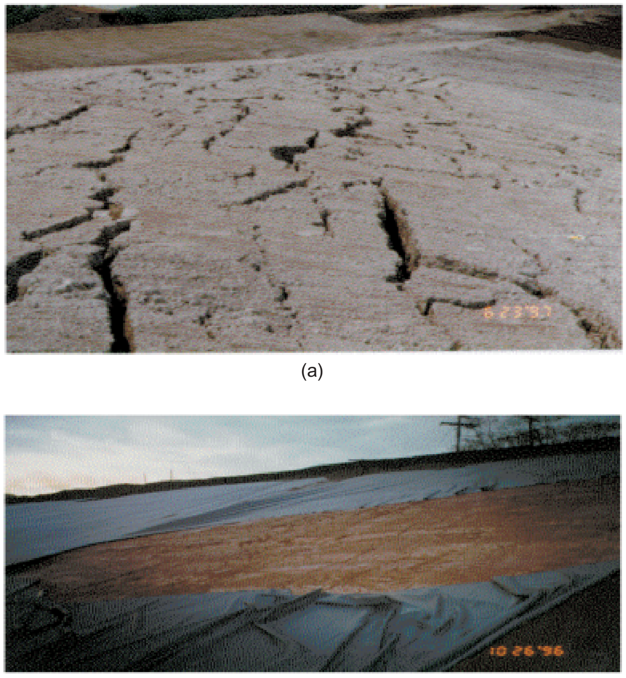
The FGD landfill site is located about 2.1 km northwest

1072-6349 © 2006 *Thomas Telford Ltd*

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 and consists of three 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 m^3. 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 slides encompassing areas of approximately 10,500 and 20,235 m^2 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. Figure 1 shows 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, and the subsequent Phase F filling will result in a depth of 46 m for a maximum waste depth of 92 m in Valley 2. This paper describes the two slides that occurred during Phase B construction, subsurface investigation, failure mechanism, engineering properties of the stabilized/solidified FGD product and PVC geomembrane/compacted clay liner (CCL) interface, back-analysis of the slides, and lessons learned.

2. DESCRIPTION OF SLIDES AND FAILURE MECHANISM

After subgrade preparation for placement of the composite liner system in Phase B, the slope inclinations ranged from 3H:1V to 5H:1V (see Figure 1). In late October 1996, shortly after placement of the drainage sand and protective FGD cover material, a network of cracks in the protective FGD cover and drainage sand was observed over approximately 10,500 m^2 of the slope. The cracks were located in the 3H:1V portions of the slope. The parallel cracks extended across the slope and were about 15 m down the slope from the crest of Valley B, as shown in Figure 2a. The width of the cracks ranged from 12 mm to 125 mm but did not show any significant vertical offset. This lack of vertical offset was caused by translational sliding occurring above the stiff CCL. Investigations at the crack locations revealed that the 0.75 mm (0.03 in) thick PVC geomembrane was in significant tension, which resulted in localized tearing of the geomembrane. Figure 2b illustrates the torn PVC geomembrane in the October 1996 slide area. During the investigation, it was observed that a thick, wet moisture film (approximately 10 mm in thickness) was present at the surface of the compacted clay immediately below 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) optimum compac-



(a)

(b)

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

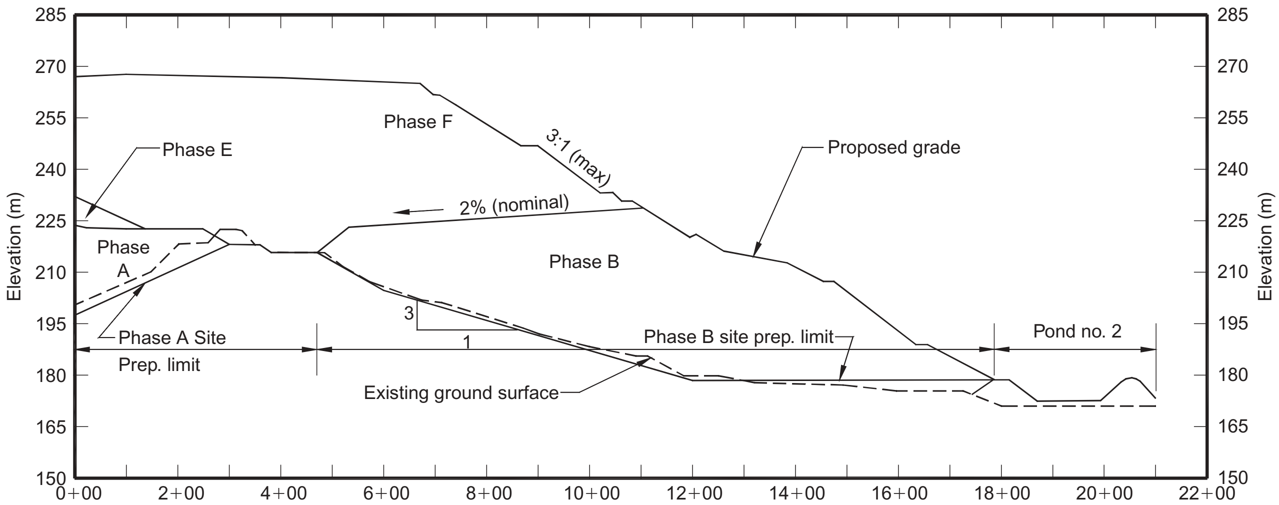


Figure 1. Typical cross-section through Phase B in Valley 2

tion moisture content of the CCL is 19.3%. Post-slide investigations revealed that the surface of the clay had a moisture content of 27%, whereas the bottom of the CCL had a moisture content equal to 21%.

