Design of a landfill final cover system

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ABSTRACT: This paper describes a final cover slope failure at a municipal solid waste containment facility. The lessons learned from this case history include: (i) slope stability analyses should be conducted whenever field conditions differ from initial design assumptions, such as a steeper slope and different geosynthetics; (ii) published values of interface strength/friction angle should not be used for final design, instead site-specific interface testing should be used; (iii) final cover slope angle should not exceed the lowest geosynthetic interface strength in the cover system to prevent tension in the geosynthetics and/or progressive slope failure of the slope; and (v) designers should resist the temptation to utilise a pre-existing final cover design without performing the necessary field reconnaissance, interface testing, analysis, and design for the new site.

KEYWORDS: Geosynthetics, Geosynthetic-lined slopes, Interface shear strength, Waste containment, Strength, Stability, Shearbox test, Failure, Final cover system, Landfill

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1. INTRODUCTION

This article describes a case history that illustrates a number of important design, specification, bidding, and construction issues for landfill final cover systems. The case history involves an unlined landfill located on the east coast of the United States. The landfill operated from 1977 to 1998 when an adjacent lined landfill was opened. The closed site consists of about 113 300 m^2 with a closed landfill area of about 89 300 m^2. The landfill site is flat and is underlain by sandy soils with groundwater near the ground surface. The closed portion of the landfill is unlined and does not have a leachate collection and removal system below the waste and so the installation of a final cover system was pursued to reduce infiltration, leachate generation and possible groundwater contamination. The applicable state regulations require a minimum of 0.6 m of interim soil cover prior to final closure, the slopes to be 3H : 1V or less, and a final cover system that contains a low permeability layer, namely a geomembrane, to reduce infiltration.

This article reviews the initial design, specification, bidding, construction problems, and the remedial measures for this final cover system. The article also provides recommendations to help avoid some of the problems encountered at this site in other future projects.

2. INITIAL DESIGN AND SPECIFICATIONS

In this case, the initial final cover system was designed by a state agency. Thus, one state agency permitted or approved the design of another state agency without any external review. This lack of external review can be problematic when the design is constructed by a private contractor because problems may develop with the design. If a state agency also performs the construction activities, external review may not be as critical because the state can assume responsibility for the design errors instead of dealing with a contractor claim.

Figure 1 shows the original design which included two unbonded nonwoven geotextiles, a 0.5-mm (20 mil) thick polyvinyl chloride (PVC) geomembrane, and a single-sided drainage composite. The single-sided drainage composite was a geonet with a nonwoven geotextile heat-bonded to the top side of the geonet. A double-sided drainage composite is a geonet with two nonwoven geotextiles heat-bonded to the top and bottom sides of the geonet. In this case, a single-sided drainage composite was used with an unbonded geotextile below the geonet and not a double-sided drainage composite.

The unbonded geotextiles above and below the PVC geomembrane were designed to provide a cushion between the geomembrane and the overlying geonet and the under-

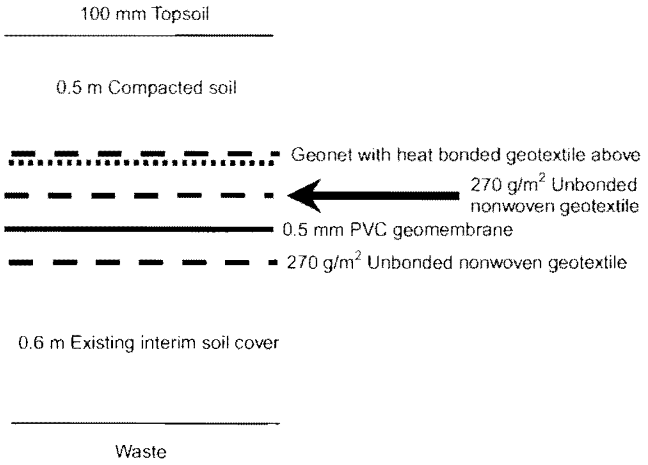

Figure 1. Original final cover system design with unbonded geotextile below geonet

