SUPPLEMENTAL MATERIAL

Selection of long-term shear strength parameters for strain softening geosynthetic interfaces

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This document consists of six Appendices.

Appendix A provides definitions of geotechnical terms used in the paper.

Appendix B provides strain softening R_{ss} values of typical geosynthetic interfaces reported by Koerner and Narejo (2005).

Appendix C provides a review of 35 years of literature regarding interface shear strength.

Appendix D provides a review of bottom liner slope stability failures spanning a period of 25 years.

Appendix E provides a proof that flattening bottom liner sideslope may worsen slope stability**.**

Appendix F discusses risk**,** reliability, and consequences relative to containment facility slope stability.

Appendix A – Definitions

The following are definitions of terms used in the present paper that are useful to define to avoid confusion.

- *Interface [shear] strength*: the present paper is concerned with the shear strength of geosynthetic surfaces, called interfaces, that slide past other geosynthetic interfaces or soils. Unless explicitly stated otherwise, any reference to ‘strength’ in the present paper should be interpreted to mean ‘shear strength.’ In the context of the present paper, interface shear strength is also meant to include the internal shear strength of multi-component geosynthetics such as geosynthetic clay liners (GCLs) and geocomposite drainage layers. Shear strength (τ) is typically a function of the effective normal stress (σ'_n), and the units of the shear strength at a particular effective normal stress are kPa. A graph of the shear strength versus the effective normal stress is called a ‘shear strength envelope.’ A straight-line approximation of the shear strength envelope, commonly referred to as the Mohr-Coulomb failure criterion, over the normal stress range of interest, is commonly used to mathematically characterize the shear strength envelope by parameters in which the slope of the line is called the ‘friction angle’, ϕ, with units of degrees, and the y-intercept of the line is called ‘cohesion’, c, with units of kPa. (Note: Sometimes the term ‘adhesion’ is used in lieu of cohesion for the y-intercept for geosynthetic interfaces. The term ‘cohesion’ is used in the present paper.)
- *Normal stress, sometimes also referred to as normal load.* The normal stress, σ_n, acts perpendicular to the direction of shearing, with units of kPa. Unless otherwise explicitly stated, the normal stress is assumed to be the ‘effective’ normal stress, σ'_n, which means that any fluid pore pressures acting on the interface are subtracted from the total normal stress.
- *Peak [shear] strength*: The highest strength measured during the shearing process under a given normal stress is called the peak strength, τ_p, which typically requires a relative displacement on the order of 2-20 mm in order to be completely mobilized, depending on the interface. The slope of a straight-line approximation of the peak shear strength envelope over the normal stress range of interest is called the ‘peak friction angle’, ϕ_p, with units of degrees.
- *Post-peak [shear] strength*: Any strength value that occurs past the point of peak strength is referred to as a post-peak strength. By convention, the term ‘post-peak strength’ is often inferred to mean a strength that is in between peak and large-displacement strengths.
- *Large-displacement (LD) [shear] strength:* By convention, the post-peak value of shear strength that occurs at approximately 75 mm of relative shear displacement is referred to

as the LD shear strength, τ_{LD}, so-established because that is the shear distance specified by ASTM D5321 (Standard Test Method for Determining the Shear Strength of Soil-Geosynthetic and Geosynthetic-Geosynthetic Interfaces by Direct Shear), and also corresponds to the maximum travel distance of many commercial devices. The slope of a straight-line approximation of the LD shear strength envelope over the normal stress range of interest is called the 'LD friction angle', ϕ_{LD}, with units of degrees. There may also be an LD cohesion value that is different than the peak value. Note: it is common in the industry, unfortunately, that the term 'residual' is used synonymously with the condition for LD. Technically these are different values, so it is imprecise to use these terms interchangeably. Nonetheless, it remains a fact that such usage can be readily found both in the literature and in vernacular usage in the geosynthetics industry.

- *Residual [shear]strength*: The lowest post-peak value of shear strength that does not decrease with continued shear displacement is referred to as the residual shear strength, τ_r. The amount of relative displacement required to achieve τ_r typically exceeds the ability of conventional direct-shear testing devices to measure in a single test. Specialized ring-shear devices are sometimes used to measure residual shear strength of interfaces based on the often-unstated assumption that the residual strength is not direction-dependent. The slope of a straight-line approximation of the residual shear strength envelope over the normal stress range of interest is called the 'residual friction angle', ϕ_{res}, with units of degrees. The residual cohesion value is often assumed to be zero.
- *Bottom liner system.* Bottom liner systems, including base and side-slope lining systems, as discussed in this paper, are characterized by high normal stresses that exist beneath waste or mining ore fills. The range of normal stresses in a bottom liner system can range from as low as 15 kPa near the perimeter, to more than 2 MPa in deep heap leach pads.
- *Sideslope*. In a general sense a sideslope could be considered as any reach of a bottom liner system with a slope greater than the LD shear strength assigned to that reach. The sideslope is generally referred to in the present paper as the lined slope at the back of the waste or mining ore mass under consideration.
- *Veneer liner system*. A veneer liner system refers to a relatively thin layer(s) of soil (generally ranging from about 0.3 to 1.5 m thick) that is spread over one or more geosynthetic layers, resulting in a relatively uniform low normal stress regime on the geosynthetics. Examples of this include the gravel-drainage layer and/or the protective-cover-soil layer for a bottom liner, the gravel-drainage layer and/or the protective-cover-soil layer for a cover system, the 'overliner' layer in a heap leach pad, or the protective-cover-soil layer on a pond liner system.

Appendix B – Strain-softening R_{ss} values of typical geosynthetic interfaces reported by Koerner and Narejo (2005)

The magnitude of strain softening, R_{ss}, is defined by Gilbert and Byrne (1996) as the ratio of residual to peak shear strength:

$$R_{ss} = \frac{\tau_r}{\tau_p} \tag{1}$$

Given that much of the geosynthetics literature and testing results are based on LD rather than the true residual, this term can receive a modified subscript as $R_{ss\text{-}LD}$ when it is known that the basis is LD. A value of R_{ss} = 100% would mean that the geosynthetic interface would not lose any of its shear strength after exceeding the peak strength and would not be considered strain-softening. A value of R_{ss} = 60% would mean that the geosynthetic interface would lose 40% of its shear strength after exceeding the peak strength, a significant loss of strength that would define that interface as highly strain-softening.

Koerner and Narejo (2005) provide peak and LD shear strength data on 48 different geosynthetic interfaces that was collected from the Geosynthetic Accreditation Institute's database of proficiency test results from 3,260 large-scale direct shear tests performed by many laboratories in general accordance with ASTM D5321. All of the data points for each type of interface are plotted and interpreted according to a best-fit Mohr-Coulomb failure criterion. Most of the tests were performed over a normal stress range of up to 700 kPa, although certain interfaces were only tested up to as little as 24 kPa. Because of the large variability in materials, changing materials and manufacturing processes by manufacturers, and unknown details in test setup protocols for each of the tests that were performed (e.g. wetting and flooding conditions are not described), the data should not be used for a final design, but it does give an indication of the range and patterns of results that can be expected for geosynthetic interfaces. Relative to the subject of the present paper, the data supports the idea that most geosynthetic interfaces *are* strain-softening. Of the 48 interfaces tested, only three interfaces indicated an $R_{ss\text{-}LD}$ value > 95%: textured HDPE geomembrane/unsaturated cohesive soil; CSPE-R geomembrane/woven geotextile; and nonwoven geotextile/granular soil.