In June 1997, another section of 3H:1V slope in Phase B slid. Figure 3 illustrates the slope cracking and condition of the PVC geomembrane in the June 1997 slide area. Construction records indicate that the CCL 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 CCL, and the moisture content of the CCL was monitored before the drainage sand was placed. The moisture content was monitored with the PVC geomembrane in place by cutting out a square section of the PVC geomembrane at a few locations and patching it after a sample of the CCL 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 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 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 possible that the shear stresses induced by traffic on these roads contributed to the 1997 slide. The observed network of cracking is similar to the network observed for the October 1996 slide, as shown in Figure 3a. The parallel cracks extended across the slope and were also about 15 m down the slope from the crest of Valley B. The width of the cracks ranged from 12 mm to 125 mm but also did not show any significant vertical offset. This lack of vertical offset again was caused by translational sliding occurring above the stiff CCL.

(a)

(b)

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

A significant finding during the 1997 slide investigation is that the average moisture content of the CCL 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 slides occurred as a result of movement 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 downslope. In some locations the geomembrane tore, resulting in the slide surface migrating up through the drainage sand and protective FGD cover material. The slides were translational in nature owing to the compacted and stiff nature of the underlying compacted clay liner and existing terrain.

3. SLIDE 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. The alluvial soils are underlain by approximately 0.6 to 1.5 m of very-stiff residual brown silty clay. Bedrock was encountered at depths ranging from 4.9 to 17.7 m. The undrained shear strength of the brown and grey silty clay was measured using unconfined compression tests on undisturbed specimens. The silty clays exhibit increasing undrained shear strength with depth, as shown in Table 1 and are separated into an upper and lower layer based on the undrained shear strength. Drained direct shear tests were also conducted, and the effective stress friction angle ranges from 30° to 32°° for these layers. A review of these high shear strength parameters led to the conclusion that sliding did not occur in the foundation soils underlying the 3H:1V slope.

The effective stress shear strength parameters of the stabilized/solidified FGD waste product were estimated to be a cohesion of 48 kPa and a friction angle of 15°. These

Table 1. Shear strength parameters for foundation soils in Phase B

Valley 2 stratum	Undrained shear strength (kPa)	Drained friction angle (degrees)
'Upper' silty clay	86	30
'Lower' silty clay	120	32

parameters were estimated from test results published by the Electric Power Research Institute (EPRI, 1984) on samples of the FGD sludge from the American Electric Power Plant in Conesville, Ohio. The FGD handling at the Conesville plant is similar to that at the Gavin Power Plant, and thus the material properties of the FGD from the Conesville Plant are comparable to those of the FGD at the Gavin 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 (USCS). The effective stress shear strength parameters of the drainage sand were also estimated to be cohesion of 0 kPa and a friction angle of 32°.

The CCL 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 USCS. The compaction specification for the CCL is a standard Proctor (ASTM D 698) relative compaction of 95% and a compaction water content that ranges from 0% to 3% wet of optimum. Total stress shear strength parameters for the compacted clay liner were estimated to be a cohesion of 96 kPa and a friction angle of 0° from the NAVFAC (1982) DM-7.1 manual. Evaluation of potential slide surfaces through the compacted clay liner indicates a high factor of safety that is not 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 CCL under the 3H: 1V slope indicate that sliding probably did not occur through these materials. As a result, the slide investigation focused on the shear behavior of the PVC geomembrane/CCL interface.

3.2. Geosynthetics testing

Following the October 1996 slide, direct shear tests were performed in accordance with standard test method ASTM D 5321 to evaluate the shear strength of the PVC geomembrane/CCL 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. The normal stress is applied pneumatically and the shear displacement is limited to 50 mm. As a result, peak and large displacement (not residual) interface shear strengths are measured in these tests. In this research, a shear rate of 1 mm/min (0.04 in/min) is applied to the direct shear test. This value of shearing rate is recommended in ASTM D 5321 for free-draining interfaces. The reason for selecting the shearing rate of 1 mm/min for the PVC geomembrane/CCL interface is that the CCL being compacted at the water content ranging from 19.3% to 27% is not fully saturated. Thus shear-induced excessive porewater pressures may not be significant during the direct shear test and are not considered in this paper. However, it is recommended to investigate the occurrence of shear-induced excessive porewater pressures at the PVC geomembrane/CCL interface in the future.