lying interim soil cover, respectively. Fortunately, the unbonded nonwoven geotextile underlying the geonet was removed from the design in the first addendum to the contract bid package. The inclusion of this unbonded geotextile/geonet interface could have resulted in a weak interface between the geonet and PVC geomembrane. This observation is based on interface shear test results presented in Hillman and Stark (2001) for PVC geomembrane/double-sided drainage composite interfaces. In this testing, the nonwoven geotextile in contact with the geomembrane delaminated leading to movement at the geotextile/geonet interface which exhibited a low shear resistance. Therefore, an unbonded geotextile/geonet interface would have exhibited even lower shear resistance than that reported by Hillman and Stark (2001).

The final cover system attempted by the contractor is shown in Figure 2 and did not include the unbonded geotextile below the geonet but all of the other components from Figure 1 remained the same. Based on field observations during construction, sliding occurred along the geonet/PVC geomembrane interface shown in Figure 2.

At this site no interface testing was conducted before, during, or after design to assess slope stability or the impact of an unbonded geotextile below the single-sided

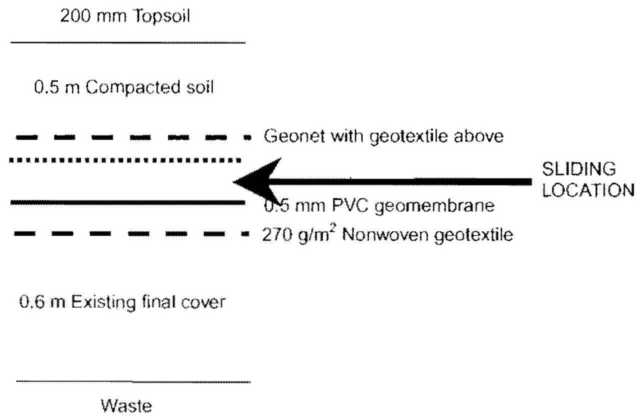

Figure 2. Final cover system bid and attempted by contractor

drainage composite. In addition, no interface testing or analysis was performed on the geonet/PVC geomembrane interface created after removal of the unbonded geotextile. Unfortunately the geonet/PVC geomembrane interface eventually proved problematic as shown in Figure 2. In summary, the stability analyses performed did not consider site-specific geosynthetics in the cover system.

The final cover design for this site utilized interface strength parameters obtained from the available literature. This is not advised because site conditions, geosynthetics, and site soils usually differ from those used to generate the literature data. The importance of site-specific testing can be illustrated by considering the PVC geomembrane interface strengths presented by Hillman and Stark (2001). A designer might be tempted to use the strength values presented in Hillman and Stark (2001), as suggested by Richardson and Chicca (2005), for the design of a site. However, Hillman and Stark (2001) tested the faille side of a 0.75-mm thick PVC geomembrane and so their interface strength values are only applicable to these materials and should not be used unless the site to be designed utilizes the faille side of a 0.75-mm thick PVC geomembrane for the field interface. At this landfill, the smooth side of a 0.5-mm thick PVC geomembrane was installed below the geonet rather than the faille side of a 0.75-mm thick PVC geomembrane, and so the Hillman and Stark (2001) data is not applicable because the faille and smooth sides of a PVC geomembrane usually yield different interface strength parameters and differences in PVC geomembrane thickness also usually result in different interface strengths. For example, less embedment of the geonet into the PVC geomembrane usually occurs with a thinner PVC geomembrane which results in a lower shear resistance. Thus, it is expected that the smooth side of the 0.5-mm thick PVC geomembrane used at this site would exhibit a different shear resistance to that of the faille side of the 0.75-mm thick membrane, as used by Hillman and Stark (2001) for the geonet/PVC geomembrane interface in Figure 2.

