Case History: Cost-Effective Low-Maintenance Final Cover on Steep Slopes (4th DRAFT - Final)

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This article presents the case history of a landfill closure design and construction for the City of Willits, California. The centerpieces of this story are the analytical design solution and the unique geocomposite pressure-relief/barrier/cushion material that were developed to address veneer stability concerns on steep slopes. The simple and cost-effective final product – 10 inches of rock over a single geocomposite – may prove to be a useful consideration for similar remote, steep-slope situations.

Problem Statement

The City of Willits is located in Mendocino County in Northern California. After 26 years of operation, the 20-acre landfill reached full capacity and was ready to receive final cover during the 1999 construction season. However, past landfill practices left relatively steep waste slopes on the site. Numerous 2.5(H):1(V) (21.8°) slopes, with some slopes as steep as 2(H):1(V) (26.6°), presented a challenge for a final cover design. In addition, the site receives up to 110 inches of rain per year, which poses slope stability and erosion design concerns.

Although a single compacted-clay cover may have been acceptable for most of the site from a regulatory point of view, there was not an adequate source of clay available. Additionally, it was recognized that the performance of clay covers is questionable as long-term barriers to infiltration. Also, since part of the site was underlain by a modern Subtitle D lining system, those areas would require a geomembrane-based cover. A composite cover system, however, was not required. Geomembrane options were immediately favored for the entire site. Various traditional geomembrane cover designs were evaluated for cost and technical feasibility. The primary technical challenge was veneer slope stability on the existing steep slopes. Another significant consideration was the long-term erosion potential of these steep slopes, with average slope lengths on the order of 100 feet, and some up to 160 feet long.

Initial Design Concept

During the design development phase, the idea of having a permanent, exposed rock-covered slope was presented. The advantages of this would be to provide an erosion-resistant armor that would require very little maintenance. The perceived disadvantage would be its appearance. However, the landfill is remotely located with limited drive-by and neighbor view-points. Someone coming across the landfill might view it as a huge rock stockpile, among other soil-borrow and forest-harvest activities. One further potential advantage of a rock armor is that it might prove less attractive to the local bear population that had grown fond of gleaning snacks from the active landfill. This concept was pursued, and ultimately led to the unique final design that was implemented.

The idea was to provide a 10-inch thick layer of exposed rock over a geomembrane. Three critical design conditions presented themselves in this case. First, seepage forces during intense rains would cause a veneer-stability problem for interfaces above the geomembrane. Second, with such a low normal load over the geomembrane on such steep slopes, relatively small pore pressures caused by landfill gas generation could cause instability for interfaces below the geomembrane. Third, knowing that coarse, open rock would be required to meet the stability concern on top of the geomembrane, puncture protection (in the form of a geotextile cushion) would be required for the geomembrane to survive construction.

Design & Analysis

Having a thin veneer of soil (in this case rock) makes the veneer layer stability especially vulnerable to pore pressures. Even relatively small pore pressures may offset a significant portion of the effective stresses that result in frictional shear resistance. From a stability point of view, a frictional veneer system that has a factor of safety greater than unity benefits from additional normal loading to help offset pore pressures. That is why winter snow and ice loading in this case is not a critical design condition, because the snow and ice would actually increase the normal load and increase the factor of safety. The critical slope stability design conditions are (1) the exposed rock layer in a high-intensity rainstorm that would cause pore pressures above the geomembrane, and (2) the first season after construction when the landfill gas pressures might be the at the maximum levels that the cover system would experience.

The slope stability issue above the geomembrane was evaluated using the infinite-slope-stability method presented by Thiel and Stewart (1993). In this case, the vertical water infiltration was selected to be the estimated maximum direct rainfall intensity of 7 inches per hour. For the 26.6° slope inclination on a 150-foot long slope, an average water flow depth of 0.9 inch (1/2 maximum depth) was calculated assuming a rock permeability of 10 cm/s. For the 21.8° slopes, the average flow depth was calculated to be 1.2 inches. Using an assumed interface friction angle of 40° (a value greater than 45° was eventually measured), the factor of safety was calculated as follows:

$$FS = \frac{c + [h\gamma \cos\beta - u_W] \tan\phi}{h\gamma \sin\beta} \qquad (1)$$

where c = interface cohesion parameter (psf); h = thickness of the rock layer normal to the slope (ft); γ = average total unit weight of the rock layer including both the moist and saturated zones (pcf); β = slope angle (degrees); u_w = pore water pressure at the base of the rock layer caused by water flowing parallel to the slope (psf); and ϕ = interface friction parameter (degrees).