Designers of important projects, or projects where stability is critical and the determination of the potential for strain softening is crucial, are advised do their own testing using project-specific materials and test conditions, and not to base their final designs on values plucked from the literature. For example, Hillman and Stark (2001) present data showing that R_{ss} = 100% for smooth PVC/ nonwoven geotextile interface for a normal stress of up to 400 kPa, presumably under dry conditions, since the paper does not mention wetting or flooding. The present author has seen similar results for this interface that were obtained under flooded conditions. Meanwhile, the Koerner and Narejo (2005) database shows $R_{ss\text{-}LD}$ ≈ 79% for that interface at the same normal stress range, with the acknowledgement that the wetting conditions were unknown for the database. The present author investigated a veneer system failure involving a similar interface between the smooth side of a PVC geomembrane and the nonwoven geotextile side of a GCL, and found that at 5 kPa, low normal stress $R_{ss\text{-}LD} \approx 100\%$ when tested submerged *but not sprayed at the interface.* However, when the interface was sprayed before assembling the geomembrane against the nonwoven side of the GCL in the test setup, the result from separate tests conducted at two different laboratories was $R_{ss\text{-}LD} \approx 67\text{-}86\%$. Furthermore, much more important than the $R_{ss\text{-}LD}$ values in that case, was the fact that *spraying* the interface reduced the peak shear strength by 60% (36° versus 16° friction), a condition which mimicked field condensation, and a result that perfectly predicted the failure. Apparently, assembling the liner interface in a dry mode and applying the normal stress sealed it adequately from becoming hydrated/lubricated in the flooded test condition, such that the flooded test actually reflected the condition of a dry interface. The lesson learned was that generally speaking, all geosynthetic interfaces being tested should be sprayed with water during the test assembly in order to mimic the inevitable condensation that occurs in the field, in addition to performing the test in a flooded condition. This illustrates the importance of performing project-specific testing and not relying upon published test results.

The other notable geosynthetic interface that may not incur significant strain softening is a geonet against an HDPE geomembrane, whether textured or smooth. While the Koerner and Narejo (2005) results indicate $R_{ss\text{-}LD} \approx 76\text{-}81\%$ for such an interface, the present author has seen convincing laboratory results indicating that the value of $R_{ss\text{-}LD}$ could be close to 100%, depending on the specific materials and test conditions. Here again, project-specific testing is strongly recommended.

It is interesting that geosynthetic-geosynthetic interfaces known to have a low potential for strain softening are generally lower-strength interfaces. This is good to know and consider when designing an intentionally weak interface where preferential slippage will occur.

It is the high-strength interfaces where reinforced GCLs (with both internal and exterior interfaces) and where textured geomembranes are used that have the highest magnitude of strain-softening potential, and that can experience more than 50% post peak strength loss. These interfaces include textured HDPE geomembranes against geotextiles or products with geotextile surfaces, such as geocomposites that have outer layers comprised of non-woven geotextiles, and reinforced GCLs that have outer layers comprised of geotextiles. These also happen to be among the most common geosynthetic interfaces used in containment liner systems. For example, Stark and Richardson (2000) described textured geomembrane interfaces as experiencing a 50-60% post-peak strength loss (meaning R_{ss}=40-50%). The results of Koerner and Narejo (2005) indicate $R_{ss\text{-}LD} \approx 52\text{-}63\%$ for these interfaces.

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Appendix C – Review of 35 years of literature regarding interface shear strength

This Appendix attempts to provide a succinct review of the literature related to the subject matter for a period spanning over one third of a century. The literature review is broken down into thematic sub-sections, not necessarily in chronological order, so as to create a context for a suitable evaluation of the subject matter of this paper as follows:

1. Initial literature related to the Kettleman Hills landfill failure.
2. Literature related to numerical analyses that could model progressive failure.
3. Three categories of states-of-practice for performing stability analyses with regard to the selection of interface shear strengths.

C1: Foundational literature related to Kettleman Hills Landfill failure and the identification of geosynthetic interfaces as being strain-softening, leading to progressive failure.

On March 19, 1988, perhaps the most well-known landfill slope failure in history involving geosynthetic interfaces occurred when Phase IA of landfill B-19 at the Kettleman Hills hazardous waste facility in southern California failed, causing 490,000 m^3 of waste to slide translationally on various interfaces of the liner system approximately 11 m (Koerner and Wong, 2011).

Both the base and sideslope of the failure surface took place along relatively weak geosynthetic interfaces involving a smooth HDPE geomembrane. The typical cross-sectional geometry of the failure included a 27 m high 2(H):1(V) sideslope with an approximately 150 m long base that daylighted at the toe, with the outer waste slope at an inclination of 3(H):1(V) (Byrne, 1994), as illustrated in Figure C1. A cross-section of the approximately 3 m thick liner system is shown in Figure C2.

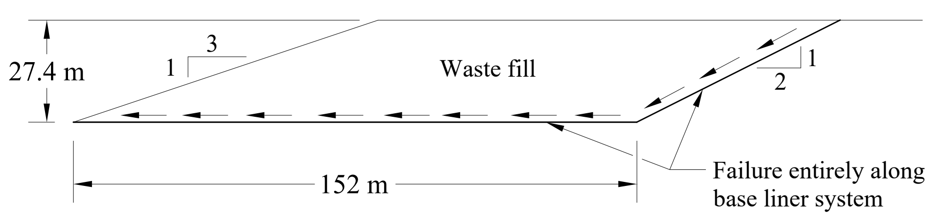

Figure C1. Representative schematic cross-section of the Kettleman Hills landfill slope prior to failure (adapted from Filz et al., 2001).

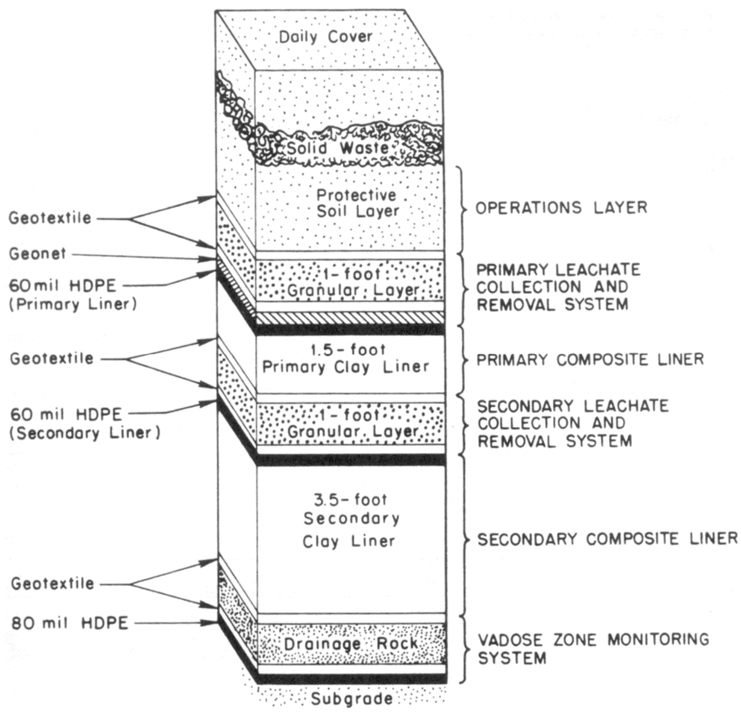

Figure C2. Detail of Kettleman Hills landfill liner system. (1 foot = 0.3 m)

This failure occurred almost exactly 35 years before the present paper was written. One of the repercussions of that failure was a sustained focus by the containment industry upon the techniques and approaches used to evaluate geosynthetic interface shear strengths and to perform slope stability evaluations for lined containment facilities. That effort and focus continue to this day. A familiarity with the historical evaluation of that failure is important for anyone wishing to understand the context of how the containment industry has arrived at its current stpo9ate of practice with regard to the establishment of geosynthetic interface shear strengths and slope stability analyses.

