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Interface shear strength part 2: Design considerations

Previously, basic interface shear strength considerations for geomembranes were reviewed in Part 1. In Part 2, additional considerations for geosynthetic clay liners (GCL) and geotextiles are reviewed, and broader design considerations for the stability of landfill barrier systems are discussed. As in all design, the goal is to develop a conservative analytical model that incorporates realistic material properties. Laboratory measured interface shear strength properties are "realistic" if the test conditions conservatively replicate limiting field conditions. This means that laboratory interface shear strength tests must mimic field normal loads, hydraulic conditions, and maximum anticipated strain levels.

Geotextile interfaces

A clear understanding of the interface shear strength of various geotextile interfaces provides a good foundation for similar interfaces on GCLs and drainage geocomposites. Variations in geomembrane and geotextiles create a wide range of achievable interface shear strengths. Part 1 of this article described the different styles of textured geomembranes. Each type of texturing will result in different peak and post-peak strength responses. Also, while a material may perform acceptably at high normal loads, it may not perform as expected under low normal loads. For example, construction of a side-slope drainage layer may require a "hook-and-loop" effect on the interface between the geomembrane and the geotextile, which has proven to be difficult to obtain with some embossed textured geomembranes.

Stark et al. (1996) presented data indicating that geotextile polymer type, fiber type, fiber denier, and mass-per-unit area had an effect on peak and post-peak interface shear strengths. Surprisingly, these same variables had little effect on the large-strain residual interface shear strength. The authors also feel that the level of needling will impact geotextile interface shear strength.

The point of all the above is that product-specific interface shear testing must be performed during the design phase for the designer to verify that at least one combination of materials is available to meet the design requirements. Once this is achieved, acceptance of an "or equivalent" must be preceded by ASTM D-5321 direct shear conformance testing of samples. The design engineer must be wary of geotextile substitutions that change polymer, fiber density, or fiber type without additional laboratory testing to confirm that no significant reduction in interface shear strength resulted. It is assumed that these interface shear strength trends are also applicable to GCLs and drainage composites that have geotextile faces. GCLs face the additional concern that bentonite may migrate through a woven geotextile under strain and reduce the interface shear strength.

Soil/geotextile interfaces have been studied extensively for reinforced soil applications. For landfill applications, the soil/geotextile interfaces rarely control the critical failure surface; the geomembrane surfaces typically form the "weak link." With nonwoven geotextiles, the soil/geotextile interfaces shear strength is typically more than 90% of the soil shear strength. For woven geosynthetics, this ratio can be as low as 60%. The ratio of post-peak to peak strength decreases as the plasticity of the clay increases.

Stability analysis considerations

Slope stability evaluations are typically performed using computer programs that can quickly calculate the stability for a very large number of assumed failure surfaces passing through the landfill. The critical failure surface is the one having the minimum factor of safety. Slope stability analysis must be performed using interface shear strengths that reflect the field conditions being evaluated. In particular, the interface shear strengths should have been measured at normal loads, strains and moisture conditions that approximate those anticipated. The anticipated normal loads and strains acting on a failure interface will not be constant over the failure surface interface. Note also that this is an iterative process, since initial material properties must be assumed before a critical failure surface is determined.

Normal load—The normal load acting at a particular point of an assumed failure surface may be difficult to determine due to (1) uncertainties in the unit weight of the waste above that point or (2) due to the geometry of the applied load.

Current regulations in California require operators to achieve an initial density of at least 1,250 lbs. per cubic yard (7.3 kN/m³ or 46 lb./ft.³). Average values of municipal solid waste (MSW) unit weight can also be estimated based upon the total gate receipts over the life of a landfill and survey data. Average values for MSW unit weight cited by landfill operations and used in practice for landfill capacity estimates typically vary from 8.6 to 10.2 k/m³ (55 to 65 lbs./ft.³) (Kavazanjian et al. 1995). Landfill-specific values of MSW unit weight will depend upon actual operational practice. For instance, significantly higher MSW unit weights have been reported for a landfill that used an unusually high percentage of daily cover soil (Richardson and Reynolds 1991).