This cover design also resulted in the friction angle of the weakest interface, namely the geonet/PVC geomembrane interface, being less than the final cover slope angle. This will lead to tensile forces developing in the geosynthetics and/or progressive slope failure of the final cover slope even if the cover system is successfully completed. It is recommended that the lowest interface friction angle should exceed the steepest slope angle (Stark and Poeppel 1994; Gilbert and Byrne 1996; Stark and Choi 2004).

3. INITIAL CONSTRUCTION PROBLEMS

Before placement of the geosynthetics for the final cover system commenced, the regulatory agency identified areas of the landfill that did not possess the specified depth of 0.6 m of interim soil cover. To remedy this deficiency the owner placed additional interim soil cover on the existing slopes. This caused some of the final slopes to be steeper than the designed and regulated slope of 3H : 1V. Figure 3 is a plan view of the landfill which is shaped like a saucepan with a long handle. The slopes which were

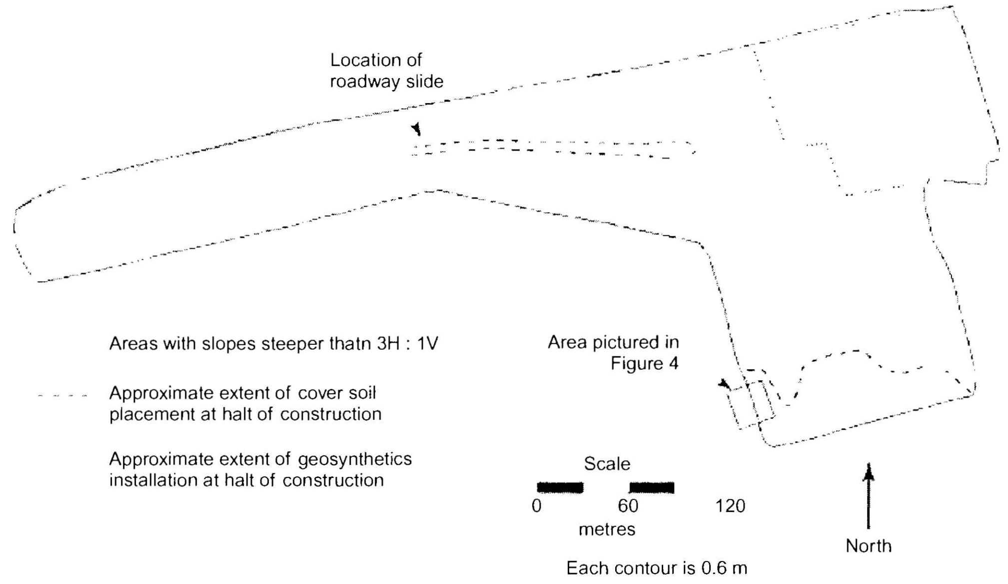

Figure 3. Plan view showing slopes steeper than 3H : 1V after placement of additional soil to achieve required 0.6 m thickness of intermediate soil cover

steeper than 3H : 1V are primarily located along the panhandle as shown in Figure 3. As a result, the field conditions differed from the final design conditions with the field slopes steeper than 3H : 1V.

The slopes of the over-steepened areas could not be remediated for the following two reasons. First, the slope toe along the panhandle could not be moved outward to reduce the slope inclination because adjacent properties were within 0.3 to 1.6 m of the slope toe. The designer sought a variance for the 0.6 m thickness of the interim soil cover to reduce the slope inclination but the permitting agency denied the request. This led to the placing of additional cover soil by the owner and the contractor trying to cover slopes steeper than 3H : 1V. Second, the contractor was not tasked with removing the recently placed interim soil cover, removing waste to flatten the slope to 3H : 1V, disposing of the waste, and replacing the interim soil cover to achieve a 3H : 1V slope in the over-steepened areas along the panhandle. In such a situation, the stability analyses should be revised to reflect the over-steepened slopes (steeper than the 3H : 1V design slope) before the contractor is allowed to proceed to prevent problems from developing. If the stability analyses show that the over-steepened areas will not be stable then the design should be revised before the contractor is allowed to take action.