Seed et al. (1990) were the first to identify the geosynthetic interfaces in the Kettleman Hills failure as potentially ‘strain-softening’; they suggested that the failure mechanism “probably involved…some degree of progressive failure.” These and other early investigators (Byrne et al., 1992; and Stark and Poeppel, 1994) deduced that the amount of strain necessary to exceed peak strengths and promote progressive failure was produced in the Kettleman Hills failure, with the strain being attributed primarily to construction activities and waste settlement. This finding resulted in general recommendations by several designers in the early 1990s (e.g. Somasundaram and Khilnani, 1991; Druschel and Underwood, 1993; Lopes et al., 1993) which suggested that the residual or degraded interface strength is the most representative shear strength for use in stability analyses of lined containment facilities *because of the very small interface displacements* that would be required at geosynthetic interfaces.

The primary lessons learned and the outcomes from this failure, in the context of the present paper, were (1) the recognition of the strain-softening behavior of geosynthetic interfaces, and (2) the failure investigations spawned numerous numerical analyses that have given invaluable insight into the non-uniform mobilization of shear stresses and strain distributions that promote progressive failure. These lessons made it clear that the standard limit equilibrium models that were, and continue to be, used to evaluate slope stability are limited when peak strengths are used with strain-softening interfaces, and should therefore be used with caution when evaluated by experienced geotechnical engineers who should be familiar with these limitations.

C2: Development of numerical continuum analyses

The keys to understanding the potential development of progressive failure are knowledge of (a) the distribution and magnitudes of mobilized shear stresses at all locations along the critical interface, and (b) how much relative displacement (shear deformation) will occur, so that the strain-softening effects on strain-softening interfaces can be assessed. The power of numerical continuum analyses is that if the material properties are well known from laboratory and field measurements (e.g. material stiffnesses, Poisson's ratios, and shear-deformation characteristics), and the spatial and temporal variations of the interfaces can be modeled, and the physical conditions of geometry and transient forces (e.g. pore pressures, seismic) can be reasonably assumed, then numerical approaches have the possibility to show where stresses and strains will exceed the peak values, and to predict the distribution of relative displacements. Such assessments would be useful at different stages of development, such as during construction, at intermediate filling stages, after the containment facility is filled, after the fill settles, and perhaps at critical moments of peak pore pressures or dynamic excitation when those issues are relevant.

Byrne (1994) presented the first numerical analysis of the Kettleman Hills failure using a finite difference program called FLAC (Fast Lagrangian Analysis of Continua¸ marketed by the Itasca Consulting Group, Inc.). The shear deformation characteristics of the geosynthetic interfaces from that project were determined from laboratory testing and then used as input into the numerical analysis. While the analysis did not directly consider strain softening at the geosynthetic interface, it was able to demonstrate that a mechanism of progressive failure occurred because the distribution and magnitude of mobilized shear stresses caused peak strength to be progressively exceeded along portions of the geosynthetics during filling of the landfill cell. This analysis corroborated the explanation of the Kettleman Hills failure that had been previously surmised by the 1990-1994 papers mentioned in Section 2.1. Byrne's analysis and evaluation using FLAC was a groundbreaking and landmark paper which demonstrated that the non-uniform distribution of mobilized shear stresses, exceedances of peak strengths, and progressive failure could not be predicted by limit equilibrium models. Byrne (1994) also presented the first example of a numerical analysis of stability along a strain-softening textured geomembrane/geotextile interface for an assumed geometry containing a 2(H):1(V) backslope 30 m high with a 60 m long base and a 3(H):1(V) waste fill slope. The geosynthetic interface shear strength was assumed to have strengths of 25° and 12° friction for peak and residual, respectively. While the analysis results predicted that that configuration would be stable with pre-peak interface strength conditions preserved, the analysis also indicated that creep reduction of the strength over time, or relatively minor seismic loading, would cause progressive failure along the slope that would in turn overstress the base resistance, resulting in collapse of the structure. A similar analysis was presented in Byrne (1994), which modeled a reinforced GCL interface and showed how increased loading of the waste mass could eventually cause a portion of the GCL to fail and precipitate progressive failure, resulting in complete collapse. Byrne (1994) expressed concern regarding the long-term ageing and creep shear and seismic performance of geosynthetic interfaces. He recommended that if factors other than residual strength are assumed in the design process, then sensitivity analyses should be performed so that the implications of peak and residual shear strengths on stability can be understood, and the reliability of the design determined.

Reddy et al. (1996) performed finite element analyses of an MSW lined landfill geometry reminiscent of that in the Kettleman Hills failure. They compared the slope stability results for assumed smooth and textured geomembrane interfaces having peak interface strengths of 11° and 30°, respectively, for three different waste stiffnesses, for different back slope inclinations ranging from 3(H):1(V) to 2(H):1(V), and for MSW waste heights from 4 to 30.5 m. They did not model strain softening, but only investigated the distribution of stresses and displacements predicted by the numerical analyses based on peak strength parameters for the liner and waste materials. This too was a groundbreaking paper in that it presented not only the non-uniform distribution of mobilized shear stresses with greater clarity and ease of understanding than the presentation by Byrne (1994), but also addressed the distribution and magnitudes of liner interface displacement. The analyses presented by Reddy et al. (1996) indicated that while the distribution of mobilized shear stresses was not significantly affected by the interface strength, the amount of relative displacement was affected greatly. Their results indicated displacements to be approximately 7 times greater for a smooth geomembrane as compared to a textured geomembrane interface (45 vs 6.5 mm, respectively) along the base of a landfill having 30 m of waste. Interestingly, the maximum displacement was about 100% greater on the base (45 mm) than on the sideslope (22 mm) for the smooth geomembrane, but this pattern was reversed for the textured geomembrane, where the maximum displacement on the base (5 mm) was about 15% less than on the sideslope (6.5 mm).

This type of result is logical because of the way the waste over the base buttresses the waste above the sideslope, but it is not predictable when limit equilibrium analyses are employed.

