Lessons Learned From the Failure of a GCL/Geomembrane Barrier on a Side Slope Landfill Cover

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ABSTRACT: A sliding failure which occurred during construction of a composite landfill cover is evaluated. Field and laboratory testing showed that gas pore pressures were a significant cause of the failure. The paper shows the sensitivity of slope stability to relatively small changes in gas pore pressure and strength properties of the cover materials. Recommendations are included to help avoid this type of failure.

KEYWORDS: Landfill, Final cover, Composite barrier, Failure, Field observations.

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1. INTRODUCTION

The stability of the final cover on side slopes is influenced by the interface shear strength that exists between the layers of the final cover, the internal shear strength of the material in each layer, and the pore gas or water pressures that develop within the cover profile. Gas and pore water pressure relief layers are commonly constructed of granular soils or geonets. The limiting interface shear strength is typically the interface friction between a geosynthetic barrier layer and the gas pressure relief layer or the water pressure relief layer. This paper examines a final cover failure resulting from excessive gas or pore water pressures at one of the geomembrane surfaces to illustrate that composite barriers will inherently fail if used on significant slopes where measures are not taken to control gas and pore water pressures.

A sliding failure occurred during construction of a 6 ha (15 acre) final cover project. The slope on which the failure occurred was inclined at 1V:4H (25%, or 14 degrees), and was 18 m (60 ft) high with no benches. The cover system design consisted of the following elements, from top to bottom:

- 15 cm (0.5 ft) of topsoil
- 30 cm (1-foot) vegetative soil
- 30 cm (1-foot) drainage layer sand
- PVC geomembrane
- Geosynthetic clay liner (GCL)
- 30 cm (1-foot) thick gas relief layer consisting of a fine sand with a measured hydraulic conductivity of 0.005 cm/s.
- Foundation soil over waste

The design also included vertical gas vents spaced on a 60 m (200-foot) square grid. The wells consisted of 45 cm (18-inch) borings 4.6 to 18.3 m (15 to 60 feet) deep, with 15 cm (6-inch) diameter slotted PVC pipe backfilled with pea gravel around the pipe.

2. THE FAILURE

Failure of the cover occurred during placement of the drainage sand layer. Figure 1 shows one of several slide areas. Each slide created one or more long tears in the geomembrane. At the time of failure, construction of the gas vents, sand gas-relief layer, GCL, and geomembrane were complete. Sand was being pushed up slope from the base. 3.2 ha (8 acres) of drainage sand 30-60 cm (1-2 feet) deep had been placed over the geomembrane.

The observed failure mode was the geomembrane stretching and then tearing at the top of the slope. The sand on top of the geomembrane, and the geomembrane, slid downslope along the geomembrane/GCL interface. The GCL did not appear to be distressed or displaced. However, a thin film of bentonite covered the GCL/geomembrane interface. Apparently, bentonite had extruded through the GCL which was placed with the slit-film side facing up.

As the failure progressed and rain eroded portions of the upper sand drainage layer, large gas bubbles formed in the geomembrane. Figure 2 shows a small gas >whale= of the geomembrane observed after the sliding failure had occurred. Even the exposed GCL appeared to be uplifted by gas pressures.

3. FIELD AND LABORATORY INVESTIGATIONS

Twelve probes were installed beneath the cover system to measure gas pressure. These were monitored over a period of two months. They revealed an average gas pressure in the gas venting layer of 1.7 kPa (6.8 inches of water, or 35 psf) in the nine most critical locations. The probe with the highest pressure, averaged over 23 readings, showed 3.3 kPa (13.3 inches of water, or 69 psf) of gas pressure, and had a single high reading of 4.0 kPa (16 inches of water, or 83.2 psf).

Strength testing was performed on samples from the PVC geomembrane/hydrated GCL interface over a normal load range of 2.4-12 kPa (50-250 lb/ft^2) after the failure. The measured Mohr-Coulomb shear strength parameters reported by the testing laboratory were 16 degrees friction (φ'), and a y-intercept (c') of 0.67 kPa (14 lb/ft^2). The interface strength parameters for the drainage sand sliding on the PVC geomembrane are a cohesion of 0 and a friction angle of 28°.

