Landfill cover failure prompts standards upgrade

By Nandakumaran Paruvakat and Gregory N. Richardson

It has been nearly five years since we investigated a landfill final cover that had become unstable during construction. That failure alerted us to field conditions that were not properly addressed by design standards then in place, opening a line of communication that has since enabled us to discuss and act upon some of these concerns. In this article, we will explore two factors that may lead to the failure of landfill side slopes during the actual construction process: the impact of cover-soil placement techniques, and the dangers of incremental cover construction. Both factors have been identified clearly in previous landfill side-slope failures. However, at the moment, they are not being considered by designers or, more importantly, by contractors.

Cover-soil placement over geosynthetics

The stability of slopes in landfills needs to be evaluated during construction and post-closure life. Methods of final-cover-stability analysis fall into two categories: the conservative infinite-slope model and the sliding block model. These stability-analysis methods are described in publications by Giroud and Beech (1989) and others. In the past, stability during construction has been addressed by considering the dead weight of the equipment that places cover soil over the geomembrane (McKelvey and Deutsch, 1991), or by considering an arbitrarily assigned braking force in addition to the equipment's dead weight (Druschel and Underwood, 1993). A principal drawback of accounting for construction stresses in this fashion is that the potential for local overstressing, especially directly beneath the tracks of the equipment that places cover soils, cannot be evaluated. This is critical particularly if the interface shear strength below the geomembrane is less than the interface shear strength above the geomembrane.

Construction shear stresses along interfaces

Soils are placed over geomembranes in landfill liners and covers either as drainage layers or protective soil layers. These soil layers typically are 1 to 2 ft in thickness. Low ground pressure (LGP) bulldozers are commonly used to push these soils upslope to form a layer of uniform thickness. The use of LGP bulldozers eliminates excessive puncture stresses on the geomembrane. However, of equal or greater importance is the fact that shear stresses will be caused along the interfaces of the geomembrane. These interfaces are the weakest planes in the system; therefore, they are the surfaces with the greatest potential for failures. If a membrane's lower interface has less shear resistance than its upper interface, the geomembrane will be subjected to tensile stresses if localized slip occurs or the displacement required to mobilize shear resistance along the lower interface is significant. The potential for localized slip will be maximized during construction.

It is necessary to evaluate stress conditions during construction because avoiding geomembrane stress is desirable. Current methods of evaluating the effect of construction equipment on the stability of landfill liners and final covers do not recognize local overstressing possibilities, because only the overall stability of the complete slope length is determined. Use of construction-equipment weight alone does not practically impact the computed safety factor, since the increased driving force is accompanied by an increase in the normal stress and a corresponding increase in the resisting force. Also, an arbitrarily assumed braking force is generally considered to be distributed over the whole slope length, and therefore has a very small effect on the computed safety factor.

The potential for large local interface-shear stresses exists when an overzealous equipment operator tries to push too much material up-slope at one time. If the resulting shear stresses below the geomembrane exceed the interface shear strength, localized slipping will result. When this occurs, the geomembrane will be subjected to tensile stresses. Geomembranes such as polyvinyl chloride (PVC) or linear low-density polyethylene (LLDPE) are flexible (i.e., they have low modulus and large failure strains) and will likely be stretched on the upslope portion of cover-soil placement locations. As a result, wrinkles will form directly beneath the bulldozer. Ultimately, tensile strains created on the upper reaches of the slope can reach failure strain portions and cause membrane tearing.

The unfortunate part of this scenario is that most construction-quality assurance monitors are unlikely to recognize this situation until it is too late. Usually, they focus on soil thickness and look for evidence of geosynthetic damage below the bulldozer's blade or tracks. No guideline has yet been developed to recognize local overstressing of membranes under the cover soils.

Analysis

Figure 1 illustrates the shear stresses transferred to geosynthetic interfaces by LGP bulldozer tracks as cover soil is being placed over a geomembrane. To achieve equilibrium, the total shear force transferred by the tracks to the soil below must be equal to the passive pressure on the bulldozer blade. So if the equipment operator were to push a high wall of soil ahead of the bulldozer instead of gradually shaving off the soil pile, passive pressures could exceed interface strength along the potential failure surface. Geomembrane failure could possibly result.
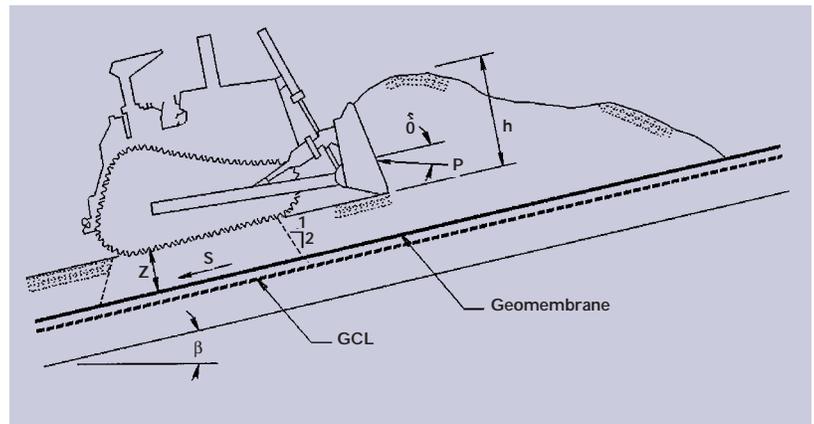

