DESIGNER'S FORUM

Fundamental mistakes in slope design

By Gregory N. Richardson

SINCE WRITING THE JANUARY/February issue's kickoff of the designer's column, I have been working on several failed landfill final covers. After coming home one night from examining yet another messed up final cover, I decided that Barry Christopher and I could wait a month before we simplified geotextile filter design. Instead, I corralled two outstanding young engineers to help me describe some fundamental mistakes we are seeing in slope designs for landfill final covers. Thanks to Rick Thiel of Thiel Engineering, Oregon House, Calif., and Pete Verma of OHM Remediation Services Corp., Pittsburgh, Pa., for their help in presenting this evaluation of parameters that can influence the stability of the final cover slopes typically associated with new Resource Conservation and Recovery Act (RCRA) Subtitle D landfills.

Subtitle D requirements

Much of Subtitle D was lifted from similar requirements found in the RCRA Subtitle C regulations for hazardous waste landfills. This includes the use of a composite barrier in the final cover and some form of vegetative or erosion-control layer. In the June 26, 1992 issue of the *Federal Register,* the EPA clarified "composite barrier" to mean the use of a geomembrane over 18 inches of a soil having a permeability of less than 1×10^{-3} cm/sec.

This was a significant backstep from the use of 10^{-7} cm/sec clays in the covers. But in general, state regulators approach the Subtitle D final covers in much the same way they approach the standard RCRA final covers used in hazardous waste landfills. The problems discussed here result from the typical side slopes in Subtitle D landfills (> 4H:1V), as compared to maximum slopes common to hazardous waste landfills (< 5 percent).

Final cover stability on side slopes is influenced by both the interface shear strength that exists between the layers of the final cover and by the pore gas or water pressures that are allowed to develop within the cover profile. Since gas or pore water pressure relief layers are commonly constructed of granular soils or geonets, the limiting interface shear strength is typically the interface friction between the geomembrane and this pressure-relief layer. The concept of interface friction is as simple as remembering the brick sliding down the incline in your first physics course, and is generally well understood by most designers. The following cover failures resulted from excessive gas or pore water pressures at the geomembrane surface. The failures illustrate several simple, yet significant, design concerns.

Composite barriers on slopes

Photo 1 shows a small gas "whale" that developed during the construction of a fairly standard final cover. The designed final cover was to include a sand gas-collection layer, a composite barrier formed by a PVC geomembrane over a reinforced geosynthetic clay liner (GCL), an 18-inch-thick pore pressure relief system and a 12-inch vegetative-support layer. The system failed during placement of the pore pressure relief layer on the geomembrane (**Photo 2**). The sand drainage layer and the PVC geomembrane slid down the GCL. No movement of the GCL was observed.

Measurements made in one of the whales showed a gas pressure of an approximately 6 inch water column (0.21 psi or 31.2 psf). It was later shown that these pressures resulted from inadequate gas relief beneath the composite barrier. Neglecting seismic forces, the slope's stability can be evaluated using **Equation 1** (next page). This model neglects toe buttressing forces and the tensile strength of the geosynthetic components. For the failure shown in Photos 1 and 2, we can assume that the cohesion is zero such that the slope stability can be expressed as:

$$FS = \frac{\tan\phi\,[1 - u/(\gamma \cdot z)]}{\tan\beta}$$

Figure 1 shows this equation for the 4H:1V slope, $FS = 1$, and a range of interface frictions, soil cover weights and excess gas or pore water pressures. Recall that the actual failure occurred between the geomembrane and the GCL, and that the gas pressure was approximately 31.2 psf. Thus, the slope would have been stable if the in-

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Photo 1. This small gas "whale" developed during the construction of a fairly standard final cover.

Photo 2. Failure on this slope occurred during placement of the pore pressure relief layer over the geomembrane. The sand and PVC geomembrane slid down the GCL.

Equation 1

$$FS = \frac{c/(\gamma \cdot z \cdot \cos^2\beta) + \tan\phi\,[1 - u/(\gamma \cdot z)]}{\tan\beta}$$

where FS = factor of safety
γ = unit weight of slope material(s)
c = cohesion
ϕ = angle of internal friction of the assumed failure interface or surface
z = vertical depth to the assumed failure interface or surface
u = the pore water pressure above the barrier or the gas pressure below the barrier
β = the slope angle

terface friction was greater than 17.5 degrees. Unfortunately, the measured interface friction angle was only 17 to 18 degrees, and a sliding failure resulted during placement of the sand drainage layer.

Note that the drainage-layer sand also could have easily slid due to its saturation, resulting in $u \approx 90$ psf. The PVC geomembrane sliding, however, showed that the failed interface was between the geomembrane and the GCL. The significant lessons learned from this failure include:

1. Designers need to evaluate the stability of the cover as it is incrementally constructed.
2. Gas pressures *must* be effectively vented or relieved if a composite barrier is used on a slope.

The HELP model, capillarity and slopes

Photo 3 shows significant rills that formed when a very silty, fine-sand, vegetative-support layer "flowed" from a cover's side slopes (the slopes were as steep as 3H:1V). The final cover has 6 inches of topsoil, 18 inches of the vegetative-support layer, and a single-bonded geonet drain over a geomembrane. Examination of the areas of failure showed that the geonet had not slid on the geomembrane—failure occurred at the interface of the silty sand vegetative layer and the geotextile bonded to the drainage net.