4. ANALYSIS

The stability of final cover systems is typically evaluated using an infinite slope model for slope stability. This method is acceptable since the individual layers within the cover profile are very thin in comparison to their areal dimensions. Giroud, et. al. (1995) discussed this technique applied to the sliding of landfill covers. Neglecting seismic forces, the stability of the slope can be evaluated using an infinite slope model by the following equation:

$$FS = \frac{\dfrac{c'}{\gamma_t \bullet d \bullet \cos^2 \beta} + \left(1 - \dfrac{u}{\gamma_t \bullet d \bullet \cos^2 \beta}\right) \tan \phi'}{\tan \beta}$$

where FS = factor of safety

γ_t = unit weight of slope material(s)

c' = cohesion component of strength on the interface

φ' = angle of internal friction on the interface

β = angle of the slope

d = vertical depth to the assumed failure interface or surface

u = pore pressure on the interface

Above the hydraulic barrier, u equals the pore water pressure on the interface which is $h_w \gamma_w \cos^2 \beta$ where h_w is the vertical height of water over the interface and γ_w is the unit weight of water. Below the hydraulic barrier, u equals the pore gas pressure on the interface. This model neglects potential toe buttressing forces and the tensile strengths of the geosynthetic components of the liner system.

The interface strength parameters and the pore pressure are generally different on the two sides of the geomembrane. Consequently, separate stability analyses are required for sliding on top of the geomembrane and sliding at the bottom of the geomembrane. Since the sliding was observed to occur on the bottom of the geomembrane, we would expect a higher factor of safety for sliding on top of the geomembrane than for sliding on its bottom.

Figure 3 gives the computed factor of safety for sliding of the geomembrane on the GCL as a function of gas pressure on the interface. These results are for 0.3 m (1 foot) of sand at a unit weight of 16.9 kN/m^3 (107 lb/ft^3) covering the geomembrane. Results are shown for four different values of cohesion. A cohesion of 0.67 kPa (14 lb/ft^2) was reported by the testing laboratory from tests on the GCL/geomembrane interface. Two engineers examined the data independently. One obtained a cohesion intercept of 0.38 kPa (8 lb/ft^2) and the other obtained 0.53 kPa (11 lb/ft^2). The fourth set of results uses a cohesion intercept of 0, which is a common practice, and in fact, one often recommended in the literature, e.g. by Koerner and Soong (1998).

Figure 3 shows that the computed factor of safety for these conditions is very sensitive to the cohesion intercept and to the pore pressure on the interface. A discussion of these sensitivities is provided by Liu, et al (1997). For example, simply ignoring the very small y-intercept value of the Mohr-Coulomb envelope decreases the computed factor of safety by 0.6! Clearly, the conventional wisdom of ignoring the cohesive component of interface shear strength may be much too conservative for this situation. Indeed, it would appear that much more attention needs to be given to precisely determining the strength parameters for the interface under these low stress conditions. Parameters selected by different engineers from the same test data give variations in computed factor of safety of as much as 0.4 When one is designing with a factor of safety of 1.5, these large sensitivities to small changes in strength parameters become very important. To proceed with an evaluation of the failure, only factors of safety computed with a cohesion intercept of 0.38 kPa (8 lb/ft^2) will be considered.

Figure 3 shows that with a cohesion intercept of 0.38 kPa (8 lb/ft^2), a pore pressure on the GCL/geomembrane interface of approximately 1.5 kPa (6 inches of water) is sufficient to decrease the computed factor of safety to 1. At a factor of safety of one, the slope should fail by sliding along the GCL/geomembrane interface, which it did. As described above, measurements of gas pressure beneath the geomembrane after failure showed an average gas pressure in the gas venting layer of 1.7 kPa (6.8 inches of water) in the nine most critical locations. Some values were higher. The average gas pressure measured at two of the probes was above 2.5 kPa (10 inches of water column).

Figure 4 shows the computed factor of safety for sliding on top of the geomembrane as a function of pore pressure at the sand/geomembrane interface and strength of the interface between sand and geomembrane. Giroud, et al (1995) examined this important mechanism of cover failure. Rain falling onto the landfill infiltrates the sand and flows downward until it encounters the geomembrane. Then it flows parallel to the slope on top of the geomembrane. An interface friction angle for the geomembrane/sand of 28° was measured in interface shear box tests. With this strength, the factor of safety for sliding of sand cover over the top of the geomembrane is greater than 1 for up to 30 cm (12 inches) of water flowing through 30 cm (12 inches) of sand cover. This result is in agreement with the field observations that sliding occurred at the GCL/geomembrane interace and not at the geomembrane/sand interface.

5. DISCUSSION OF FAILURE CAUSE AND POTENTIAL REMEDIES

Given the regulatory requirement to place a composite cover on the 1V:4H slope, the failure in this case history could have been prevented by providing a combination of (i) increased shear strength at the geomembrane/GCL interface, and (ii) providing additional pore-pressure relief below the barrier layer system.