Gilbert and Byrne (1996) and Gilbert et al. (1996) provided a complex analytical model that was corroborated by numerical analyses that emphasized the strain-softening nature of geosynthetic interfaces, and emphasized that the available shear strength will depend on the deformations (i.e. relative displacements) along those interfaces. Due to the compression and lateral spreading of the waste material on the slope and base of landfill, strains and slippage occur along the liner system interfaces. This mobilizes shear resistance, and with strain-softening interfaces this slippage can promote progressive failure. The analytical model considers the waste stiffness as compared to the geosynthetic interface stiffness, along with the strain-softening characteristics of the interfaces to determine the likelihood that progressive failure might occur. Gilbert and Byrne (1996) strongly recommended that a factor of safety greater than one be achieved in all containment slope designs, assuming that residual strengths are mobilized along the liner system.

Filz et al. (2001) later performed essentially the same task as that which was accomplished by Byrne (1994) by using numerical methods to explain the Kettleman Hills failure. Filz et al. (2001) also further advanced the concept of progressive failure by clearly demonstrating that the non-uniform mobilization of shear strength could not be predicted by limit equilibrium analyses. They compared the numerical results obtained using limit equilibrium results and concluded that, in the narrow case of the geometry and material interface properties similar to those in the Kettleman Hills case history, a shear strength set 10% higher than the measured residual shear strength would be appropriate for a limit equilibrium analysis.

Jones and Dixon (2005) presented one of the most insightful numerical studies that has been published to model the behavior of a strain-softening textured geomembrane/geotextile interface in response to waste settlement. The base case was a landfill with a 3(H):1(V) waste fill slope, a 3(H):1(V) lined sideslope with a height of 30 m, and a lined flat base with a length of 100 m. Numerical modelling of the shear strength of the textured geomembrane/geotextile interface were input based on laboratory test data having peak and residual friction angles of 24.5° and 12.8°, and cohesions of 3.2 and 2.5 kPa, respectively. The effects of varying the sideslope inclination from 3(H):1(V) to 1:1, and the waste height from 10-60 m were investigated for a waste stiffness that would achieve a long-term settlement of around 20%. The mobilized shear strengths and displacements along the interface were calculated in response to stresses induced by the elastic settlement of the waste body. Their results indicated that displacements along the base of the landfill occurred to varying extents, resulting in pre-peak to post-peak strength conditions, but in no case did the interfaces for any of the cases reach an LD strength condition along the base. For the sideslope, however, deformations exceeded the peak strength condition by differing amounts depending on the sideslope gradient. For the 30 m high 1:1 and 1.5(H):1(V) sideslopes, and for the 2(H):1(V) sideslopes that were 40 m and higher, very large displacements were predicted over the entire sideslope, some exceeding 3 m at the crest, which would result in residual shear strengths along the entire sideslope. The result would likely be tearing of the geomembrane (integrity failure) at the crest of the slope. For the 2(H):1(V) and 3(H):1(V) sideslopes that were 30 m high, the maximum displacements were less than 25 mm and were predicted for the lower half of the sideslopes, which would put them in a post-peak strength regime. The maximum displacements were about 32 mm for the 3(H):1(V) sideslopes that were 60 m high, resulting in a more degraded post-peak strength.

An interesting comparison can be made between the results of Jones and Dixon (2005) and those of Reddy et al. (1996), both of whom used nearly identical geometries (approximately 100 m long base, 30 m high waste, and various sideslope inclinations) for their numerical studies. One of the main differences in their assumptions regarded the peak shear strength of the textured geomembrane/geotextile interface. Jones and Dixon (2005) assumed 24.5° friction and 3.2 kPa cohesion, whereas Reddy et al. (1996) assumed a more ambitious 30° friction and 12 kPa cohesion. For the case of a 2(H):1(V) backslope, the Jones and Dixon (2005) results predicted that 79% of the base area and 93% of the sideslope would be in post-peak strength mode, whereas the Reddy et al. (1996) results predicted that the entire geosynthetic interface would remain in a pre-peak condition with no strength degradation. The Reddy et al. (1996) results point to the value of having a higher peak strength, as it is potentially strong enough to resist the initiation and propagation of strain softening that leads to progressive failure. That said, the assumed peak strength should be realistic and reliable.

Another interesting evaluation performed by Jones and Dixon (2005) compared two sets of limit equilibrium results. The first set of limit equilibrium results used a ‘rule based’ method of employing peak strength on the base and LD strength on the sideslope. The second set of limit equilibrium results were based on average shear strengths on the base and sideslope that were obtained from the numerical analyses. Compared to the results obtained using the shear strengths obtained from the numerical continuum analyses, the ‘rule based’ factors of safety were underestimated (i.e. conservative) by 13% for the geometries of the 3(H):1(V) sideslopes, had a negligible difference for the 2(H):1(V) sideslopes, and were over-estimated by 13% for the 1:1 sideslopes. A close review of the estimated shear strength distributions in the numerical results (Table 4 of their paper) indicates that there is a complex distribution of mobilized shear stresses along the base and sideslopes as a result of displacements, a distribution that is not comprehended by the simplistic ‘rule based’ approach. This is why there are varying amounts of discrepancy between the two approaches. As the ratio of the base length to sideslope length decreases, there is more strain-softening activity on both the base and sideslope. Also, as the steepness of the sideslope increases for a fixed base length, there is increased strain-softening activity on both the base and sideslope. The reason that the two approaches gave the same results for 2(H):1(V) sideslopes was simply a result of offsetting errors. Essentially, the numerical results showed that all of the cases exhibited some degree of post-peak shear strength reduction along the base, and most of the cases showed some degree of shear strength that was greater than residual on the sideslopes. Based on the simple geometry evaluated in that study, it was concluded that the 'rule based' approach of employing peak strength on the base and LD strength on the sideslope is a simplistic approximation that allows limit equilibrium analyses to provide an approximately correct answer that is usually, but not always, on the conservative side. This conclusion largely vindicates the recommendations of Stark and Poeppel (1994), which come to a similar conclusion, and are presented in more detail in Section 3.1.

Stark (2022) presented a webinar which presented the results of numerical analyses similar in concept to the Jones and Dixon (2005) study in modeling the behavior of a strain-softening textured geomembrane/geotextile interface in response to waste settlement on bottom liner systems of varying geometries. Stark (2022) presented a landfill example and varied the base length of the lined landfill from 61-152 m, the sideslope gradient from 3(H):1(V) to 6(H):1(V), and the horizontal sideslope length from 80-240 m. For the particular geometries and interface strengths modeled, the results indicated that peak strength could be used on the base unless the ratio of base-to-sideslope length was less than 0.5, in which case the post-peak strength (between peak and large-displacement) should be used, or if less than 0.4, then large-displacement strength should be used. The results for the sideslope liner indicated that for the longest (240 m) and steepest (3(H):1(V)) slope the residual strength should be used, for intermediate combinations the large displacement strengths should be used, and for combinations of shorter/flatter slopes the post-peak strengths (between peak and large displacement) could be used. Even though the gentlest case of an 80 m long 6(H):1(V) slope indicated that the peak strength would be preserved, Stark (2022) still recommended using a post-peak strength for the slope. Given the nature of a webinar, many details could not be presented, and it is expected that the results of the research presented there will soon be published (perhaps even before the present paper).

C3: Literature representing three categories of approaches to the issue of peak versus residual shear strength, and that of progressive failure in bottom liner systems

Three categories of states-of-practice for performing stability analyses are defined regarding the selection of interface shear strengths for bottom liner systems:

1. State-of-practice that recommends that post-peak shear strengths should always be used along parts of or the entirety of the critical slip surface.
2. State-of-practice that allows that peak strength is an acceptable design basis, but which also recognizes that post-peak shear strengths should be considered for certain conditions.