The 0.75 mm (0.03 in) 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 CCL. Interface testing is being conducted at the University of Illinois to determine whether the faille side reduces the impact of moisture collection at the PVC geomembrane/CCL interface compared with the smooth side. It is conceivable that the faille finish may be able to channel some of the moisture away from the interface compared with 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. A normal stress of 7.2 kPa is smaller than the maximum normal stress (σ'_n) exerted by the drainage sand and protective FGD cover material in the slide area, which is approximately 13.5 kPa. A normal stress of 13.5 kPa corresponds a 0.3 m thick drainage sand (total unit weight 18.8 kN/m^3) and a 0.61 m thick FGD waste (total unit weight 14.4 kN/m^3) on a 3H:1V slope as shown below:

$$\sigma'_n = [18.8(\mathrm{kN/m^3}) \times 0.3(\mathrm{m}) + 14.4(\mathrm{kN/m^3}) \times 0.61(\mathrm{m})] \times \cos(18.4°) = 13.5\mathrm{kPa} \quad (1)$$

In general, shear strength parameters should be evaluated using tests conducted for the range of normal stresses encountered in the field. However, in this case history a normal stress of 7.2 kPa corresponds to the average normal stress on the PVC geomembrane/CCL interface, and the range of normal stress is 0 to 13.5 kPa. To accelerate the failure investigation, a normal stress of 7.2 kPa was used in laboratory testing to evaluate the effect of compaction moisture content and pre-wetting on the PVC geomembrane/CCL interface. Figure 4 shows that the peak interface friction angle (δ) decreases from approximately 21° to about 14° with an increase in compaction moisture content from 19.8% to 27.3%. As shown, the optimum moisture content for the standard

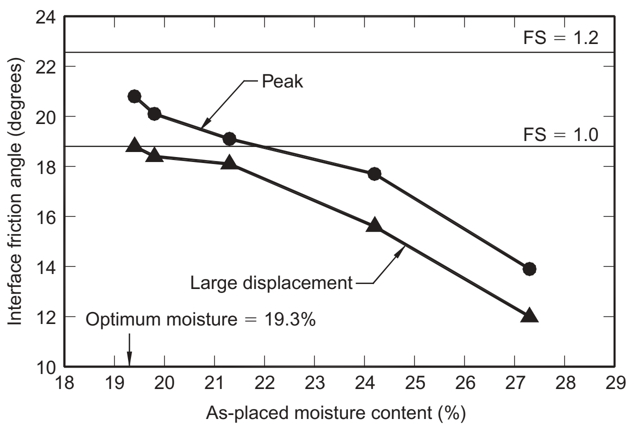


Figure 4. Effect of compaction moisture content on PVC geomembrane/CCL interface friction angle at a normal stress of 7.2 kPa

Proctor compactive effort is 19.3%. Figure 4 also shows that the interface exhibited a post-peak strength loss of approximately 10% that is essentially constant with increasing compaction moisture content.

The large displacement interface friction angle (δ_{LD}) decreases from approximately 18° to about 12° with an increase in compaction moisture content from 19.8% to 27.3%. The small post-peak interface strength loss of the smooth side of the PVC geomembrane is attributed to the large and intimate contact area between the geomembrane and CCL and the high flexibility of the PVC geomembrane (Hillman and Stark 2001).

Figure 4 also presents the results of an infinite slope analysis to demonstrate the importance of compaction moisture content and interface friction angle 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:

$$\mathrm{FS} = \left[\frac{1}{\tan\beta} - r_{\mathrm{u}}\left(\frac{1}{\tan\beta} + \tan\beta\right)\right]\tan\delta \tag{2}$$

where β = slope angle (degrees), δ = interface friction angle (degrees), and r_{u} = pore pressure ratio ($= u/\gamma z$) defined by Bishop and Morgenstern (1960). If pore pressure on the slip surface is zero, the pore pressure ratio (r_{u}) becomes zero, and thus the FS of the 3H:1V (18.4°) slope can be calculated as

$$\mathrm{FS} = \frac{\tan\delta}{\tan\beta} = \frac{\tan\delta}{\tan(18.4^\circ)} \tag{3}$$

An FS of unity corresponds to an interface friction angle of 18.4° for a 3H:1V slope. As shown in Figure 4, a peak friction angle greater than 18.4° corresponds to a compaction moisture content less than approximately 22%. 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 an FS greater than unity. Also shown is a compaction moisture content less than 19% is required to develop a peak interface friction angle that would correspond to an FS of 1.2. A compaction moisture content less than 19% would be less than the optimum moisture content, which may preclude meeting the hydraulic conductivity criterion of not exceeding 1.0×10^{-9} m/s. Thus relationships similar to those in Figure 4 can be developed for a site and used to establish the range of water and dry unit weight, i.e. the compaction window, to achieve the hydraulic conductivity and interface shear strength requirements as suggested by Daniel and Wu (1993).