4. FIRST SLOPE FAILURE

The following paragraphs illustrate some of the problems that developed when the contractor initiated construction of the final cover system shown in Figure 2. Access limitations at the north side of the panhandle prevented the creation of an access road at the slope toe so the contractor started placing vegetative cover soil over the installed geosynthetics on the crest of the panhandle from the east towards the west (see Figure 4). Upon reaching the western end of the panhandle, the contractor intended to place material down the western-most slope and then install cover soil from the bottom to the top of the slope along the north and south sides of the panhandle.

Figure 4. Aerial view of panhandle and southwestern slope

Before proceeding too far down the panhandle, a portion of the roadway slid to the north off the narrowest portion of the panhandle crest (see arrow in Figure 4). The slide movement clearly occurred at the geonet/PVC geomembrane interface (see Figure 5) because the overlying single-sided drainage composite tore (see Figure 6) and this left the PVC geomembrane exposed, and essentially undamaged, on the slope. The drainage composite tore because there was no constructed seam in the drainage composite near the crest of the slope where the tensile forces were the highest.

Because a well-defined and unsupported slide block developed, this instability provided an ideal opportunity to back-calculate the interface friction angle that was mobilized along the geonet/PVC geomembrane interface. The small amount of tensile resistance of the geonet that was mobilized when the geonet tore was not considered in the limit-equilibrium back-analysis of this slide described subsequently but it was considered in a subsequent

Figure 5. Panhandle slide showing exposed surface of the PVC geomembrane

Figure 6. Panhandle slide showing torn single-sided drainage composite (see arrow) and intact geomembrane

back-analysis using the stability methodology presented by Giroud *et al.* (1995).

Back-analysis of slope failures, especially those with simple geometries such as the panhandle slide, provides a good estimate of the mobilized or field interface strength (Stark and Eid 1998). Laboratory shear tests try to simulate field conditions but for a number of reasons laboratory shear tests are only an approximation of the field conditions (Stark *et al.* 2000). In addition, problems with the interpretation of laboratory test results, which will be discussed subsequently, are alleviated by using a back-analysis to estimate the field interface strength. For example, two series of interface shear tests were conducted by a commercial testing laboratory on the geonet/PVC geomembrane installed at this site using stockpiled material to investigate the range of the interface strength parameters for this interface after the failure. Each series of interface tests were conducted using normal stresses of 4.8, 12.0 and 19.2 kPa to simulate the field normal stresses and estimate the peak failure envelope. The peak friction angle from these two series of tests varied from 14.4° to 18.4° and the adhesion/cohesion intercept for this smooth interface varied from 1.5 to 0.2 kPa, respectively, even though the same materials and laboratory were used for each set of tests. The commercial laboratory reported an adhesion for the geonet/PVC geomembrane interface at normal stresses from 4.8 to 19.2 kPa even though the smooth interface typically does not exhibit an adhesion as discussed below.

Back-analysis of the panhandle instability was performed in the present study using the computer program XSTABL Version 5 (Sharma 1996) and the stability method of Spencer (1967) as coded in XSTABL. Spencer's stability method was used for the back-analysis because it satisfies all conditions of equilibrium and provides the best estimate of the factor of safety (Duncan and Wright 1980). The geometry for the cross-section used in the back-analysis is shown in Figure 7 and was determined from a site visit by the first author.

Table 1 presents the cover soil properties used in the back analysis. These properties were obtained from a report prepared by a consulting engineering company hired by the state. The adhesion of the geonet/PVC geomembrane interface was assigned a value of zero in the back-analysis because this is a smooth interface that does not exhibit a 'Velcro' effect. Thus, the friction angle of the geonet/PVC geomembrane interface was back-calculated; that is, varied using XSTABL until a factor of safety of unity was obtained.