3. State-of-practice that suggests that peak strength is a generally viable design basis.

C3.1 State-of-practice that recommends that post-peak shear strengths should always be used along parts of or the entirety of the critical slip surface

Papers that promoted a design practice that advocated always using LD or residual shear strengths in the slope stability design of lined containment facilities began appearing in the 1990s after the Kettleman Hills failure. These papers included Somasundaram and Khilnani (1991), who promoted the employment of large displacement (50 mm) strength for all interfaces for a specific landfill project in California, on account of the small amount of displacement it would take to exceed peak strength; Byrne (1994), which was previously discussed; Stark and Poeppel (1994), who provided the design guidance described below; and Gilbert and Byrne (1996), who strongly recommended that a factor of safety greater than one be achieved in all containment slope designs, assuming that residual strengths are mobilized along the liner system.

The design guidance provided by Stark and Poeppel (1994) was perhaps the most pragmatic and easily understood that has been offered to the lined containment design profession to this day. It can be summarized as follows:

- For landfill bottom liners, assign residual shear strengths to the sideslopes and peak shear strengths to the base of the liner system, and satisfy a static factor of safety greater than 1.5.
- Assign residual strengths to the sideslopes and base of the liner system and satisfy a static factor of safety greater than unity (which is the same design guidance recommended by Gilbert and Byrne, 1996, and again quite succinctly by Gilbert, 2001).

Because the title of the Stark and Poeppel (1994) paper focused on a test method (torsional ring shear) rather than the subject of design guidance, the impact of the paper on the design profession was likely more muted than it could have been. This deficiency was corrected by Stark and Choi (2004), who presented the same design guidance to the industry along with further discussion, and a title more directly related to peak versus residual shear strength recommendations for landfill liners. In addition, they also addressed considerations related to low-normal stress cover systems that will be discussed in Section 6.

Additional publications followed in the 2000s that conformed to the recommendations from the papers from the 1990s that prescribed the use of large-displacement or residual shear strengths. Thiel (2001) acknowledged that a variety of approaches could be used for evaluating slope stability and favored the recommendations of Stark and Poeppel (1994) and Gilbert and Byrne (1996) described above. Because strain softening may occur in many areas in an unpredictable manner, Thiel (2001) recommended that designers should attempt to position the critical slip plane above the primary geomembrane to the extent feasible for a given project. Filz et al. (2001)

stated, with regard to geosynthetic interfaces, "*it is unsafe to use peak strengths in combination with customary values of safety factor when designing MSW landfills"* because of the softening behavior of the interfaces. Kavazanjian et al. (2006) generally promoted the first half of the Stark and Poeppel (1994) approach, stating: "The use of a post-peak shear strength for sideslope liner systems would appear to be a reasonable and prudent measure considering the large deformations side slope liner systems are usually subjected to due to waste placement and postplacement waste settlement. However, the rationale for using a post-peak strength for the base liner system is less clear…" Jones and Dixon (2003) prepared a comprehensive international literature review and guidance on the stability of landfill lining systems for the UK Environment Agency, which included discussion of the strain-softening characteristics of geosynthetic interfaces, and the proclivity of these types of interfaces to lead to progressive failure scenarios. They suggested that the practice of applying residual shear strengths on the sideslopes and peak strength on the base 'is a valid assessment of global stability' of a liner system when using limit equilibrium.

Two state regulatory agencies published design guidance for landfills that required the use of large displacement or residual shear strength parameters for landfill bottom liner system interfaces. Beginning in the mid- to late-1990s the California Department of Water Resources recommended that large-displacement shear strengths at 75 mm of displacement be used for all bottom liner systems, which is presumed to be on the interface having the lowest peak strength, and a final buildout static factor of safety of 1.5 (to one decimal place) (Driller, 2022). The Ohio EPA (2004) recommended the use of residual shear strengths on the interface having the lowest peak strength for all bottom liner systems having a slope greater than 5% and required a minimum static factor of safety of 1.50 (to two decimal places).

C3.2: State-of-practice that allows that peak strength is an acceptable design basis, but which recognizes that post-peak shear strengths might be considered for certain conditions

Daniel and Scranton (1996) is a USEPA publication that reported on an update of the EPA-sponsored Cincinnati GCL test plot study. A general Q&A session elicited the question: "Should designs for waste containment structures be based on peak or residual shear strengths?" The documented response from the three project managers of the study (Robert Koerner, Dave Daniel, and Rudy Bonaparte) indicated that the decision should be project-specific while 'checking the design for residual strengths', and that if the safety factor using residual strengths is greater than one then the design should be acceptable. The same three project managers reiterated this position in another EPA publication (Bonaparte et al. 2002), stating that "careful consideration must be given to the shear strength deformation conditions used in design (i.e., peak, large displacement, or residual)."

Thiel (2001) discusses a range of issues related to the selection of peak versus residual shear strength for geosynthetic interfaces on bottom liner systems, and also addresses progressive failure mechanisms. Thiel (2001) concludes that even though a case could be made for using peak strengths in certain circumstances with an appropriately high factor of safety, there should be a check to ensure that the factor of safety is greater than 1.0, assuming that hydrated residual shear strengths exist along the entire lining system. The latter condition usually controls the design.

Sabatini et al. (2002) stated that using peak strength for geosynthetic interfaces was a state of the practice being used in the USA at that time, but their paper appeared to limit the discussion of the use of peak strength to the flat base portions of bottom liner systems, because it did not explicitly consider shear strength variability on a sideslope. In addition, they cautioned that a lined sideslope "influence could be much more significant for other waste mass geometries" relative to the potential for progressive failure. Elsewhere, Sabatini et al. (2002) also supported the use of large-displacement interface shear strengths for all geometries as a secondary check in order "to address the potential for progressive failure due to waste-settlement-induced liner system shear stresses, construction-induced shear stresses, and/or interface creep".

Dixon and Jones (2003) provided guidance recommendations to the UK Environment Agency for the assessment of landfill liner stability and integrity. That guidance expressly avoided offering prescriptive recommendations and emphasized the need to involve experienced geotechnical specialists in the slope stability design process and in the justification of the factors of safety. Recognizing that limit equilibrium approaches were (and are) the traditional technique for evaluating slope stability, they suggested that if 'a cautious estimate' was made of the critical geometry and material properties, and if proper account is taken of potential 'actions' that could debilitate or destabilize the slope, such as pore pressures, construction damage, downdrag and settlement, fill sequencing, etc., then factors of safety in the range of 1.3 to 1.5 should be adequate. Dixon and Jones (2003) state that the "primary aim in many stability calculations is to ensure that post-peak shear strengths are not mobilized and hence to control deformations", they suggested that a possible alternative approach is to allow peak strength to be exceeded and use residual strengths in the analysis, thus ensuring both stability and integrity, and allowing a factor of safety as low as 1.2. In this case, protection layers may be needed in order to accommodate potential deformations that may arise, to avoid compromising the integrity of the liner system.