4. FIELD INVESTIGATION

4.1. General

Following the October 1996 slide, a field and laboratory testing program was initiated by the owner to investigate the cause of the slide 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 CCL at the geomembrane interface and at 50 mm depth intervals before and after placement of the drainage sand and protective FGD cover material. Measurements were made in areas adjacent to the June 1997 slide area and before the slides were repaired. Both of the slides were repaired in July 1997. The only data available on the moisture content of the CCL in the slide areas before the slides 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/CCL 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 (tensile geotextile) installed in the slide area during the repair and during construction of the liner in an adjacent area where a haul road was to be built using a tensile geogrid. The measurements were made by installing grid points at known locations and at a predetermined distance (+3 m) 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 FGD 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 in the geosynthetics was assessed as $(L_{\mathrm{f}} - L_0)/L_0$, where L_0 and L_{f} are the initial and final lengths of the tensile geosynthetics between selected grid points. The engineering tensile properties of the 0.75 mm (0.03 in) thick PVC geomembrane are listed in the PGI-1104 specification (i.e. strength at break = 12.8 kN/m, elongation at break = 380%, and

modulus at 100% elongation = 5.6 kN/m). The shear stress exerted by the drainage sand and protective FGD can be calculated as follows:

$$\tau = [18.8(\text{kN/m}^3) \times 0.3(\text{m}) + 14.4(\text{kN/m}^3) \times 0.61(\text{m})] \times \sin(18.4°) = 4.55(\text{kN/m}^2) \quad (4)$$

The mobilized shear force per unit width can be obtained by multiplying the shear stress (4.55 kN/m^2) by the length of a sliding mass. However, the length of a sliding mass is not documented in this case. Assuming the length of the sliding mass is 3 m, the mobilized shear force per unit width will be 13.65 kN/m, which is slightly greater than the strength of the 0.75 mm thick PVC geomembrane (12.8 kN/m). In this case, localized tearing of the PVC geomembrane occurs.

4.2. Moisture content of compacted clay liner

The moisture content of the CCL was measured at 50 mm depth intervals before and after placement of the drainage sand and protective FGD 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 at the PVC geomembrane/CCL interface 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 FGD 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.

The moisture content of the CCL at 50 mm depth intervals before and after placement of the drainage sand and protective FGD cover material was used in an effort to determine the source of the moisture content that caused the increase in the moisture content of the CCL at the PVC geomembrane/CCL interface shown in Table 2. Table 3 presents the profile of moisture content against depth at location 4600, which is typical of the other two locations. Table 3 shows 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 caused solely by water exiting the CCL and migrating into this interface. The exact source of the additional moisture is not known, but it may be related to the underlying natural soils. Another source of the additional moisture is condensation, which is discussed subsequently.

Table 3. Moisture content profile of compacted clay liner at sample location 4600

Depth (mm)	Moisture content (%)	
	Before construction	After construction
25	14.9	20.3
75	18.1	21.6
125	17.9	22.1
175	21.2	18.7
225	20.9	23.5
275	22.0	25.9
325	24.3	24.3
375	21.6	21.2

Figure 5 illustrates the impact of testing the PVC geomembrane/CCL interface under a pre-wetted condition for a compaction water content of 19.6% (approximately optimum) and 21.3%. These two points are shown as dark squares in Figure 5. The pre-wetting of the interface involved using a spray bottle to pre-wet the geomembrane/CCL interface prior to shearing to simulate moisture build-up under the geomembrane that is illustrated in Table 2. The pre-wetted interface tested at a moisture content of 19.6% exhibits a higher peak and larger displacement friction angle than the interface tested at a moisture content of 21.3%. The decrease of friction angle with the increase of clay moisture may be related to a decrease of matric suction with the increase of clay moisture in the CCL and at the interface between the CCL and PVC geomembrane.

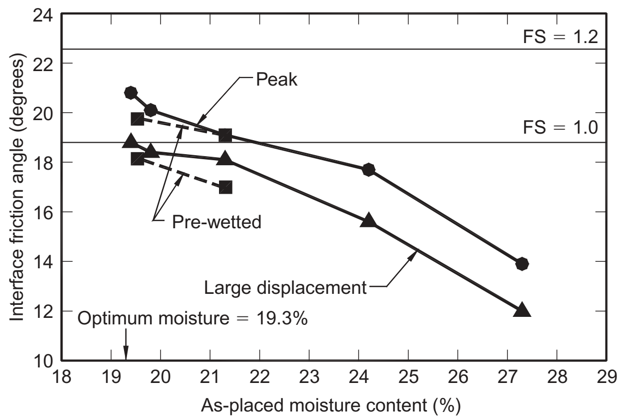


Figure 5. Effect of pre-wetting PVC geomembrane/CCL interface on friction angle at a normal stress of 7.2 kPa

Table 2. Change in moisture content of compacted clay at PVC geomembrane/CCL interface due to construction activities

Moisture content	Sample location no.		
	4555	4572	4600
Optimum	19.3%	19.0%	19.8%
Before placement of drainage blanket and protective FGD 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 peak and large displacement friction angle relationships from Figure 4 are superimposed on the pre-wetted data in Figure 5. Figure 5 shows that the pre-wetted interface strengths are less than or equal to the not pre-wetted interface tests. As discussed subsequently, the likelihood of moisture build-up at the interface suggests that the moisture content at a PVC geomembrane/CCL interface will usually increase during construction, and thus interface testing should reflect a moisture content that is several percentage points above the as-compacted moisture content, and the interface probably should be pre-wetted. This can be accomplished by pre-wetting the geomembrane/CCL interface with a spray bottle prior to shear testing.