The silty sand had a friction angle of 31 degrees and was very fine (more than 30 percent nonplastic fines). The permeability of the silty sand was 1 x 10^{-4} cm/sec. A HELP analysis was performed to evaluate the geonet design and the moisture in the vegetative-support layer. The analysis indicated an acceptable inflow into the geonet and little change in the moisture content of the silty sand vegetative-support layer. Using the same slope stability analysis previously presented, **Figure 2** shows the effect of saturating the sand and topsoil on the sliding factor of safety. The higher the saturated zone is allowed to rise in the cover profile, the lower the factor of safety. Given that the interface friction between the geotextile and the silty sand is less than the sand's shear strength (e.i., ≤ 30 degrees), the factor of safety approaches unity for 3H:1V slopes if only the sand is saturated, and 4H:1V if the entire cover section is saturated.

Examination of the silty sand vegetative-support layer at the areas of failure showed that much of the silty sand was saturated. Such saturation results from the capillarity forces within the silty sand, and the capillary break formed at the geonet/geotextile/silty sand interface. The height of the capillary zone of saturation due to vertical infiltration into the silty-sand is more than 165 cm (65 inches) or more than the entire layer thickness. Recall that the HELP3 model does not incorporate capillary breaks so the vertically infiltrating water will continue to drain to the geonet even if capillary forces would prevent it in real life. It was clear, therefore, that the silty sand vegetative layer had saturated during the extremely wet weather the East Coast experienced the past two years. This dramatically reduced the sliding factor of safety and led to failure at the silty sand geotextile interface on the steeper portions of the slopes. Note that the silty sand layer will not begin to drain until it is saturated. Then, it will drain at a rate given by a unit vertical gradient.

We can learn several significant lessons from this failure:

1. The HELP model may significantly underestimate the saturation of layers above the lateral-drainage layer
2. The HELP model may significantly underestimate the rate of infiltration into the geonet by ignoring the capillary break, i.e., the unit gradient design method of lateral drains (Thiel, 1993) may be appropriate for lateral drains forming a capillary break.
3. Soils with a high percentage (> 15 percent) of nonplastic fines should be avoided on side slopes because the fines lead to saturation of the soil.

Closure

Obviously, a designer's ability to be sensitive to problems that can lead to side slope failures is heightened by exposure to the failures of others. The failure mechanisms presented here will certainly lead to extensive failure of many future covers. As designers, we must question the need for composite barriers on such side slopes and

Photo 3. This very silty, fine-sand, vegetative-support layer is "flowing" from a landfill cover's side slopes.

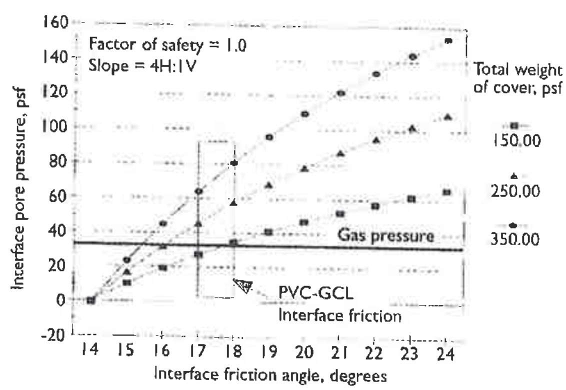

Figure 1. Allowable interface pressure

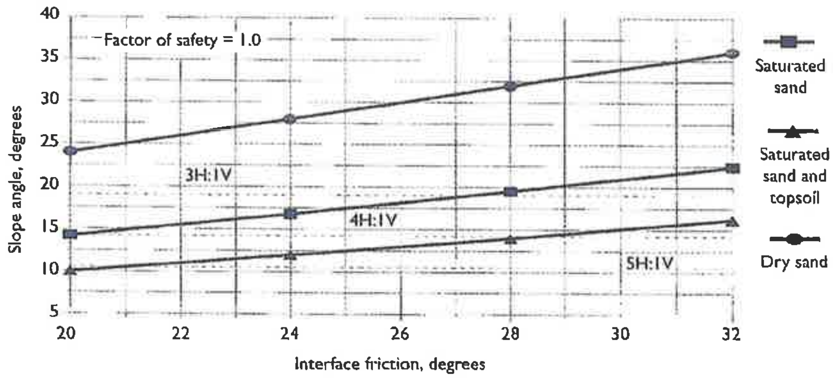

Figure 2. Slope vs. interface friction

better understand the effects of simple capillary forces on our designs. In the first example, nonplastic fines maintained the gas-venting layer in a near-saturated condition such that gas transmission was minimal. In the second example, nonplastic fines in the vegetative-support layer also created a saturated condition which led to unit gradient drainage that exceeded the capacity of the drainage geonet. Such conditions are not unique to these sites.

The role of direct shear testing for interface strength can be critical for side slope applications. A normally insignificant apparent cohesion of 5 psf or 10 psf can be very significant in evaluating cover stability on side slopes. It is important that the design limiting interface shear strength be tested for as follows:

- Perform direct shear testing using only an experienced laboratory, which means both an experienced technician and well-maintained and calibrated equipment.
- The normal load range should bracket the anticipated dead weight loading of the final cover, 100 psf to 700 psf usually is good.
- Hydration/loading rate may be important with composite interfaces.
- The shear displacement curves should be studied, and shear strength parameters selected in a conservative manner.

The two examples discussed here involved silty sand soils, so the apparent cohesion of the interface is minimal. This is not always true.

Regulatory derived tools must not be used with blind faith—the designer stands alone at the moment of failure. Regulatory review will do little to soften the liability exposure experienced after such failures. Your comments are welcome and should be sent to *GFR*, 345 Cedar St., Suite 800, St.Paul, MN 55101-1088 USA; fax 612/225-6966, e-mail gfr@ifai.com. GFR

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References

Thiel, R.S. and Steward, M.G. (1993)"Geosynthetic Landfill Design Methodology and Construction Experience in the Pacific Northwest," Geosynthetics '93 Conference Proceedings, Industrial Fabrics Association International, St. Paul, Minn.