Figure 1. *Shear stresses caused by cover placement over the membrane.*

The factor of safety can be expressed as follows:

Equation 1:

$$\text{Factor of safety during construction} = \frac{(\gamma z + \sigma)}{(\gamma z + \sigma)} \cdot \frac{\cos \beta \tan \phi}{\sin \beta + s}$$

where:

γ = unit weight of soil being placed

z = thickness of soil layer below equipment track

σ = vertical stress at interface due to equipment track

β = slope angle

ϕ =   interface friction angle

s =   shear stress transmitted to interface due to passive resistance of soil being pushed by bulldozer

The total force generated by the bulldozer, P, can be assumed to equal the passive resistance of the soil being pushed as follows:

Equation 2:

$$P = \tfrac{1}{2}\, \gamma\, h^2\, B\, K_p \cos \delta$$

where h is the height of the soil being pushed, Kp is the passive earth-pressure coefficient for the soil, B is the width of the bulldozer blade, and δ is the friction angle between the blade and the soil being pushed. The actual stress transferred to the underlying geomembrane, s, is influenced by the bulldozer's area of contact, A, with the geomembrane.

The geomembrane contact area is a function of the thickness of the soil layer being placed and the vertical stress distribution of the track load through that layer. The vertical stress distribution effect is approximated by assuming a 1H:2V distribution of the vertical stresses generated by the bulldozer. This leads to a reduction factor for vertical stress distribution, c, defined as:

Equation 3:

$$c = l \cdot b\, /\, (l+1)(b+1)$$

where l and b are the length and width of the bulldozer's tracks, respectively.

Assuming a typical soil-placement thickness of one foot, Equations 2 and 3 can be substituted into Equation 1 to determine the maximum height of soil that can be pushed. The factor of safety equals one:

Equation 4:

$$h = \sqrt{\frac{4\, bl\, (\gamma z + \sigma)\, (\cos \beta\; \tan \Phi - \sin \beta)}{c\, \gamma\, B\, kp\; \cos\; \delta}}$$

The maximum thickness of cover soil that can be pushed by a bulldozer before localized failure occurs along the interfaces of liner or cover systems can be estimated with the above relationship. On examining the terms on the right hand side of the equation, it is apparent that the critical depth is a function of the equipment characteristics (B, b, 1, σ) soil unit weight (γ), slope angle (β), interface strength (Φ), and cover soil strength (K_p, δ).

Typical application
Figure 2 shows the maximum height of soil that can be safely pushed by a CAT D3B LGP bulldozer for an interface with a friction angle of 18º and a range of slope and soil properties. Note that the allowable "push"

height decreases with increasing slope and increasing soil shear strength ($K_p \cos\delta$). Interestingly, the critical blade height for typical side slopes of 4H:1V ($\beta \geq 14°$) is actually less than the 29-plus-in. height of a typical bulldozer blade. Thus, even limiting the height of soil being pushed to the dozer blade height may lead to construction stresses that produce tension in the geomembrane. For a common 4H:1V side slope, **Figure 3** shows that the critical soil-push height increases with the increasing weight of the bulldozer and thickness, H, of soil the 'dozer is operating on.
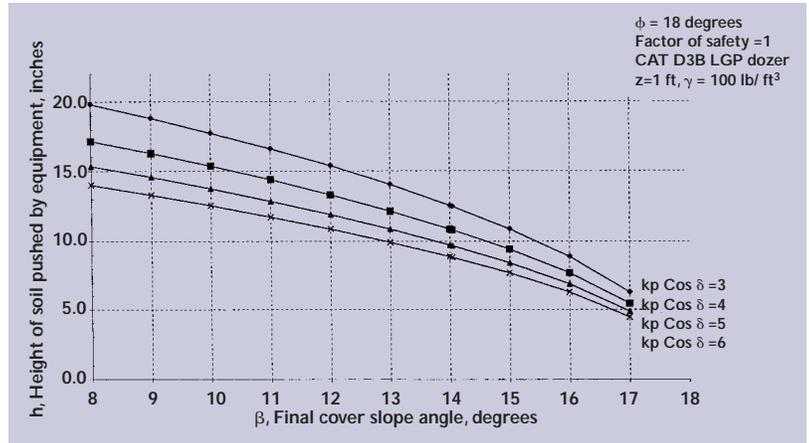

Figure 2. Cover-slope vs. height of soil pushed by equipment.