4.3. 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 FGD cover material prior to the slide occurred between 25 September and 25 October 1996. During this month, air temperature records show that the daily differential temperature (difference between the maximum and minimum values) ranged from 4°C to 16°C. In the section of the slope that slid in June 1997, the PVC geomembrane was also exposed to daily differential air temperatures of 4–16°C.

After the June 1997 slide, it was observed that the PVC geomembrane exposed to the midday 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. This temperature differential occurred even though the PVC geomembrane is grey in color and not black. 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. Table 4 shows a large temperature differential between the exposed and covered geomembranes. The temperature differential ranges from 12.2°C to 17.7°C higher for the exposed PVC geomembrane than for the covered geomembrane at four different locations. It is believed that the higher surface temperature of the exposed geomembrane heated up the underlying CCL, which caused water vapor to be present in the interface. Water vapor then accumulated under the PVC geomembrane. With placement of the moist sand layer, the geomembrane cooled rapidly, which resulted in the condensation of water under the geomembrane. It is anticipated that this condensation contributed to the increase in moisture content of the CCL at the geomembrane interface. Condensation is another factor that causes 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.

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 can yield unconservative estimates of the field shear strength because the end effects are incorporated in the back-calculated shear strength (Stark and Eid 1998). Because the width of the 1996 and 1997 slides ranges from 15.2 to 22.9 m and the depth of the slide surface is only 0.75 m, the 3-D effects are assumed to be small for these slides. The depth of the slide surface corresponds to an average 0.43 m of drainage sand and 0.32 m of protective FGD material in the slide area As a result, a 2-D analysis was used to back-calculate the mobilized interface strength for the 1996 and 1997 slides.

The 2-D veneer slope stability analysis presented by Koerner and Soong (2005) was used for the back-analysis. The analysis proposed by Koerner and Soong (2005) is suitable to back-calculate the mobilized PVC geomembrane/CCL interface friction angle for a finite-length slope including a passive wedge at the toe and a tension crack near the crest. In the 2-D back-analysis, the conditions at the time of sliding were estimated to include the PVC geomembrane being placed on the CCL at a slope inclination of 3H:1V, i.e. 18.4°, and a maximum slope length of 23.5 m. The PVC geomembrane is loaded with a 0.43 m average thickness of drainage sand and 0.32 m average thickness of protective FGD cover material. The drainage sand and protective FGD cover material exhibit total unit weights of 18.8 kN/m^3 and 14.4 kN/m^3, respectively. A thickness-weighted average total unit weight based on the thickness of the drainage sand and protective FGD cover material of 16.9 kN/m^3 was used in the back-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°. The effective stress shear strength parameters of the stabilized/solidified FGD waste product were estimated to be a cohesion of 48 kPa and a friction angle of 15° from EPRI (1984). A thickness-weighted average effective friction angle based on the thickness of the drainage sand and

Table 4. Temperature of exposed and covered PVC geomembrane

Measurement location	Temperature of exposed PVC geomembrane (°C)	Temperature of covered PVC geomembrane (°C)	Temperature difference (°C)
1	43.3	30.6	12.7
2	42.8	30.6	12.2
3	44.4	26.7	17.7
4	43.3	28.9	14.4

protective FGD cover material was calculated to be 26° and used in the analysis. Because of the granular nature of the drainage sand and the protective FGD cover material, the average cohesion was assumed to be zero.

5.1. General

Figure 6 illustrates the slope profile used in the back-analysis, modified from Koerner and Soong (2005) assuming that the cohesion of both the drainage sand and the protective FGD is zero, and that the adhesion between the drainage sand and PVC geomembrane is zero. In Figure 6, W_A and W_P represent the total weight of the active wedge and the passive wedge, respectively, and N_A and N_P represent the effective normal force acting on the failure plane of the active and passive wedges, respectively. The symbols h and L represent the total thickness of the cover materials and the length of the slope, respectively. The symbol δ represents the interface friction angle between the drainage sand and PVC geomembrane. In contrast, the symbol ϕ' represents a weighted average effective friction angle of the drainage sand and protective FGD cover material. The symbol E_A is the interwedge force acting on the active wedge from the passive wedge, and the symbol E_P is the interwedge force acting on the passive wedge from the active wedge.

Koerner and Soong (2005) provide a quadratic equation to analyze the veneer slope stability by setting $E_A = E_P$.