The inclination of the slope in the panhandle instability area was approximately 2.74H : 1V (20.0°) based on the topographic survey performed by the contractor after

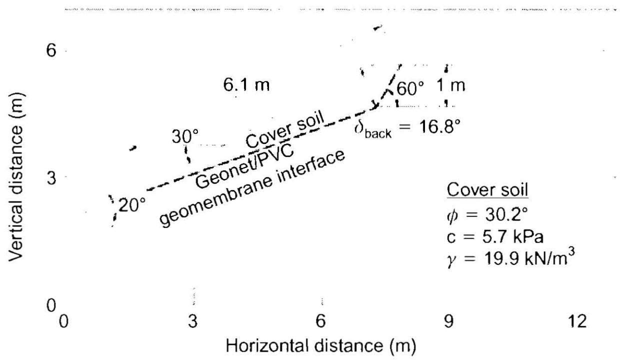

Figure 7. Geometry used in back-analysis of panhandle slide

Table 1. Cover soil and geonet/PVC geomembrane interface parameters used in back-analysis

Material	Unit weight (kN/m^3)	Cohesion/adhesion (kPa)	Friction angle (°)
Cover Soil	19.9	5.7	30.2
Geonet/PVC geomembrane interface	N/A	0	16.0 (back-calculated)

placement of the additional cover soil by the owner (see Figure 3). This slope angle yielded a back-calculated interface friction angle of 16.0° after applying a 5% reduction to correct for the three-dimensional effects of the cover soil along the sides of the slide block (Stark and Eid 1998). The back-calculated friction angle of 16.0° is in agreement with the range of friction angles (14.4 to 18.4°) reported by the commercial laboratory.

The adhesion value was set to zero in the back-analysis because the geonet/PVC geomembrane interface does not exhibit an adhesion, which means that there is no shearing resistance at zero normal stress or 'Velcro' effect. This can be visualised by placing the geonet on a sample of the PVC geomembrane and observing little to no resistance when the geonet is slid along the surface of the geomembrane with no normal stress applied. A non-zero value of adhesion indicates that the interface exhibits a shear resistance at zero normal stress. An example of a geosynthetic interface that exhibits an adhesion value greater than zero is a textured high-density polyethylene (HDPE) geomembrane/non-woven geotextile interface where the geotextile does not slide easily over the geomembrane because of a 'Velcro' effect (Stark *et al.* 1996). The inclusion of a non-zero adhesion for an interface subjected to low normal stress can be non-conservative because the component of shearing resistance derived from adhesion can be much larger than the component derived from the friction angle, which is proportional to the normal stress. The adhesion is not normal stress-dependent and is multiplied by the length of the failure surface at all normal stresses. In summary, the back-analysis showed that the geonet/PVC geomembrane interface installed at the site exhibited a friction angle of about 16.0° and an adhesion of zero.

The inclination of the slope in the panhandle instability area was approximately 2.74H : 1V or 20.0° which meant that the slope angle exceeded the back-calculated interface friction angle (16°) of the geonet/PVC geomembrane interface. This caused tensile stresses to develop in the geosynthetics until they tore and the slope failed because no toe buttress was present in this case. If a toe buttress was present, the slope may have remained stable, at least for a short time, by relying on the toe buttressing. However, progressive failure and the development of a post-peak interface strength, namely an interface strength that is lower than the peak interface strength, has been shown to develop even with a toe buttress if the slope angle is greater than the peak friction angle of the critical interface (Stark and Poeppel 1994; Gilbert and Byrne 1996; Stark and Choi 2004). As a result, the weakest interface should exhibit an interface friction angle that exceeds the slope angle to prevent the occurrence of shear displacement on the weakest interface, development of a post-peak strength along the interface, and progressive failure.

Giroud *et al.* (1995) present a limit equilibrium method to evaluate the stability of geosynthetic–soil layered systems of slopes. The main benefit of this method is that the factor of safety is expressed by an equation that consists of the sum of the five terms shown in Equation 1. This allows the contribution of various factors in the geosynthetic–soil layered system to be identified and quantified. For example, the first two terms of the equation represent the friction and adhesion strength parameters of the geosynthetic interface. The next two terms of the equation represent the shear strength and geometry parameters for the soil buttress at the slope toe. The last term in Equation 1 represents the geosynthetic tension of the geosynthetics in the slope.