The variation of density with depth can have a significant influence on the results of static and dynamic stability and seismic response analyses. The dashed line on **Figure 1** shows this density-depth relationship developed for one southern California landfill (Puente Hills) on the basis of field measurements of density and laboratory measurements of waste compressibility (Earth Technology 1988). Based upon the Earth Technology density-depth profile, the initial and average unit weights cited above, and representative compressibility values for MSW reported by Fassett et al. (1994), Kavazanjian et al. (1995) developed a "Puente Hills" MSW unit weight profile shown by the solid line on **Figure 1**. This is commonly used in stability analyses of MSW landfills in the absence of landfill-specific data.

Given that the Kavazanjian unit weight predictions are based on landfills in Southern California, it is expected that the moisture contents for these landfills are significantly less than those in this study. Discussions with Teresa Dodge with Puente Hills indicate that the average moisture content of their waste is estimated to be approximately 22%. It is recommended that designer's adjust the density obtained from **Figure 1** to agree with the typical moisture content of the waste in question. This adjustment in unit weight is performed using the following relationship:

$$\gamma_T = \frac{(1 + w)\ \gamma_{Puente}}{1.22}$$

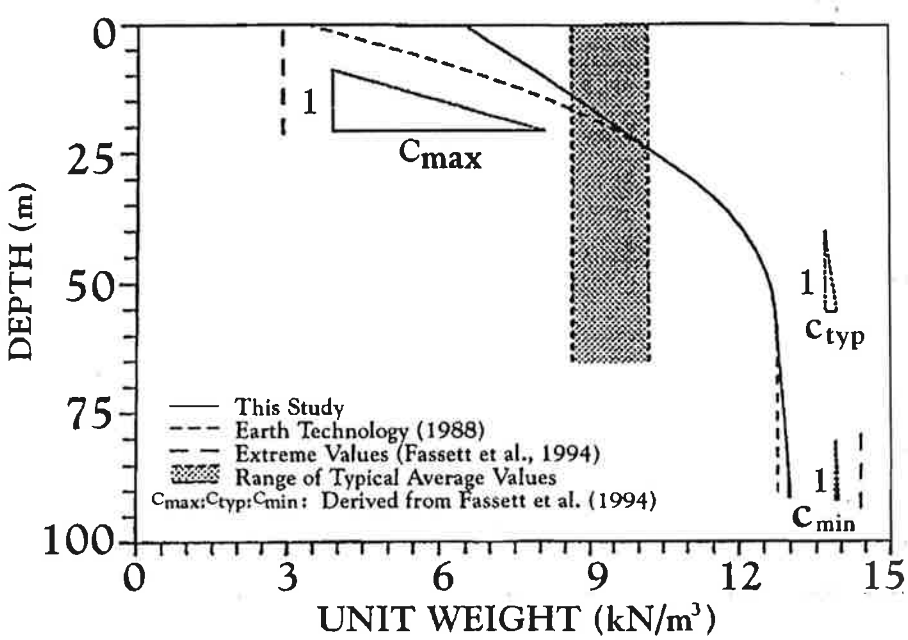
where w is the moisture content at the landfill in question, γ_T is the unit weight of the waste at a water content of w, and γ_{Puente} is the unit weight of waste obtained from **Figure 1**. Moisture contents in Eastern landfills are commonly in the range of 30-35% and in a leachate recirculation landfill, the moisture content can exceed 60%.

The normal load acting on a portion of a critical failure surface is commonly taken as the average unit weight of the waste above the surface times the depth of waste above that portion. The normal load should be used to select the appropriate shear strength from a laboratory-generated shear strength envelope (see **Figure** 2 of Part 1 of this series).

Strain level—The amount of strain or relative displacement experienced that exists in the materials prior to failure must be estimated. In landfills, the strain is typically the result of waste subsidence or seismic-induced slope movement. Subsidence strain occurs as the waste degrades and settles relative to surrounding berms or canyon walls. Since the amount of subsidence decreases as you move to the lines system, so does the strain. For this reason, Stark and Poeppel (1994) have recommended that stability analyses should use peak strengths for the lower portion of the landfill and large strain residual strengths for those upper portions settling relative to the liner system. Similar conclusions were reached by Gilbert and Byrne (1996).

