Avoiding Surprises in Slope Stability

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The ability to analyze and compute factors of safety for slopes has increased greatly in the last decade. This ability has increased to the point that factors of safety can be computed using even complex limit equilibrium methods, such as Morgenstern and Price (1960), which used to be too computationally intensive for microcomputers; the finite-difference method, such as FLAC/Slope; and the finite-element method, such as PLAXIS. In addition, these methods can be performed using two-dimensional and three-dimensional geometries.

Despite these computational resources, these methods still need geotechnical insight to ensure the factor of safety is computed for all possible failure mechanisms and the relevant loading conditions, soil properties, and boundary conditions. Unfortunately, even the best limit equilibrium analyses and numerical methods do not always accurately predict field behavior if a condition or material property was not properly modeled or considered. In developing proper models, users should have an open mind to all potential failure mechanisms, material properties, and boundary conditions such as pore-water pressures, before initiating the analysis so all possible mechanisms and slope conditions are considered.

A Unique Case History

Hazardous and municipal solid waste (MSW) landfills in the U.S. must have a low hydraulic conductivity composite liner system with an overlying drainage system. The bottom liner system for a waste containment facility often consists of a low hydraulic conductivity compacted soil liner (CSL) overlain by a geomembrane. Figure 1 shows the geomembrane covering a portion of the CSL during construction of a slope prior to failure. The stability of these liner systems is usually controlled by the interface shear resistance between the CSL and overlying geomembrane. As a result, failure surfaces through the CSL are rarely analyzed for slope stability purposes because it is assumed that a soil/soil interface in the CSL will be stronger than the overlying soil/geomembrane interface.

During the fall of 2001, a slope failure occurred at an MSW landfill near Waverly, OH, causing significant damage to the liner system. This failure occurred after landfill personnel,

Figure 1. Slope overview and placement of geomembrane over completed low permeability compacted soil liner. (photo by Craig L. Walkenshaw)

who were constructing the liner system, decided to save time by spreading the granular drainage layer (GDL) from the top of the slope to the bottom on the completed geomembrane (Figure 2). Landfill personnel who were performing the construction decided to spread the GDL from slope top to bottom because there was easy access for gravel trucks to unload at the top of the slope instead of accessing the bottom of the slope.

Figure 2. Overview of sideslope and protective sand placement on geomembrane from top of slope to bottom.

The repair of this slope failure delayed the construction for over a month and cost the facility hundreds of thousands of dollars. The slope was repaired by removing the GDL, the geomembrane, and the CSL down to a firm base, which involved the subgrade in some areas. Afterwards, the sideslope liner system was reconstructed, consisting of the CSL and the geomembrane, and then placing the GDL from the bottom to the top of the slope. A different on-site borrow source with a lower plasticity and in situ moisture content was used for the CSL. The CSL was placed in loose lifts a maximum 0.2 m thick with compaction to 95 percent of maximum Standard Proctor dry density. Each lift was kneaded into the underlying lift to help ensure integrity between the lifts.

This case history is surprising because slope movement did not occur at the CSL/geomembrane interface but along a soil/soil interface within the CSL. It is surprising that a soil/soil interface would be weaker than a geomembrane/CSL interface, especially at the low effective normal stresses present. A low effective normal stress is significant because the low applied stress may not be sufficient to fully engage the texturing of the geomembrane into the upper surface of the CSL during placement of the overlying drainage material. This condition nearly always results in the weakest interface developing at the CSL/geomembrane interface and the geomembrane sliding downslope along the surface of the CSL.

Site Description and Construction

The case history site is located in the pre-glacial Teays River Valley in Pike County in south-central Ohio. The fluvial deposits at the project site range from zero to 25 m thick and are mostly clay with liquid limits ranging from 45 to 56 percent, more than 95 percent passing the No. 200 sieve, and 55 to 73 percent being smaller than 0.002 mm. This clay soil was used to construct the CSL used in the liner system. During construction, a slope failure occurred in a nominal 3H:1V sideslope during placement of the granular drainage media. The composite liner system consists, from bottom to top, of 1.5 m of CSL, a 1.5-mm-thick HDPE geomembrane that is textured on both sides, and 0.3-0.6 m of protective/granular drainage sand.

The granular drainage layer (GDL) was placed during sideslope construction from the top of the slope to the bottom. This placement practice is not recommended because the GDL is unbuttressed and imparts shear stresses in the liner system due to the weight of the GDL and equipment operating on the inclined slope. Thus the recommended practice is to place the GDL from the bottom to the top of a slope so the GDL buttresses itself and reduces the shear stresses and shear displacement applied to underlying liner system interfaces. Even though placement from top to bottom is not recommended, it is used because of easy access and ease of pushing material downslope versus upslope.