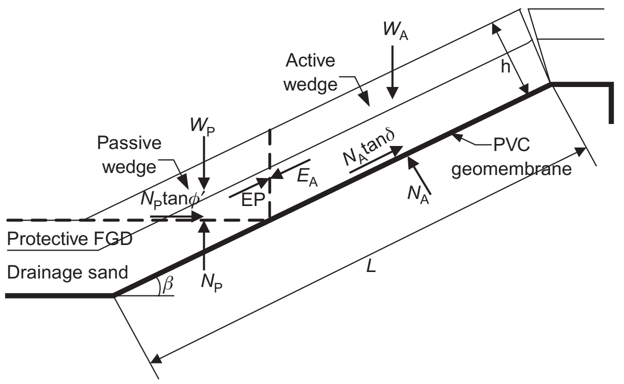


Figure 6. Force equilibrium profile of a finite-length veneer slope (modified from Koerner and Soong 2005)

The equation for the factor of safety (FS) is expressed as follows:

$$a(\text{FS})^2 + b(\text{FS}) + c = 0 \tag{5}$$

where

$$a = (W_A - N_A\cos\beta)\cos\beta$$

$$b = -[(W_A - N_A\cos\beta)\sin\beta\tan\phi' + N_A\tan\delta\sin\beta\cos\beta + W_P\tan\phi'\sin\beta]$$

$$c = N_A\tan\delta\sin^2\beta\tan\phi'$$

The values of W_A, N_A, and W_P are readily calculated using the geometric condition of the slope in Figure 6 along with the thickness-weighted average of the total unit weight of 16.9 kN/m^3. Table 5 summarizes the value of each input parameter used in the back-analysis.

In the 2-D back-analysis the shear resistance of the PVC geomembrane/CCL interface was obtained by setting FS equal to unity in Equation (5). With the assumption that the cohesion both of the drainage sand and of the protective FGD is zero, and the adhesion between the drainage sand and PVC geomembrane is zero, a PVC geomembrane/CCL interface friction angle (δ_{bc}) was back-calculated to be 16.4° using Equation (5) and the slope profile in Figure 6.

5.2. Back-analysis of 1996 slide

Figure 4 can be used to show that a back-calculated peak interface friction angle (δ_p) of approximately 16.4° corresponds to a compaction moisture content of about 25.2%. In other words, to mobilize a δ_p of 16.4°, the moisture content of the CCL at the PVC geomembrane/CCL interface had to increase from to about 25.2%. This corresponds to a 3.7% increase in the CCL moisture content from the average CCL compaction moisture content of 21.5% in the 1996 slide area.

Figure 4 also shows that a large displacement interface friction angle (δ_{LD}) of approximately 16.4° corresponds to a moisture content of about 23.1%. In other words, if a large displacement interface friction angle was mobilized along the PVC geomembrane/CCL interface, the moisture

Table 5. Summary of input parameters for back-analysis of veneer slides

Input parameter	Description	Value
FS	Factor of safety	1.0
β	Angle of the slope	18.4°
h	Total thickness of drainage sand and FGD cover	0.75 m
L	Slope length	73.5 m
ϕ'	Thickness-weighted average effective friction angle of drainage sand and FGD cover	26°
δ	Back-calculated interface friction angle	Varied
γ	Thickness-weighted average total unit weight of the drainage sand and FGD cover	16.9 kN/m^3
W_b[a]	Weight of loaded truck	650 kN
w[a]	Contact length of truck tire	1.0 m
b[a]	Contact width of truck tire	0.6 m
I[a]	Influence factor at the geomembrane interface	0.9
a[a]	Acceleration of truck	0.1g

[a]Input parameters used to reflect truck traffic in 1997 slide back-analysis.

content of the CCL would only have to increase from 21.5% to 23.1%. This increase corresponds to only a 1.6% increase in the CCL moisture content from the average compaction moisture content in the 1996 slide area.

Initially the back-analysis results were perplexing because the compaction moisture content of the CCL in the 1996 slide area is 21.5%, which corresponds to a peak interface friction angle of about 19°, as shown in Figure 4. Thus, to achieve the back-calculated interface friction angle of 16.4°, CCL compaction moisture content had to increase from 21.5% to 23.1% if a large displacement interface friction angle was mobilized, or increase from 21.5% to 25.2% if a peak interface friction angle was mobilized. After the October 1996 slide it was noted that a thin layer of wet clay (approximately 10 mm thick) had formed on the surface of the CCL, which had a moisture content of 27%. Thus it is likely that the moisture content of the geomembrane/CCL interface did increase to 25.2% and a peak interface friction angle of 16.4° was mobilized in the 1996 slide area. This is in agreement with no roads being constructed in the 1996 slide area and no traffic loads being applied to this area, as was the case with the 1997 slide area. This back-analysis suggests that the 1996 slide was caused by moisture condensing under the PVC geomembrane or being drawn to the geomembrane/CCL interface, resulting in an increase in the moisture content at the surface of the CCL and reducing the interface friction angle to about 16.4°.

5.3. Back-analysis of 1997 slide

Because the 1996 and 1997 slide areas are essentially the same, the back-analysis described above is applicable to both slide areas. Thus the back-calculated interface friction angle for the PVC geomembrane/CCL interface is also approximately 16.4° for the 1997 slide area. Figure 4 shows that a peak and large displacement interface friction angle of approximately 16.4° corresponds to compaction moisture contents of about 25.2% and 23.1%, respectively. These back-analysis results were also perplexing because the average compaction moisture content of the CCL in the 1997 slide area is only 19.5%, which corresponds to a peak interface friction angle of about 21.0° as shown in Figure 4. Thus the 1997 slide should not have occurred because the peak interface friction angle (21.0°) is 4.6° greater than the back-calculated angle of 16.4°.