Final cover stability during construction

Final cover systems for landfill side slopes are designed to ensure the slopes' long-term stability under various service load conditions. However, recent slope failures have shown that many of these cover systems are particularly sensitive to rain-induced failure during various stages of construction. **Figure 4** shows a typical section for a landfill side slope final cover. The slope must be analyzed for final and incremental failures from erosion and also for forces generated by seepage into the cover.

Soil erosion

Large-scale erosion of final-cover systems on landfill slopes is most likely to occur during construction of the granular drainage layer. Though the initial runoff coefficient is likely to be smaller for permeable granular materials, the sand drainage layer may quickly saturate (see below) and have nearly 100% runoff. Therefore, the amount of erosion needs to be evaluated (estimated) using methods such as USDA Universal Soil Loss Equation (USLE–USDA, 1975). The USLE predicts average annual soil loss as the product of six quantifiable factors. The equation is:

Figure 3. Equipment weight vs. height of soil pushed by equipment.

Equation 5:

$A = R\,K\,L\,S\,C\,P$

where:

A = average annual soil loss, in tons/acre
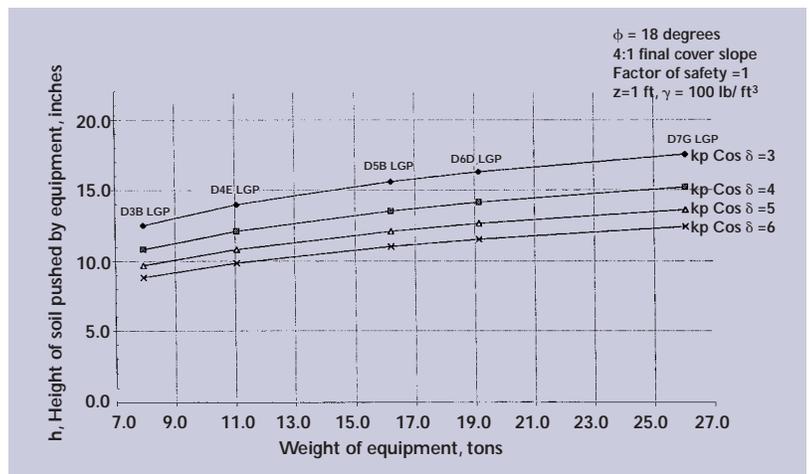R = rainfall and runoff erosivity index (R = 200, typical Midwest)

K =   soil-erodibility factor, tons/acre (K= 0.45 sand)
LS =   slope-length/steepness factor (LS = 5.8 for 4H:1V slope 100-ft long)
C =   cover-management factor (C = 1.0)
P =   practice factor (P = 1.0)

Typical data input to this equation for a 4H:1V slope in the Midwest is shown, indicating a potential annual soil loss of 522 tons/acre/year for the exposed sand. *This indicates a significant soil loss potential due to surface erosion for the exposed sand*. 522 tons/acre/year translates into a uniform erosion of more than 2.5 in. Considering that rill and gully erosion are very likely during construction, it is conceivable that, in many areas, sand could be completely eroded from the membrane surface.
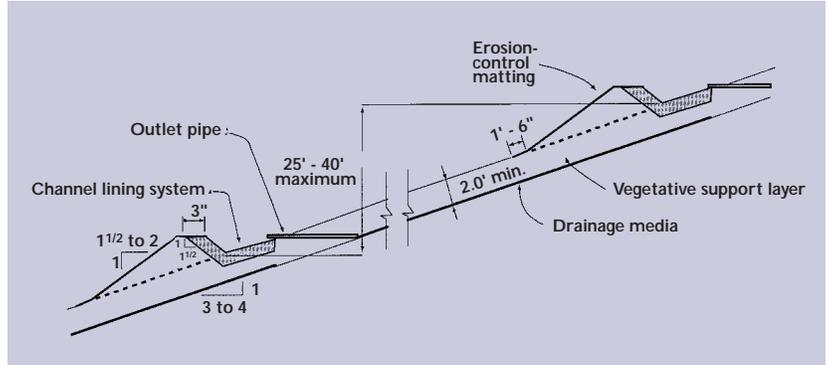