A series of articles was written for the Geotechnical Fabrics Report trade magazine under the leadership of Greg Richardson during the period 1998-2002 (Richardson et al., 1998; Richardson and Thiel, 2001a; Richardson and Thiel, 2001b; Richardson, 2002; Thiel et al., 2002) on the subject of slope stability and interface shear strength of geosynthetics. The general recommendation that emerged from this series of articles as related to the selection of the appropriate geosynthetic shear strength was that limit equilibrium analyses should be used with shear strengths adjusted for anticipated deformations, such as residual strength for sideslopes and peak strength along the base - with engineering judgement remaining a key factor in practice. Also referenced in those articles was the Gilbert and Byrne (1996) recommendation to check that the factor of safety was greater than unity for all residual conditions. It was also emphasized that since the publication of the EPA seismic design guidance for landfills (Richardson et al., 1995) the use of peak strengths has not been appropriate for facilities that might experience movement due to the design earthquake.

C3.3: State-of-practice that suggests that peak strength is a generally viable design basis

Compared to the hundred(s) of papers that have alluded to the idea of testing, measuring, and utilizing geosynthetic interface shear strengths that would be some degree less than the peak values obtained from laboratory testing, there has been a paucity of literature suggesting that peak strength is an outright appropriate standard to use to obtain slope stability in geosynthetic lined containment facilities. Only one such paper stands out in this category, namely that of Koerner (2003). This paper stands out all the further in that Dr. Robert Koerner suggested that large displacement or residual shear strength could be used or considered for containment liner analyses in years previous to (e.g., Daniel and Scranton, 1996) and after (e.g., Qian and Koerner, 2010) this 2003 paper.

Koerner (2003) proposed that, with the exception of sites that could be subjected to significant seismic shaking, containment liner systems could be designed to never exceed peak shear strength, thus allowing for the use of peak strength as a design basis. While Koerner's premise that containment facilities can be designed with materials and geometries so as to never exceed the peak strength is conceivably valid, the article is deceptive in its apparently simplistic logic. Chief among the unspoken complexities that underlie the article's premise is the fact that the ordinary and common approach for performing slope stability analyses, namely limit-equilibrium, provides no information regarding the magnitude and distribution of displacements within the slope, or how they vary along the slip interface (Duncan 1996). This complexity is substantially different from, and not comparable to, the case of materials such as "cast iron, fiberglass, graphite, etc." because of the highly complex distribution of stresses and strains in geotechnical slopes, which are not rigid bodies, as compared to structural elements such as columns and beams.

The tendency to optimistically promote the use of peak strength in geosynthetic liner system stability analyses is symptomatic of the geotechnically difficult subject of addressing the consequences of strain-softening materials. Even though Terzaghi had recognized as early as 1936 that over-consolidated, stiff-fissured clays (i.e., strain-softening soil materials) presented a special category of slope stability considerations (Duncan and Dunlop, 1968), LaRochelle (1989) suggests that it was only at the time his paper was written (ca. 1989) that the geotechnical profession had "finally accepted the evidence that the peak strength cannot be relied upon when dealing with problems of stability in strain-softening soils". This same sentiment was echoed seven years later by Duncan (1996), who stated: "*The only fully reliable approach in this case is to use the residual strength rather than the peak strength in the analysis.*"

Despite these divergent views, there was no doubt that Koerner (2003) was giving voice to a contingent of the design profession that was regularly engaged in the practice of containment system engineering and were employing peak strength as their design basis. From the author's participation in conferences and discussions in the USA throughout the 1990s it was apparent that there was a segment of the design profession that did not necessarily believe that landfill bottom liner stability analyses had to assume shear strengths that were degraded below the peak strength, but rather, believed that it was possible to use conservative selections of peak strength, combined with an adequate factor of safety, to provide a reliable design. In addition, several designers who were not practicing in seismically active areas believed that the rationale presented in the literature that suggested the use of large displacement or residual shear strengths, whether on slopes or flat bottom areas or both, was not based on mechanisms that would typically apply to their designs.

Designers may have felt that the example set by Kettleman Hills did not apply to their designs. The Kettleman Hills landfill design used a smooth HDPE geomembrane with multiple low-strength interfaces (geotextile, geonet, and compacted clay with a moisture content) and had 2(H):1(V) side slopes. While this configuration may have been true for Kettleman Hills, it is not representative of many municipal solid waste landfills that have 3(H):1(V) slopes and textured geomembrane interfaces, and that strongly favor needle-punched GCLs instead of compacted clays. In addition, it has to be admitted that there had been relatively few widely known occurrences of progressive failure along geosynthetic interfaces in the industry up to that point, especially for strong textured geomembrane interfaces, which might seem to corroborate the justification of some design practitioners for their use of peak strength in their designs. To this assertion we say here that time is not on their side. No studies have been done on the rate of progressive failure and, given the weakly-understood dynamics of the ageing and creep of geosynthetic interfaces, the initiation of failure could at first or for a seemingly long time be quite gradual, or simply awaiting an unusual triggering mechanism. We do well to remember that we are still in the infancy (not even half a century) of environmental containment project lifetimes that are often understood to be multiple centuries. Furthermore, there have been such failures, as is discussed in the next section.

Appendix D – Review of bottom liner slope stability failures spanning a period of 25 years

Failures define the limit states of any endeavor and provide the ultimate litmus test of engineering approaches. It was through a process of inductive inference based on observation, beginning in the 1930's with Karl Terzaghi, that the lessons of progressive failure were elucidated, a process that continues with the use of geosynthetics. As observations are accumulated, more elements of deductive reasoning are able to be used in the process of the science and engineering of geotechnical slope stability analyses. We are not out of the woods yet, however, in our understanding of the initiation and propagation of progressive failure along strain-softening geosynthetic interfaces. This section of the present paper presents a summary of historical slope stability failures that have occurred in bottom liner systems.

D1: Summaries of historical containment system slope stability failures

Here we consider four references that provide summaries of lined containment system bottom liner slope stability failures spanning the years 1988 to 2013: Breitenbach (1997); Stark (1999); Koerner and Wong (2011); and Bonaparte et al. (2020).

Breitenbach (1997) summarizes observations that were made of twelve heap leach mining slope failures over the period 1985-1993 that generally occurred during the first lift or two of ore placement. Nearly all of the failures were wedge-shaped, with the bases of the wedges sliding along the geomembrane interface contacts with the underlying clayey bedding fill layers, or in three cases with geotextile layers, where residual shear strength conditions were generally presumed to exist at the time of failure. Very few details were provided for any of the failures because they were all confidential, but it can be deduced, based on the understandings gained from the literature discussed in Section 2 of the present paper, that relatively small deformations along these weak interfaces could have resulted in a mechanism of progressive failure that was promoted by the heap pad filling. Breitenbach noted that excess pore pressures could have presumably played a part in several of the failures due to the low-permeability/high-plasticity clay foundation materials set below the liner, and unknown phreatic solution levels above the liner at the time of failure.

Stark (1999) presents a table listing 13 landfill slope failures involving geosynthetics, of which six were veneer-type failures with low normal stresses, either on covers systems or new bottom liner systems under construction, and seven involved higher normal stresses along base liners in operating landfills. Very few details of the failure histories are presented other than the waste fill slope inclinations and heights, the volumes of the slide masses, and a description of the slide interface. From the few details given it is clear that four of the failures are the same as the failures identified by Koerner and Wong (2011) as L-1, L-4, L-6, and L-7. One of the 13 interfaces was described as having a textured geomembrane (case G-7, which was a veneer failure that involved about 1500 m^3 of slide material), with the rest involving smooth geomembrane or other interfaces such as GCLs.