After the June 1997 slide, it was observed that a similar thin layer of wet clay (approximately 5 mm thick) had also formed at the top of the CCL, and it exhibited a moisture content of 22.5% even though the average compaction moisture content in this area is 19.5%. This increase in moisture content probably reduced the peak interface friction angle from 21.0° (moisture content of 19.5%) to about 18.5° (see Figure 4) as the final moisture content at the PVC geomembrane/CCL interface rose to 22.5% (June 1997 slide). However, this increase in moisture content in the 1997 slide area still does not explain the 1997 slide, because an interface friction angle of 18.5° is still greater than the back-calculated of 16.4°. Figure 4 shows that an interface friction angle of 16.4° corresponds to a large displacement interface friction angle, δ_{LD}, for a moisture content of 23.1%. A moisture content of 23.1% is in agreement with a moisture content of 22.5% for the thin layer of wet clay that had formed at the top of the CCL. Thus the back-analysis of the 1997 slide indicates that a large displacement interface shear strength was mobilized in the 1997 slide area but a peak interface shear strength was mobilized in the 1996 slide area. Reasons for the difference in the mobilized interface friction angles were sought.

Stark and Choi (2004) show that a large displacement shear strength is less likely to develop in a veneer stability situation than in a composite liner situation because detrimental shear displacements are less likely in the veneer situation. Detrimental shear displacements are less likely for the veneer situation because of the presence of low shear stresses, low normal stresses (which limits detrimental, i.e. damage-inducing, shear displacements to a geosynthetic interface), smaller shear displacements required for stress transfer in soil or FGD cover material, and small settlement of the compacted soil or FGD veneer as compared with MSW. However, a large displacement shear strength can develop by placing cover soil from the top to the bottom of the slope, from traffic loadings, and if the slope angle of the veneer system is greater than a peak interface shear strength of the weakest interface (Stark and Choi 2004). In this case history, detrimental shear displacements were probably introduced to the PVC geomembrane/CCL interface by the use of a temporary haul road by heavily loaded trucks constructed across the 1997 slide area. The truck traffic is cited for mobilization of a large displacement interface shear strength in the 1997 slide area, because the slope was stable after liner construction and stable after construction of the temporary haul road. However, shortly after heavily loaded trucks carrying FGD for disposal began using the haul road, cracking developed upslope of the haul road. The temporary haul road angles across the slope from the top to the bottom of the slope.

The truck traffic on the haul road induced both static and dynamic forces in the PVC geomembrane/CCL interface, which probably initiated detrimental shear displacement because the interface friction angle of 18.5° is only slightly greater than the δ_{LD} of 16.4°. As a large displacement interface friction angle was being mobilized in the 1997 slide area, the PVC geomembrane began to stretch downslope. As the geomembrane was stretching, additional detrimental shear displacement probably occurred along the PVC geomembrane/CCL interface, causing other areas of post-peak shear strength to develop until the PVC geomembrane finally ruptured. This progressive failure mechanism probably facilitated the development of a larger slide mass in the 1997 slide area than in the 1996 slide area.

To account for the heavily loaded trucks on the slope, the analysis presented by Koerner and Soong (2005) is used. In the 1997 slide area, the trucks would bring FGD into Phase B for disposal, so the worst-case scenario of the trucks going down the slope loaded with FGD was assumed. The trucks involved are Payhauler 350 C Rear Dump trucks with an average weight (truck plus FGD waste) of 650 kN (65 tons). The Payhauler is supported by eight tires, each with a width of 0.6 m and contact length of 1.0 m. Thus the average contact stress applied by one wheel of a Payhauler dump truck with an average load of 650 kN is 135.4 kN/m^2. The soil and FGD cover thickness in the 1997 slide area is 0.75 m. The stress influence factor at the PVC geomembrane interface for a ratio of tire width to cover thickness of 0.8 is about 0.9 from Koerner and Soong (2005). Thus the equivalent equipment force per unit width at the geomembrane interface for a Payhauler 350 C Rear Dump truck is 121.9 kN/m. Based on construction accounts, the typical speed of the Payhauler 350 C Rear Dump trucks is about 21 km/h (13 mph), which converts to an acceleration of about 0.1g based on the time to reach the typical speed in 6 s. The other parameters used in this back-analysis are a slope inclination of 3H:1V, i.e. 18.4°, a maximum slope length of 23.5 m, sand and FGD cover thickness of 0.75 m with a thickness-weighted average total unit weight of 16.9 kN/m^3. Table 5 summarizes the values used in the back-analysis for each input parameter. Inclusion of the dynamic forces induced by the Payhauler 350 C Rear Dump trucks results in a back-calculated δ for the PVC geomembrane/CCL interface of approximately 18.7° for the 1997 slide area. The back-calculated δ of 18.7° is in excellent agreement with the 18.5° that corresponds to the final moisture content at the PVC geomembrane/CCL interface being 22.5%. This is in agreement with field observations of the slope being constructed and performing satisfactorily until the heavily loaded dump trucks started traversing the slope. Shortly after the dump truck traffic started along the slope, the cracking in the slope developed.