Seismic-induced slope failures are commonly evaluated using an approximate displacement analysis technique first proposed by Newmark (1965). This technique simulates an earthquake using a static load applied horizontally and uses conventional slope stability programs to define the critical failure surface. If the resulting factor of safety is greater than one, all is well. If the factor of safety is less than one, then significant displacements due to the earthquake are indicated. In this latter case, the analysis is repeated to determine the horizontal acceleration that produces a minimum factor of safety of 1.0, termed yield acceleration, and a resulting failure surface. The displacement of this material above this failure surface is then evaluated using an earthquake acceleration time-history for the site by assuming the block slides every time the seismic acceleration exceeds the yield acceleration. Using a simple double integration technique, the designer can estimate the resulting displacement. In reality, most designers use a chart prepared by Makdisi and Seed (1978) to estimate the displacement for a given earthquake magnitude event. For large seismic relative displacements, the true large-strain residual shear strength of the critical interface must be used. Since direct shear boxes used in ASTM

Figure 1: **Weight/depth relationships for the Puente Hills, Calif., landfill.**



Standard D-5321 are generally limited to 3-4 in. of displacement, the post-peak strength from this test may require further reduction. For example, Stark et al. (1996) presented data from a textured geomembrane/nonwoven geotextile interface that showed a peak interface friction angle of 32°, a post-peak (50 mm displacement) friction angle of 21°, and a large-strain residual friction angle of 13°.

Project specification considerations

Having properly designed the landfill, the designer must ensure that the interface shear strength requirements are reflected in the project specifications. The following recommendations are made:

• Do up-front testing during the design to determine a successful combination of materials. There may be other successful combinations, but at least you will know one that works.

• Describe in the specifications the materials that worked.

• Specify your strength requirements along with the test setup conditions (normal load, hydration, consolidation, shear speed), and require that all products meet this before it is accepted. The designer will need to decide how many points need to be established to satisfy design and confidence requirements. Here, some designers specify minimum required shear strengths at specified normal loads, while others specify in terms of a minimum friction angle and apparent cohesion. The latter may cause some confusion if used to specify interfaces that have highly nonlinear response to normal loads.

• After the contract is awarded, the final purchase of materials is contingent upon the successful shear strength testing of materials actually proposed and being manufactured for the project. Typically, someone is available to take a sample of proposed materials at the manufacturer's plant early in the actual production run for shear testing.

• Once the materials are verified to meet the project interface shear strength requirements, the balance of the production run can be verified for shear strength based on simple index tests. For example, for geotextiles use mass and grab tensile strength; for geomembranes, use asperity height.

Since many designers are not part of the construction process, the need for interface strength testing of all "or equivalent" materials must be clearly spelled out in the Construction Quality Assurance (CQA) plan for the project.

CQA considerations

The greatest challenge facing the CQA program is to confirm that the quality of interface shear strength is not compromised by changes in materials or construction practices. For the most part, CQA staff must rely on ASTM D-5321 direct shear conformance testing of samples and a combination of index and visual inspection of the materials being installed. As discussed in Part 1, the industry still lacks a simple index test that can compare the roughness of material samples that pass the ASTM D-5321 testing and field samples in question. Thus, faced with non-uniformity in supplied materials, the CQA staff may have to do "single point" ASTM D-5321 testing to confirm suitability or simply reject the material.

In general, no changes in geomembrane or boundary material, geotextile or soil, should be allowed without performing additional ASTM D-5321 laboratory testing.

Summary

The subject of interface shear strength now justifies its own sessions at geosynthetic conferences, so we acknowledge our limitations in discussing this subject in only two articles. However, the authors hope that the basics related to selecting design interface shear strength have been covered. History has shown that as soon as we become confident in the design of geosynthetic materials, a new and better material will evolve that reduces this empirical confidence. GFR

References

Earth Technology. 1988. "In-place stability of landfill slopes, Puente Hills Landfill, Los Angeles, California." Report no. 88-614-1. The Earth Technology Corporation. Long Beach, Calif.

Fassett, J.B., Leonards, G.A., and Repetto, P.C. 1994. "Geotechnical properties of municipal solid wastes and their use in landfill design." *Proc. WasteTech 94 - Landfill Technology Conference*. National Solid Waste Management Association, Charleston, S.C.

Gilbert, R.B., and Byrne, R.J. 1996. "Strain-softening behavior of waste containment system interfaces." *Geosynthetics International*. Vol. 3, no. 2. 181-203.