Figure 3 shows the damaged geomembrane near the toe of the slope due to the slope movement. This type of geomembrane damage is typical when slope movement occurs because the geomembrane and other geosynthetics easily tear under the applied shear and normal stresses. A view from upslope of the slide movement shows the drainage material overlying the torn geomembrane and the sliding surface below the torn geomembrane (Figure 4). The failure surface is located at a depth of 75–150 mm into the CSL and not at the geomembrane/CSL interface.

Figure 3. Failed slope and damaged geomembrane.

More importantly, Figure 4 shows that the failure surface extends the entire length of the slide because the GDL and geomembrane are missing from upslope of the slide mass to where the slide block stopped. A close look at the torn geomembrane (Figure 5) clearly shows the slide surface occurring below the torn geomembrane, from which it may be concluded that sliding occurred in the compacted CSL first and then the geomembrane tore as a result of this movement.

Observations indicate that the sliding surface did not correspond to a specific compacted lift interface. This is evident

Figure 4. Slide surface and torn geomembrane.

Figure 5. Close-up of torn geomembrane which shows slide surface is below geomembrane/CSL interface.

because the sliding surface was only about 75-150 mm below the top of the compacted CSL. Generally, a lift interface is deeper than that at the top of the compacted CSL, unless some additional material was placed on the top of the last lift so the final CSL thickness meets the regulatory requirement of about 1.5 m or final grade. Figure 5 shows the failure surface is slickensided, having a polished appearance, caused by clay particle reorientation into face-to-face/parallel arrangements. Slickensides occur when large strains or displacements are experienced along a well-defined failure surface.

The compaction specification for the CSL to achieve a saturated, vertical hydraulic conductivity of less than 1×10^{-7} cm/sec is a relative compaction of 95 percent or greater, based on standard Proctor compaction and a moisture content of > 1 percent wet of optimum. From laboratory test results, the maximum dry unit weight and optimum moisture content for the CSL soil are 15.8 kN/m^3 and 21.6 percent, respectively. Daily field compaction reports provide insight to the compaction procedure, which included a maximum loose lift thickness of about 200 mm, compacting each lift with a minimum of 12 passes using a sheepsfoot compactor to achieve the minimum compaction and to break up soil clods in the compacted fill.

Table 1 presents a summary of the field dry unit weights and moisture contents for Lifts 3 through 6. Lift 6 corresponds to the top of the CSL, and Lifts 5 through 3 are located sequentially below the top of the CSL. The field compaction data shown in Table 1 were obtained using a nuclear density gauge. All of the tests for Lifts 3 and 4 passed the requirements of ≥ 95 percent Standard Proctor relative compaction and a moisture content of ≥ 1 percent wet of optimum. Not all of the field tests for Lifts 5 and 6 passed the moisture content requirement, but the average moisture contents (25.4 and 26.0 percent, respectively) were well above 1 percent wet of optimum.

These field test results indicate that the CSL was compacted wetter than optimum moisture content for Lifts 5 and 6, whose highest moisture content was 26.7 percent, than for Lifts 3 and 4, whose highest moisture contents were 25.5 and 26.5 percent, respectively. In addition, the average moisture contents for Lifts 5 and 6 (25.4 and 26.0 percent) are slightly higher than for Lifts 3 and 4 (25.0 and 24.2 percent). Thus, the moisture contents of Lifts 5 and 6 are slightly higher than Lifts 3 and 4, although some of the field moisture contents for Lifts 5 and 6 are near optimum moisture content, and thus below the required +1 percent wet of optimum.

After the slide, four moisture content specimens were obtained from the exposed slide surface. The measured moisture contents are 24.9, 28.6, 26.5, and 26.9 percent, with an average of 26.7 percent, which is in agreement with the as-compacted average for Lift 6 of 26.0 percent. The slide surface moisture contents correspond to an average moisture content of about 4.1 percent above the optimum moisture content. The post-slide moisture contents indicate that CSL moisture will likely remain constant or increase due to the presence of the overlying geomembrane heating the surface of the CSL and water vapor condensing under the geomembrane. As a result, designers should perform shear testing at the as-compacted or higher moisture contents, not lower moisture contents, to estimate field shear strength values.