In the back-analysis of the panhandle slide, no soil buttress was involved because the cover soil did not extend to the slope toe. As a result, the two soil buttress terms were removed from the factor of safety expression in Equation 1 as shown in Equation 2. Equation 1 was further simplified by removing the adhesion term because the PVC geomembrane and geonet interface does not exhibit an adhesion as shown in Equation 2.

$$FS = \frac{\tan\delta}{\tan\beta} + \frac{a}{\gamma t \sin\beta} + \frac{t}{h}\frac{\sin\phi}{\sin(2\beta)\cos(\beta+\theta)} + \frac{c}{\gamma h}\frac{\cos\phi}{\sin\beta\cos(\beta+\theta)} + \frac{T}{\gamma h t} \tag{1}$$

where a is the geosynthetic interface adhesion (kPa); δ is the geosynthetic interface friction angle (°); c is the soil cohesion (kPa); ϕ is the soil friction angle (°); β is the slope angle (°); t is the thickness of the soil cover (m); h is the vertical height of the slope (m); γ is the unit weight of the cover soil (kN/m^3); T is the geosynthetic tension (kN/m).

$$FS = \frac{\tan\delta}{\tan\beta} + \frac{T}{\gamma h t} \tag{2}$$

Koerner (1998) gives an average peak tensile strength of a bi-axial geonet in the machine direction (the geonet installed at this site is bi-axial and the geonet was oriented with the machine direction aligned down the slope) of 12.0 kN/m. Using Equation 2, a factor of safety of unity, slope geometry parameters in Figure 7, and the tensile resistance of the geonet from Koerner (1998), the interface friction angle, δ, can be back-calculated and compared with the XSTABL back-analysis using Equation 3:

$$\delta = \tan^{-1}\left(\tan\beta\left[FS - \frac{T}{\gamma h t}\right]\right) \tag{3}$$

Based on field observations, Figure 4 for example, the following values were used in Equation 4 to back-calculate the geonet/PVC geomembrane interface friction angle: $\beta = 20°$ (slope angle) (2.74H : 1V); $t = 1.0$ m; $h = 3.1$ m (for a slope length of 6.1 m and slope angle of 20°, see Figure 7); $\gamma = 18.1$ kN/m^3; $T = 12.0$ kN/m :

$$\delta = \tan^{-1}\left(\tan(20^{\circ})\left[1 - \frac{12.0\,\mathrm{kN/m}}{(19.9\,\mathrm{kN/m^3} * 2.1\,\mathrm{m} * 1.0\,\mathrm{m})}\right]\right) = 14.5^{\circ} \tag{4}$$

The XSTABL back-analysis yielded a back-calculated friction angle of about 16° so the geonet tensile strength may have contributed the equivalent of about 1.5° of shear resistance to this geonet/PVC geomembrane interface because of the low normal stress applied. Given the slope angle was about 20° (2.74H : 1V) in this area, the geonet was probably placed in tension shortly after the cover soil was applied to the slope crest because the slope angle exceeded the interface friction angle. The geonet/PVC geomembrane interface started to deform until the geonet tore and the slide occurred shortly thereafter as evidenced by the road only having advanced slightly past the slide area (see Figure 4). Accepted design practice is for geosynthetics to be designed without tension, that is, having a friction angle that exceeds the weakest interface.

In summary, the geonet/PVC geomembrane interface in the panhandle slide area exhibited a mobilized friction angle of 15 to 16° and an adhesion of zero. This interface friction angle was insufficient to support the 1.0-m thick of cover soil on the north slope of the panhandle and resulted in the first slope instability at the site.