For the Willits project this equation was solved for the 2(H):1(V) (26.6°) slope as:

$$FS = \frac{0 + \left[0.83 \cdot (110) cos(26.6) - \left(\frac{0.9}{12} \right) \cdot (62.4) cos(26.6) \right] tan(40)}{0.83 \cdot (110) sin(26.6)} = 1.6 \quad \therefore OK$$

Similar calculations for the 21.8° slopes resulted in an even higher factor of safety. The design for the conditions of gas pore pressures below the geomembrane was more difficult because no design methodology previously existed. To illustrate just how sensitive the final cover would be to gas pore pressures, consider the infinite-slope geometry shown in **Figure 1**, with gas pore pressure "u_g" being exerted from below. The infinite-slope stability equation for this situation is the same as Eqn (1) above, except that u_w is replaced by u_g. **Figure 2** illustrates the relationship between factor of safety and gas pressure for various assumed interface friction parameters. For example, assuming an interface friction angle of 40 degrees below the geomembrane, the gas pressure would have to be 6.5 inches of water column to create a factor of safety below unity. This is a precarious situation indeed, noting that gas pressures of 8-16 inches of water column have been measured at one site that experienced a final-cover slope failure (Richardson et al. 2000). To address this design condition, a gas pressure-relief layer is required below the geomembrane to limit the gas pressure to a maximum design value, and a specified minimum interface shear strength would be required.

Thiel (1998) presents a methodology for designing gas-relief layers below final covers that was developed as a result of this project. The result of using this design methodology is that a blanket gas-relief layer with intermittent strip-drains is incorporated into the final cover. The required gas transmissivity and spacing of the strip drains are given by the following equation:

$$u_{max} = \frac{\phi_g \gamma_g}{\theta_g} \left(\frac{L^2}{2} \right) \qquad (2)$$

where u_{max} = the maximum differential gas pressure from the mid-point between strip drains and the edge of a strip drain; ϕ_g = the assumed gas flux in units of volume per surface area per time, such as cubic feet per minute per square foot of cover (cfm/ft^2); γ_g=the gas unit weight; θ_g = gas transmissivity of the gas-relief layer; and L = the spacing between the strip drains.

For the City of Willits final cover, the maximum gas pressure under the cover at the mid-points between strip drains was established at 3 inches of water column.

This would result in a minimum static slope stability factor of safety of nearly 1.4 (see Figure 2). The back-pressure in the strip drains was calculated to be approximately 1 inch of water column. Therefore u_{max}, which is the difference between the maximum pressure and the back-pressure, would be 2 inches of water column. Eqn (2) was solved for this project such that using a strip-drain spacing L of 50 feet would require the gas-relief layer to have a minimum gas transmissivity θ_g of 0.001 cfm/ft (1.5×10^{-6} m^2/s).

Material Selection and Fine Tuning

The preliminary design concept and analyses described above resulted in the following design parameters:

- 10 inches of rock with a minimum hydraulic conductivity of 10 cm/s
- Minimum interface shear strength above the geomembrane of 70 psf under a normal load of 100 psf (equivalent friction angle of 35°).
- Minimum interface shear strength below the geomembrane of 84 psf under a normal load of 100 psf (equivalent friction angle of 40°).
- Gas-relief layer with a minimum gas transmissivity θ_g of 0.001 cfm/ft (1.5×10^{-6} m^2/s).
- Strip-drain spacing of 50 ft. (There are also minimum flow capacity requirements for the strip drains which are beyond the scope of this paper.)

Once these design parameters were established, materials needed to be specified that could meet these parameters. The rock cover material was easy to specify. A local quarry could provide 2-inch uniform crushed rock to meet the requirements. For the other layers, a standard approach would be to separately procure and provide (from bottom to top) a gas-relief layer made either from a geosynthetic or a granular material, a highly textured geomembrane (most likely polyethylene because it is the material that can provide the most substantial texturing), and a geotextile cushion. However, a more intriguing and potentially cost-saving alternative presented itself in the form of a triple geocomposite that comprised the functions of gas relief, barrier layer, and cushion. The product consisted of the following 3 elements, from bottom to top:

- Gas-relief layer: A 28 oz/sy nonwoven-needlepunched (NWNP), polyester (PET) geotextile made from 45-denier trilobal, water-quenched fibers that have been lightly-to-moderately needle-punched. This results in a thick, coarse material with good transmissivity.
- Barrier layer: 30-mil PVC geomembrane.
- Cushion layer: 16 oz/sy NWNP-PET geotextile made from 15-denier fibers.

The three layers are bonded together using heat-activated thermoset polymer pellets. The bond process can be controlled by pellet density, temperature, and pressure to provide a specified peel strength, and results in a shear strength well in excess of the design requirements. To prevent intrusion of the 18-mesh pellets into the highly porous gas-venting fabric, a 6-oz/sy NWNP fabric is punch-bonded to the 45-denier fabric.