Table 1. Range of field compaction parameters for Lifts 3 through 6 of the CSL

Lift Number	Range of Dry Unit Weight (kN/m^3)	Standard Proctor Relative Compaction (%)	Range of Compaction Moisture Content/ Average (%)	Moisture Content Above Optimum (%)
3	15.0-15.9	95.0-100.0	22.8-26.5/25.0	1.2-4.9
4	15.1-15.3	95.5-97.1	23.2-25.5/24.2	1.6-3.9
5	15.0-15.3	95.3-97.2	22.3-26.7/25.4	0.7-5.1
6	15.0-15.7	95.1-99.8	21.6-26.7/26.0	0-5.1

Limit Equilibrium Analysis of Slide

Back-analysis of the observed sideslope instability was performed using Janbu, Spencer, and Morgenstern and Price stability methods. Using an effective stress friction angle of 19.5 degrees based on consolidated-undrained $\overline{CU}$) triaxial compression tests with pore-pressure measurement on specimens of the CSL, the back-calculated effective stress cohesion ranges from 0.70 to 1.25 kPa for an average of about 1 kPa.

For comparison purposes, the same effective stress strength parameters were used to calculate the factor of safety if the GDL was placed from the slope toe and pushed up the slope instead of downslope with all of the other variables remaining the same. The Morgenstern and Price stability method yielded a factor of safety of about 1.3 instead of about unity for top-down placement of the GDL. The factors of safety for the Janbu and Spencer stability methods are 1.30 and 1.35, respectively, for soil placement from the bottom to the top of the slope. Therefore, placing the cover soil from the bottom to the top of the slope would have increased the factor of safety by 30 to 40 percent, meaning the slope probably would have been stable if the cover soil had been placed from the slope bottom to the top.

Other Stability Situations

The interim or temporary slopes that can develop during earthwork construction or waste containment development can also provide stability surprises. Most state agencies require slope stability analyses only for the final configuration of a waste containment facility. As a result, stability analyses are usually not performed for slope conditions that develop during fill and construction of the various phases or cells of the landfill. This is important because these interim or temporary slopes frequently involve toe excavation to join the existing and new liner systems or for additional waste capacity while additional waste placement continues at the top of the slope. These two potentially destabilizing activities, cutting the toe and loading the top of the slope, can occur simultaneously.

Toe excavations should be carefully modeled in the stability analyses to reflect the exposure of weak materials, the reduction in buttressing stresses, and different boundary conditions such as dissipation or build-up of pore-water pressures to prevent a slope stability surprise. Other interim conditions that should be modeled include blasting in or near the slope to facilitate excavation and seismic activity that may occur during the period in which the slope toe is unbuttressed.

Surprising slope stability can also develop when clayey embankment slopes, such as levees, undergo weathering. These slopes are usually designed with the peak strength in mind and an appropriate factor of safety. However, slopes in overconsolidated fissured London Clay revealed that the clay around some of these slopes can weaken with time, to the point that their strengths are nearly as low as the same material when normally consolidated. This condition is termed "fully softened shear strength."

Field observations indicate that high plastic clays in embankments that are subject to cyclic drying and wetting can suffer a similar strength loss, especially at shallow depths, which helps explain the shallow nature of observed slides in clayey embankments. Slope stability analyses should also consider rainfall, flood, and drawdown-induced pore-water pressure mechanisms, as well as the effect of slope movement on the mobilized shear strength to ensure a slope stability surprise does not occur.

Next Steps

The ability to compute two- and three-dimensional factors of safety for slopes has increased significantly in the last decade. However, these computational resources and methods still need guidance to ensure the factor of safety is computed for all possible stability conditions and failure mechanisms, not only the likely mechanisms. The analyses should utilize relevant loading conditions, soil properties, and boundary conditions, but also a range in these parameters to assess the sensitivity of the results. This is important because slope instability usually results in significant repair costs that are frequently amplified by litigation.

This guidance should extend to changes in field conditions that differ from the design condition. These conditions could include an increase in slope inclination, height, and/or length; placement from top of slope to the bottom instead of bottom to top because it is quicker and easier; increased unit weight due to a material change; and increased rate of loading. Frequently, decisions about such changes have to be made quickly to comply with the construction schedule.

As a result, proper quantitative evaluation is not performed, which can result in a slope stability surprise. The increased use of design/build contracts can contribute to designers having to quickly assess the impacts of field changes because of pressure from members of the design/build team. A protocol for assessing the impact of such field changes should be established via the contract documents so that sufficient time is allowed to evaluate the changes and to avoid slope stability surprises.

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