The authors conducted site-specific interface testing of the geonet/PVC geomembrane using the torsional ring shear test procedure described in Hillman and Stark (2001). Three ring shear tests were conducted using stockpiled geosynthetics from the site and normal stresses of 4.8, 12.0 and 19.2 kPa to simulate the 0.6 m of cover soil. The ring shear tests were conducted at a shear displacement rate of 0.37 mm/min and the specimens were not submerged. The geomembrane and geotextile of the single-sided drainage composite were glued to a Lucite ring using a thin coat of epoxy cement and allowed to cure separately for 24 h under a normal stress of approximately 19.2 kPa to create the geonet/PVC geomembrane. After curing of the epoxy cement, the geomembrane and drainage composite were placed in contact with each other in the ring shear device. The specimen container and geosynthetics were marked to ensure that the geosynthetics did not slip during shear. The ring shear tests were conducted at a laboratory temperature of 20°C.

Figure 8 presents the shear stress–displacement relationships from the 0.5 mm PVC geomembrane/single-sided drainage composite ring shear interface tests. Using these relationships, the peak and residual failure envelopes for the geonet/PVC geomembrane interface were constructed and are shown in Figure 9. A residual failure envelope is also plotted, not a large displacement failure

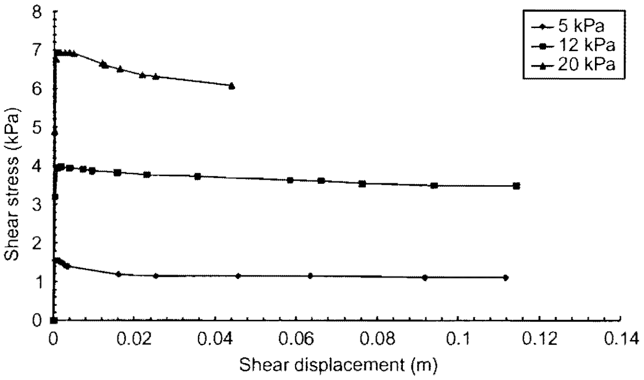

Figure 8. Torsional ring shear test results on geonet/PVC geomembrane interface

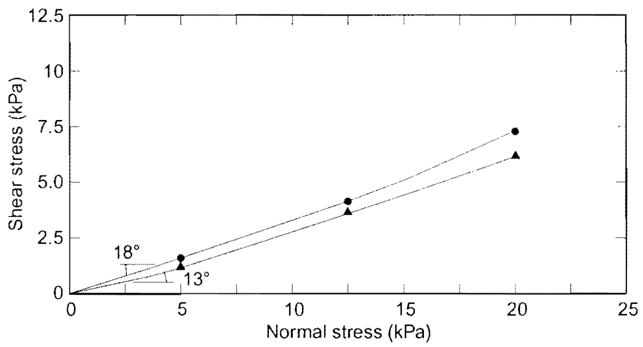

Figure 9. Peak and residual torsional ring shear failure envelopes for geonet/PVC geomembrane interface

envelope, because a ring shear device was used and a shear displacement of about 110 mm was achieved to reach a residual strength condition.

The peak and residual failure envelopes were stress dependent and corresponded to peak and residual friction angles of approximately 18 and 13°, respectively. The peak strength increased in a non-linear fashion at a normal stress of 20 kPa because the geonet started to embed into the PVC geomembrane. These friction angles are also less than the slope angles of 18.4° (3H : 1V) to 21.8° (2.5H : 1V) observed at the site and confirm that instability should have occurred. The measured peak friction angle is also in agreement with the back-calculated friction angle of 16° and range of friction angles measured by the commercial laboratory (14.4 to 18.4°) with the observed difference being caused by differences in field and laboratory conditions.

5. SECOND SLOPE FAILURE

In an effort to keep the project progressing while the designer evaluated the panhandle slide, the contractor moved to the southernmost portion of the landfill, namely the bottom of the 'pan' (see dashed box in Figure 4), where installation of the geosynthetics had been recently completed (geosynthetics were still being installed in the north-east corner, i.e. the top of the 'pan', at this time).