This product, which is not a standard product, was pursued because of its potential cost effectiveness and greatly simplified installation. Samples of the product were procured and tested to verify conformance with the design requirements described above. The following tests were performed:

- Direct shear tests were performed for all interfaces from the subgrade to the rock covering. All results met the minimum requirements.

- The protection-effectiveness of the cushion between the rock and the geomembrane was tested by a full-scale field test using crushed rock from the local quarry and construction equipment. Results were excellent with no damage even for quite abusive construction conditions.

- The gas transmissivity of the bottom geotextile was the most difficult to evaluate since there is no standard test method for measuring gas transmissivity. In conjunction with the geosynthetic manufacturer and a testing laboratory, a test method and analysis were developed as a result of this project. Details are provided in Thiel (1999). The results showed that the thick, coarse bottom geotextile would provide adequate gas transmissivity under the anticipated moist field conditions and normal loads. In general, the test values exceeded the minimum requirement by a factor of 10. This factor of safety was considered adequate to address the potential for long-term biological and chemical degradation of the geotextile transmissivity. The generation of landfill gas pressures reduces substantially over time, with most of the reduction occurring within the first 5 years.

Construction and Performance Results

Once the difficult design decisions had been made and justified as described above, the construction process was found to be very straightforward. Following is a description.

- The landfill subgrade had been previously prepared by the owner.

- The strip drains (ADS flatpipe) were laid out on a pre-designed grid with occasional vents (Photo 1).

- The geocomposite arrived in 25-foot wide custom roll lengths averaging 150 feet long, with some up to 330 feet long (Photo 2). The edges along each side of the rolls had a 12-inch plastic film inserted above and below the PVC-geomembrane during manufacturing. This kept the geomembrane free of adhesive pellets to allow wedge welding to take place uninterrupted (Photo 3). The adhesive on the unbonded fabric along the edges allowed the overlapped fabric to be easily heat bonded with air (Photo 4). The design allowed the roll ends to be overlapped instead of welded.

- The rock was end dumped and dozed into place (Photo 5). Initial test inspections were made to determine if dozing up or down the slope would affect the geocomposite. Normally it is advised to push only up-slope, and the specifications were so written. However, for the combination of equipment and materials used for this job it was found acceptable to push down slope. The rock covering went much smoother than expected.

Performance

Normally, gas pressure problems with final covers will manifest during an advanced stage of construction. In this case, there were no gas-bubble problems during or after construction. The final cover (Photo 6) has successfully endured several intense rainstorms since the end of construction in October 1999.

Conclusions

This project demonstrated that open-minded cooperation and communication between the owner, engineer, and geosynthetic manufacturer can result in a cost-effective value-added project. In addition, The City of Willits final cover project resulted in several innovations that include:

- New design and testing methodologies for addressing gas pore pressures beneath landfill final covers.

- Development of a new value-added geocomposite that demonstrates the versatility and usefulness of geosynthetics in solving environmental/civil design issues.

- A simple, cost-effective, low-maintenance final cover design that incorporates a geomembrane in a steep-slope application.

Acknowledgements

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Rick Thiel provided slope stability and design analysis consulting for the project, and is president of Thiel Engineering in Oregon House, CA. **Randy Wall** was the project manager and engineer of record for the project, and is a senior engineer with the IT Group in Sacramento, CA.

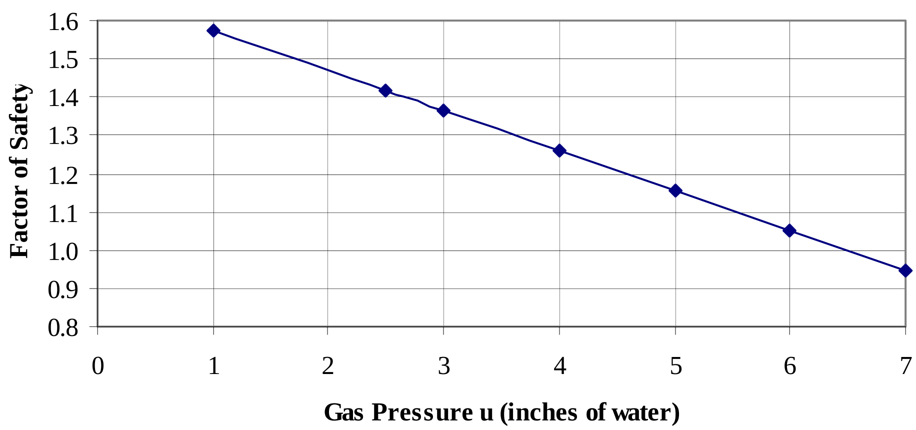

Figure 2 - Gas Pressure vs Factor of Safety for Willits Landfill Cover