Koerner and Wong (2011) presented a report, followed by a webinar in 2014, on twenty large landfill bottom liner failures worldwide, eight of which involved translational sliding along geomembrane liner systems, all occurring between 1988 and 2003. The first one was the Kettleman Hills failure in 1988. Koerner and Wong (2011) presented back-analyses of each of the failures in an attempt to determine the triggering mechanisms. The presentation did not attempt to determine whether the back-calculated shear strengths represented peak or post-peak conditions but concluded that the triggering mechanisms for all of the eight failures along geomembranes involved excess liquid pore pressures, whether in the form of liquid buildup above the liner, or unconsolidated wet clay below the liner. A summary of the failures is as follows:

- L-1: USA, 1988 (Kettleman Hills). Translational failure of 490,000 m^3 of waste along interfaces of smooth geomembrane, geonet, and wet compacted clay.
- L-2: France, 1994. Translational failure of 60,000 m^3 of waste along the interface of a smooth geomembrane and wet compacted clay, similar in geometry and mechanism to the Kettleman Hills failure.
- L-3: Portugal, 1995. Failure not related to use of geosynthetics in liner: Portuguese landfill that was lined and experienced foundation failure below 110,000 m^3 of waste.
- L-4: USA, 1996. Translational failure of 100,000 m^3 of waste along an interface of an unreinforced geomembrane-backed GCL that became hydrated.
- L-5: South Africa, 1997. Translational failure of 300,000 m^3 of waste due to the addition of liquid to waste, with sliding through the saturated waste, and at the base toe along the interface of a geotextile atop a smooth polypropylene geomembrane.
- L-6: Colombia, 1997. Translational failure of 1,200,000 m^3 of waste that had been pressurized with injected leachate. The base of the landfill was lined with a PVC geomembrane, although it is not known if the failure surface was actually along the liner because the failed waste was fluidized.
- L-7: South Africa, 1997. Translational failure of 200,000 m^3 of waste with sliding along an interface of a smooth geomembrane and an underlying geotextile on a 2.5(H):1(V) sideslope.
- L-8: USA, 2000. Translational failure of 300,000 m^3 of waste along the interface of a smooth geomembrane on top of a compacted clay.

- L-9: UK, 2003. Translational failure of 15,000 m^3 of waste, with most of the failure surface along a 2.5(H):1(V) sideslope occurring at an interface between a textured geomembrane over a GCL. A relatively small base area of the failure occurred along an interface of a smooth geomembrane over a GCL.

Bonaparte et al. (2020) reported on 16 large landfill stability failures that occurred in the USA between 1984 and 2019, three of which involved translational sliding along geomembrane liner systems. Two of the failures are the same as those designated by Koerner and Wong (2011) as L-1 and L-4, which occurred in 1988 and 1996. The third failure, which occurred in 2013, occurred along a textured geomembrane liner system interface. Bonaparte et al. (2020), which discusses the lessons learned over this period as related to slope stability, includes, among other issues, the potentially weak and sensitive shear strengths of geosynthetic interfaces, the potential for progressive failure due to strain incompatibilities, and unanticipated fluid pore pressures above and below the liner system due to wet waste, landfill gas, the addition of liquid, or saturated, unconsolidated low-permeability soils that receive additional total stress. Excess pore pressures are cited as a significant contributory factor in most of the failure histories described in Bonaparte et al. (2020).

A fourth bottom liner landfill slope stability case history that was reported by Bonaparte at al. (2020) involved a degree of waste mass movement within acceptable limits (i.e. less than 150 mm) and some soil cracking and tears near the anchor trench at the tops of slopes which were caused by an earthquake in California. In that particular case the design is considered to have performed well given the severity of the earthquake. Also, some lessons were learned regarding where not to take destructive samples for testing seams (namely at the crests of slopes). That case history was a testament of the relative durability of well-designed facilities to sustain earthquake loading.

A predominant common theme mentioned in all four of these references is the significant involvement of fluids and pore pressures in the slope stability failure histories of lined facilities. The present author's observation is that this important observation is often downplayed in designs that include optimistic low-head assumptions as the design basis for slope stability analyses.

D2: Rumpke landfill failure: an example of progressive failure along soil bottom

The Rumpke landfill failure in 1996 in Ohio involved the translational failure of 1.1 million m^3 of municipal waste whose toe moved laterally approximately 250 m (Schmucker and Hendron, 1998; Stark et al., 2000). This failure, which was included in the Koerner and Wong (2011) and Bonaparte et al. (2020) inventories of failures, did not involve geosynthetics, but is being included in this discussion because the base of the failure surface was within a 2-5 m thick, strain-softening, native brown clay (colluvium) material whose peak and post-peak strength properties were closely representative of a textured geomembrane interface with a non-woven geotextile, which is a common bottom liner interface in many landfills, mining leach pads, and tailing piles. The base of the landfill in the native strain-softening clay presented a slightly adverse slope and was approximately 250 m long. The backslope of the failure day-lighted up through the waste at a near-vertical inclination, creating an approximately 60 m high scarp in the waste (Figure 5). The average interim slope of the landfill waste before the slide was 2.6(H):1(V). The toe of the landfill had previously been excavated to create an approximately 2.5 m vertical profile in order to allow for landfill expansion construction and was thus completely unbuttressed.

There were two separate forensic investigations into the cause of the Rumpke landfill failure, which are summarized in Schmucker and Hendron (1998) and Stark et al. (2000). While the conclusions of these separate studies varied somewhat regarding the suspected triggering mechanisms for the failure, both studies agreed that the underlying cause of the failure was exceedance of the peak strength of the native brown colluvial clay layer that underlaid the base of the unlined landfill. Schmucker and Hendron (1998) surmised that the most likely triggering mechanism for the landslide event was the buildup of pore pressure at the toe of the landslide area, which was likely a result of frozen ground at the toe, with additional contributions coming from continued filling and toe excavation. Stark et al. (2000) surmised that the triggering mechanisms were primarily related to strain incompatibility of the shear strength characteristics of the brown clay layer as compared to the MSW; lateral spreading of the MSW; overbuilding of the waste slope; rock blasting in an adjacent quarry; and excavation at the toe of the waste. All the potential triggering activities discussed by the forensic studies could have individually or collectively induced progressive failure of the native brown clay layer that led to the slope failure.