Kavazanjian, E., Jr., Matasović, N., Bonaparte, R., and Schmertmann, G.R. 1995. "Evaluation of MSW properties for seismic analysis." *Proceedings of the Geoenvironment 2000 Specialty Conference*. ASCE. Vol. 2. 1126-1141.

Makdisis, F.I., and Seed H.B. 1978. "Simplified procedure for estimating dam and embankment earthquake-induced deformations." *Journal of Geotechnical Engineering*. ASCE. Vol. 104, no. 7. 849-867.

Newmark, N.M. 1965. "Effects of earthquakes on dams and embankments." *Geotechnique*. Vol. 15, no. 2. 139-160.

Richardson, G., and Reynolds, D. 1991. "Geosynthetic consideration in a landfill on compressible clays." *Proc. Geosynthetics Conference 1991*. Industrial Fabrics Association International, Roseville, Minn.

Stark, T.D., and Poeppel, A.R. 1994. "Landfill liner interface strengths from torsional-ring-shear tests." *Journal of Geotechnical Engineering*. ASCE. Vol. 120, no. 3. 597-615.

Stark, T.D., Williamson, T.A., and Eid, H.T. 1996. "HDPE Geomembrane/Geotextile Interface Shear Strength." *Journal of Geotechnical Engineering*. ASCE. Vol. 122, no. 3. 197-203.

nately, thickness variation between the two tests is as much as 50%. This translates into a large variation in strain rate. Note that faster strain rates result in higher yield stresses. Obviously, standardization is in order here.

Because of these inconsistencies with ASTM D 1621, the ASTM D 35 committee has been in the process of casting a compressive strength standard of their own. Unfortunately, their attempts have been ongoing since the mid-1980s. Conjecture about why we are still without a standard in D 35 is as follows:

- Geonets and drainage composites are geometrically diverse; thus, they do not lend themselves well to standardization with regard to specimen shape and size. Unfortunately, we have to bite the bullet and exclude some material from this type of testing.
- The test procedure is based upon loads being applied vertically. There has been a great deal of discussion about loads imposed at various angles to vertical. This detail raises complications about friction and eccentric loading that are far beyond the index test.
- The test specimen is placed in an unconfined manner between two steel plates and loaded normally. There has been discussion about total or partial confinement of the specimen. Confinement typically has the effect of increasing the compressive response. However, to what degree this would occur is debatable.

Based upon the above opinions as background, the following recommendations are proposed:

- The compression test for geonets and drainage geocomposites should be a quick index test. It is anticipated that the results of the test will be used to evaluate product uniformity.
- The loading platens used for the test should be rigid enough to resist bending, and they should be capable of applying a uniform stress. The upper loading platen should be attached to the normal stress assembly in such a manner that no stress is placed on the specimen until the commencement of the test. The size of the platens should be at least 56 mm^2 (9 in.^{2}), as shown in **Figure 2.**

The loading mechanism should consist of a constant rate of extension (CRE) testing machine capable of applying compressive loads at a constant rate of deformation of 10% of the thickness of the material, as determined by the textile procedure of ASTM D 5199. Loading should be applied normally to the geosynthetic's surface without confinement, as shown in **Figure 2.**

If the material is less than 13 mm (0.5 in.) thick, multiple piles should be used to achieve a minimum thickness of 25.4 mm (1.0 in.). Steel plates greater than 50 mm (2 in.) by 50 mm (2 in.) should be sandwiched between the specimen piles.

Test specimens should be cut square, 50 mm (2 in.) by 50 mm (2 in.). The specimen should be taken no less than 300 mm (12 in.) from the edge of the stock and it should be examined before testing to verify that it is representative of the stock from which it was taken.

I am of the opinion that with such a standard, the geosynthetics community would be far better off than continuing the debate about what are the attributes of a perfect compressive performance test. Let's work together at ASTM to make such a standard method a reality. GFR

References

ASTM D 1621, Standard Test Method for Compressive Properties of Rigid Cellular Plastics.

GRI Standard-GN1, Compressive Behavior of Geonets.

GRI Standard-GC4, Compression Behavior of Prefabricated Edge Drains and Sheet Drains.