Based on field observations and the back-analysis, it is concluded that the October 1996 and June 1997 slides were caused mainly 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. Another factor in the 1997 slide area is the presence of traffic loading, which probably introduced detrimental shear displacement and mobilization of a large displacement interface shear strength in the 1997 slide area. Mobilization of a large displacement interface shear strength over the slide area was probably facilitated by progressive failure occurring along the PVC geomembrane/CCL interface.

6. CONCLUSIONS

This paper describes the failure mechanism, subsurface investigation, engineering properties of the stabilized/solidified FGD product and geomembrane/CCL interface, back-analysis, cause, and lessons learned from two veneer slides. Construction conditions experienced during the installation of a landfill geocomposite liner system led to two veneer slides on a 3H:1V internal slope during construction. 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 slides developed by movement along the PVC geomembrane/CCL interface during or shortly after placement of the protective FGD 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 large displacement values as the compacted clay moisture content increased 8 percentage points. When performing geomembrane/CCL interface direct shear testing, the moisture content of the CCL should be several percentage points above the as-compacted moisture content to account for a moisture increase at the interface due to condensation, and/or the interface should be moistened with a spray bottle. To reduce the accumulation of moisture at a PVC geomembrane/CCL interface due to condensation, it is recommended that the PVC geomembrane be quickly protected by other cover materials to reduce heating of the PVC geomembrane. In addition, shear stresses induced by surface traffic 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 or large displacement value.

ACKNOWLEDGMENTS

The contents and views in this paper are the authors' and do not necessarily reflect those of any of the represented corporations, agencies, organizations or contributors. The first author wishes to express his appreciation to G. Camporini, D. Sheets, and AEP construction personnel under the direction of R. Taylor, for their cooperation and support during the events described in this paper, and during the preparation of this paper. The first author would also thank Ohio EPA's South-East District Office for their valuable collaboration and prompt review. This support is gratefully acknowledged.

NOTATIONS

Basic SI units are given in parentheses.

a	acceleration of truck (m/s^2)
b	contact width of truck tire (m)
FS	factor of safety (dimensionless)
h	total thickness of drainage sand and FGD cover (m)
I	influence factor at the geomembrane interface (dimensionless)
L	slope length (m)
r_u	pore pressure ratio (dimensionless)
W_b	weight of loaded truck (N)
w	contact length of truck tire (m)
β	slope angle (degrees)
δ	back-calculated interface friction angle (degrees)
δ_{LD}	large displacement friction angle (degrees)
δ_p	peak friction angle (degrees)

γ thickness-weighted average total unit weight of drainage sand and FGD cover (N/m^3)
σ'_n maximum normal stress (Pa)
τ shear stress (Pa)
ϕ' thickness-weighted average effective friction angle of drainage sand and FGD cover (degrees)

ABBREVIATIONS

CCL	compacted clay liner
FGD	flue gas desulfurization
MSW	municipal solid waste
PGI	PVC geomembrane institute
PVC	poly(vinyl chloride)
QA	quality assurance
QC	quality control
USCS	Unified Soil Classification System

REFERENCES

ASTM D 698. *Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lb/ft³)*. ASTM International, West Conshohocken, PA, USA.

ASTM D 5321. *Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method*. ASTM International, West Conshohocken, PA, USA.

Bishop, A. W. & Morgenstern, N. (1960). Stability coefficients for earth slopes. *Géotechnique*, **10**, No. 4, 129–150.

Daniel, D. E. & Wu, Y. K. (1993). Compacted clay liners and covers for arid sites. Journal of Geotechnical Engineering, ASCE, **119**, No. 2, 223–237.

Electric Power Research Institute (1984). *Monitoring Fixed FGD Sludge Landfill: Conesville, Ohio*, EPRI Project Number 1406-2 and Publication Number CS-3702, prepared by Michael Baker Jr., Consulting Engineers, Inc., 4301 Dutch Ridge, Beaver, PA 15009.

Hillman, R. P. & Stark, T. D. (2001). Shear strength characteristics of pvc geomembrane-geosynthetic interfaces. *Geosynthetics International*, **8**, No. 2, 135–162.

Koerner, R. M. & Soong, T.-Y. (2005). Analysis and design of veneer cover soils. *Geosynthetics International*, Special Issue on the Giroud Lectures, **12**, No. 1, 28–49.

NAVFAC (1982). *Soil Mechanics, Design Manual 7.1*. Naval Facilities Engineering Command, Department of the Navy, Alexandria, VA, 244 pp.

PGI-1104. *PVC Geomembrane Material Specification 1104*. PVC Geomembrane Institute, University of Illinois, Urbana, IL, www.pvcgeomembrane.com, 1 January 2004.

Stark, T. D. and Choi, H. (2004). Peak versus residual interface strengths for landfill liner and cover design. *Geosynthetics International*, **11**, No. 6, 491–498.

Stark, T. D. & Eid, H.T. (1998). Performance of three-dimensional slope stability methods in practice. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **124**, No. 11, 1049–1060.

The Editors welcome discussion on all papers published in Geosynthetics International. Please email your contribution to discussion@geosynthetics-international.com by 15 August 2006.