Increased Shear Strength at Interface. There may be several ways to increase the shear strength of the interface within the composite liner. One way would be to provide a textured geomembrane, and flip the GCL over such that the needle-punched nonwoven (NPNW) side of the GCL formed the interface with the geomembrane. Not only does a NPNW geotextile often have greater shear strength against a geomembrane than a woven geotextile, but the NPNW geotextiles carriers for GCLs tend not to extrude bentonite to the interface under field conditions. Note that even though bentonite has not been observed to extrude from a NPNW interface, the effectiveness of the NPNW side of a GCL to provide an effective composite liner barrier has been demonstrated by Harper et al (1993). A key point is that the shear strength of this interface needs to be thoroughly evaluated by the designer during the design phase and verified by conformance testing on the actual materials to be used in construction for the specific site conditions. The shear strength of the interface should not be taken from assumed or published values, nor should it be left up to the contractor to choose materials based on manufacturer's representations.

Pore Pressure Relief

This design included a 0.3 m (1 foot) thick gas venting layer and gas relief vents spaced every 60 m (200 ft). Based on the fact that gas bubbles were observed in the geomembrane following the failure, the gas venting system was insufficient to keep the pore pressure from developing in the barrier system from gas pressures. There is some basis to believe that the material used for the gas barrier layer met the specifications for hydraulic permeability (0.001 cm/sec) but had insufficient gas permeability under field conditions. Although gas-relief layers are often recommended, and even required by regulations, to the authors' knowledge there is no design guidance published in the literature describing how a gas-relief layer might be designed relative to landfill cover stability. Clearly this needs to be changed. Thiel (1998) has developed a methodology that gives the gas permeability of the drainage layer using established intrinsic permeability relationships. This gas permeability is then used with Darcy's law to predict actual in-service gas pressure gradients.

6. REPAIR OF FAILED FINAL COVER

The failure was repaired by convincing the local regulatory agencies that a composite cover system was not necessary to limit infiltration at this site, and that an adequate environmental closure could be achieved with a single GCL barrier. Results from HELP modeling that showed limited infiltration and experience from other GCL covers were used to convince the Michigan regulators that the GCL barrier would provide adequate protection. They required several lysimeters be placed under the GCL to demonstrate and verify that the GCL provided adequate protection.

The PVC geomembrane was removed, and most of the GCL that had been previously deployed was reused. GCL areas that were damaged by the reconstruction were replaced, and strip drains were placed below the GCL to aid in gas venting. The transmissivity and spacing of the strip drains was determined using the design methodology of Thiel (1998) mentioned above.

7. CONCLUSIONS

Field measurements, analysis results, and construction history all indicate that gas pressures were a significant element that contributed to this failure. Designers need to give more attention to the potential effects of pore water and pore gas pressures on the stability of composite liner systems and take more proactive steps to reduce the potential for excessive pore pressures.

Because of the low normal loads of landfill covers, especially during construction, the stability of these systems is extremely sensitive to pore pressures above and below the geomembrane. Pressures that might normally be considered rather small can provide an uplift force that is a significant percentage of the normal force provided by the overlying materials. This results in a reduced effective normal stress that translates into a reduced shear resistance. An apparent safety factor of 1.5, calculated based on measured shear strength values, but without consideration for pore pressures, was inadequate to meet the field conditions.

8. LESSONS LEARNED

1. The stability of a composite barrier on side slopes can be compromised by relatively small excess water and gas pressures of magnitudes that are common to typical operational conditions in municipal waste landfills.

2. Gas and pore water pressures should be anticipated in the design of landfill covers, and relieved to the extent necessary to obtain the required factor of safety.

3. The shear strength of the interface between a smooth geomembrane, and the slit-film side of a geotextile-based GCL can be affected by hydration of the bentonite, and extrusion of bentonite through the slit film. The combination of a smooth geomembrane and a geotextile-based GCL should be avoided on side slope covers.

4. Interface strength testing should be performed on the specific material combinations for each cover design at the normal stress and hydration conditions appropriate to site

conditions. The test method and equipment should be calibrated and appropriate for the low normal loads and shear stresses for cover designs. Use of generic or published data for interface shear strength, or dependence on others not familiar with the design to provide the required testing, can lead to expensive failures.

5. Landfill covers are extremely sensitive to small changes in shear strengths, and effective normal stresses. The designer needs to be aware of these effects and account for them using appropriately conservative input values, combined with an adequate factor of safety appropriate for the level of confidence in the input parameters.

A composite barrier should not be used on slopes unless the designer fully considers the above factors.

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LIST OF FIGURES

Figure 1: Tear in geomembrane cover

Figure 2: Gas bubble in geomembrane cover

Figure 3: Factor of Safety for PVC/Geomembrane Interface on 1V:4H Slope

Figure 4: Factor of Safety for Geomembrane/Sand Interface on 1V:4H Slope