Figure 4. *Typical landfill side slope.*

Seepage forces
The seepage force in a saturated soil layer is given by:

Equation 6:

$$F_{seep} = \gamma w\ a \sin \beta$$

where β is the slope angle and a is the vertical thickness of the soil layer. Using an infinite slope model, the factor of safety against slope failure is given by the following:

Equation 7:

$$FS = \frac{\text{Resisting Forces}}{\text{Driving Forces}} = \frac{\gamma_b \cos \beta \tan \delta}{\gamma_{SAT}\ a \sin \beta}$$

where γ_b is the buoyant unit weight of the soil layer and γ_{SAT} is the saturated unit weight of the soil layer. This equation can be further reduced to:

Equation 8:

$$FS = \frac{\gamma_b}{\gamma_{SAT}} * \frac{\tan \delta}{\tan \beta} \approx 0.5 \frac{\tan \delta}{\tan \beta}$$

The exposed sand can saturate in as little as 2.5 hours (assuming a drainage layer 1.0 ft thick with k = 1.0E-03 cm/sec and porosity of 0.3), such that Equation 6 is a real possibility. For a 4H:IV slope, Equation 8 indicates that the required interface friction between the sand and the geomembrane under seepage flow conditions would have to exceed 26.5º. This is an exceptionally high interface-friction angle for a sand-textured geomembrane interface.

Even if a steady seepage condition (required for causing the seepage forces) has not been established in the drainage layer but the layer contains fines, saturation alone can cause slumping of the soils. Shear strength of loose saturated "cohesive" soils could be very close to that at their liquid limit. Increased weight of the drainage layer due to saturation alone can cause sloughing.

During the typical layer-by-layer construction of a final cover, many acres of the drainage material on the slopes may be exposed at one time. If significant rainfall occurs during this construction stage, seepage forces will develop quickly, which will significantly increase the required interface friction angle. Clearly, rainfall of only moderate duration—as little as two and one-half hours—can result in a sliding failure of the sands that form the drainage layer.

Construction impact

Crews must avoid placing a large area of the sand drainage layer when there is the potential for major precipitation. This may be accomplished by either limiting the exposed area of drainage sands before the vegetative cover is placed. At a minimum, the specifications should warn that exposed sand drainage layers on slopes have a high likelihood of failing during prolonged rainfall.

Summary

The designer must evaluate intermediate steps in the construction of a final cover system on side slopes to avoid potential construction-generated long-term damage to the individual components and short-term catastrophic surface-water related failures of the partial system. The authors recommend that, at a minimum, designers consider the following restrictions when making their design specifications:

•The height of soil pushed by a bulldozer operating over a geomembrane must be limited to reduce the potential for soil shear failure against the geomembrane.

•The area of granular drainage layer placed on slopes should be limited to reduce the potential for catastrophic erosion.

At the very least, the CQA personnel and contractor should be educated as to the dangers that these construction practices can create for the final cover. We encourage readers to provide comments on these or similar cover-construction problems and solutions that have been implemented. GFR

Nandakumaran Paruvakat, Ph.D., P.E., is a designer at Foth & VanDyke, Green Bay, Wis.

Gregory N. Richardson, Ph.D., P.E., is principal of G.N. Richardson & Associates, Raleigh, N.C.

References

Druschel, S.J. and Underwood, E.R. 1993. Design of Lining and Cover System Side slopes. *Geosynthetics '93 Conference Proceedings*. St. Paul, Minn.: IFAI.

Giroud, J.P. and Beach, J.F. 1989. Stability of Soil Layers on Geosynthetic Lining Systems. *Geosynthetics '89 Conference Proceedings*. St. Paul, Minn.: IFAI.

McKelvey, J.A. III and Deutsch, W.L. Jr. 1991. The Effect of Equipment Loading and Tapered Cover Soil Layers on Geosynthetic Lined Landfill Slopes. Fourteenth Annual Madison Waste Conference Proceedings. Madison, Wis.: Department of Engineering Professional Development, University of Wisconsin–Madison. Paruvakat, N. and Sturzl, M.R. 1998. Cover Soil Placement over Geosynthetics. *20th International Madison Waste Conference Proceedings*. Madison, Wis.: Department of Engineering Professional Development, University of Wisconsin–Madison.

U.S. Department of Agriculture, Soil Conservation Service. 1975. Procedure for Computing Sheet and Rill Erosion on Project Areas, *Technical Release No. 51*. Washington, D.C.: USDA, SCS, Engineering Division.