The contractor started placing cover soil from the bottom to the top of the south slope. As fill was being placed up the south slope, tension started developing in the single-sided drainage composite at the top of the landfill. This tension manifested itself by failure of some of the plastic ties holding the panels of the drainage composite together and exposing the PVC geomembrane. As a result, soil placement did not reach the top of the south slope before the drainage composite started pulling apart (see box in Figure 4) and sliding off the top of the landfill.

Upon observing the drainage composite sliding off the top of the landfill, the contractor concluded the geonet/PVC geomembrane interface was inadequate to support the placement of cover soil and ceased work. Eventually the owner terminated the contract so that the state could redesign the cover system. The redesigned cover system was completed at an additional cost of US$1.4 million over the contract price of US$2.0 million. The redesigned cover system utilized a textured HDPE geomembrane and a double-sided drainage composite to develop a Velcro effect with the geomembrane. The cover was constructed using a different contractor. The first contractor was paid for all work done but unfortunately its termination was not a good business experience for the contractor even though it was not involved with the original design.

6. PERFORMANCE OF PVC GEOMEMBRANES

The good news from this case history is the performance of the 0.5-mm thick PVC geomembrane. After 24 months of ultra-violet (UV) light exposure in the period after most of the single-sided drainage composite had slid off the slope, the 0.5-mm thick PVC geomembrane still met all of the original project specifications and did not require removal and replacement. The exposed PVC geomembrane was tested for volatile loss (ASTM D1203), dimensional stability (ASTM D1204), puncture resistance (ASTM D4833), hydrostatic resistance (ASTM D751), brittleness (ASTM D1790), and basic tensile properties of plastic sheeting (ASTM D882 Method A) and the test results exceeded the PVC Geomembrane Institute (2004) 1104 specification for new PVC geomembranes. This is in agreement with other case histories showing the long-term durability of PVC geomembranes in a variety of environments (Stark *et al.* 2001, 2005). Conversely, all of the single-sided drainage composite was damaged by UV light exposure over the 24 months that the material was uncovered and had to be removed from portions of the cover that did not fail due to cover soil placement.

7. LESSONS LEARNED AND RECOMMENDATIONS

The technical and construction-related lessons and recommendations generated from this challenging project include the following suggestions.

- Design entities should not utilize prior designs without performing the necessary field reconnaissance, testing, analysis, and design for the new site. Engineering properties of geosynthetics and soils vary from site to site and site-specific testing, analysis, and design should be conducted.
- New slope stability analyses should be conducted when field conditions differ from the design conditions, such as a change in geosynthetics or an increase in slope inclination, and/or length. In this case the stability analyses represent a 3H : 1V slope with an unbonded nonwoven geotextile between the geomembrane and geonet but the actual slope inclination was as steep as 2.5H : 1V and the unbonded geotextile was removed leaving the weak geonet/PVC geomembrane interface.
- Published values of interface friction angle only should be used for preliminary design. Appropriate shear tests utilizing the actual materials involved should be conducted on the potentially weak interfaces prior to bidding to confirm that the required interface strength can be achieved with commercially available products.
- The final cover slope angle should not exceed the lowest geosynthetic interface strength in the cover system because progressive failure can occur along that interface and lead to slope instability. At a minimum this condition can lead to tension developing in the geosynthetics, damage to the geosynthetics, shear displacement occurring along the weak interface, and/or development of a post-peak strength.
- External review should be required for designs that are created and approved by different state agencies if the design is to be submitted for bid and construction by private contractors. If a state agency will perform the construction activities, external review may not be as critical because the state can assume responsibility for design errors.

NOTATIONS

Basic SI units are given in parentheses.

a geosynthetic interface adhesion (Pa)
δ geosynthetic interface friction angle (°)
c soil cohesion (Pa)
δ soil friction angle (°)
β slope angle (°)
t thickness of soil cover (m)
h vertical height of slope (m)
γ unit weight of cover soil (N/m^3)
T geosynthetic tension (N/m).

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