Bonaparte et al. (2020) summarizes shear strength test results performed by Geosyntec and Eid et al. (2000) using remolded samples in a ring-shear device that indicated that the brown clay exhibited drained, fully softened peak friction angles in the range of 23°–24° and residual friction angles in the range of 10°–13°. The laboratory test results showed that the native brown soil exhibited strain-softening stress–strain characteristics, with peak shearing resistances developed after only a few millimeters of displacement. Figure 6 demonstrates the similarity in the strain-softening potential between the Rumpke native brown clay and a typical textured geomembrane/geotextile interface. Figure 6(a) compares the shear strength envelopes of the native brown clay as determined by Eid et al. (2000) to that of a typical textured geomembrane/geotextile interface, with both having the same degree of strain softening from peak to residual strength (in the case of the clay), or peak to LD strength (in the case of the geosynthetics). Figure 6(b) shows shear-displacement curves at a normal stress of 50 kPa for these two different strain-softening interfaces. The main difference between the shear responses between these interfaces is that the native brown clay material exceeds peak strength after 2-3 mm of shear displacement, while the textured geomembrane/geotextile interface exceeds peak strength after 4-15 mm of displacement and has a more gradual decline in strength than is exhibited by the clay. These levels of displacement to trigger post-peak strength loss are significantly less than the amount of displacement that might be required to develop peak strength in the waste material, thus creating an unbalanced development of mobilized shear stresses between these two materials, which results in the exceedance of peak strength in the strain-softening geosynthetic interface, which can then lead to progressive failure. This mechanism of progressive failure in which two materials with vastly different stress-displacement characteristics comprise the critical failure surface, is referred to as 'strain incompatibility.'

In the context of the present paper, a primary lesson to be learned from this failure is clear. Namely that it is a plausible outcome for there to be progressive failure along the base of a containment facility that has strain-softening characteristics similar to those of a textured geomembrane set against a nonwoven geotextile, where the failure daylights up through the waste without the constraint of a lined sideslope.

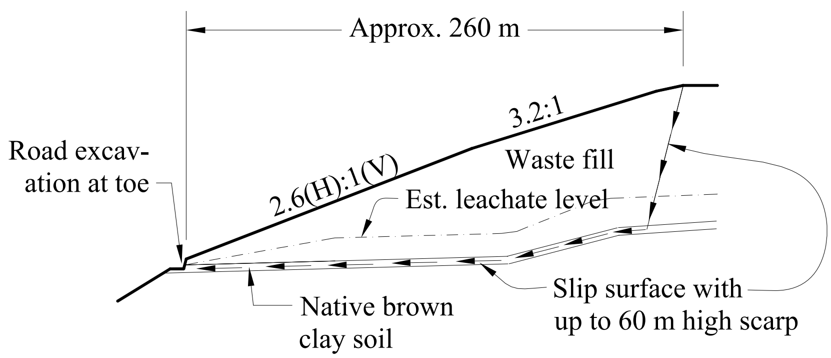

Figure D1. Representative cross-section of the Rumpke landfill slope prior to failure (adapted from Schmucker and Hendron, 1998).

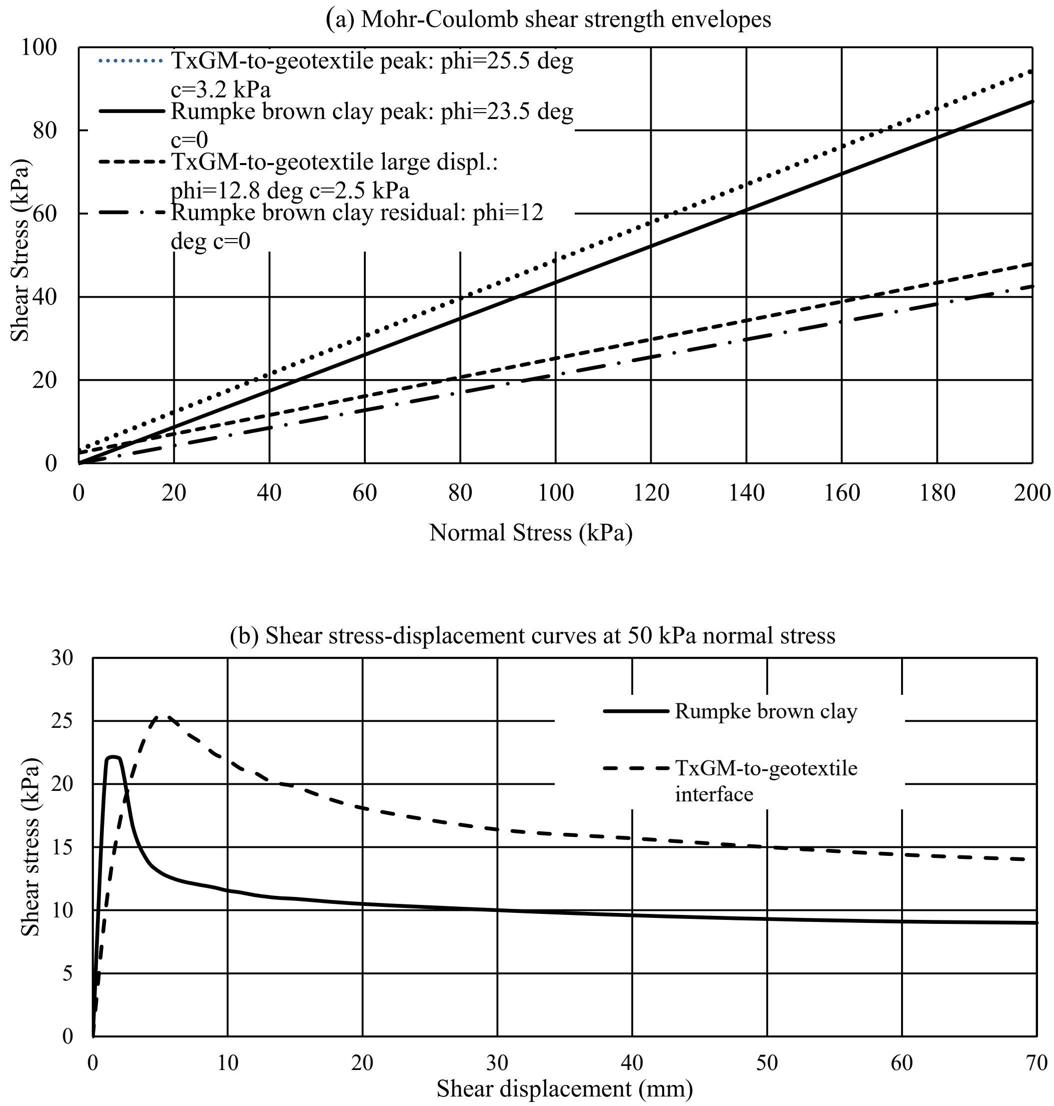

Figure D2. Comparison of shear strength characteristics of Rumpke brown clay (adapted from Eid et al., 2000) and typical textured geomembrane/geotextile interface (adapted from Jones and Dixon, 2005).

Appendix E – Proof that flattening bottom liner sideslope may worsen slope stability.

Although flattening of the lined sideslope at the back of the waste or mining ore mass might have a greater chance of preserving peak strength on the sideslope, this Measure does not necessarily improve slope stability, and may even worsen it for bottom liners. A relatively steep sideslope for bottom liners can be a more stable configuration than a flatter sideslope because a steeper sideslope typically allows more base area to develop forces that resist sliding.

Sabatini et al. (2002) noted that the steep sideslope in their example had no influence on their slope stability results, and therefore avoided a discussion of shear strength along sideslopes. Jones and Dixon (2005) compared limit equilibrium stability results for a typical landfill cross-section that utilized a strain-softening geosynthetic interface and varied the sideslope from 3(H):1(V) to 2(H):1(V) to 1:1. When a uniform shear strength was used along the base and sideslope, whether it was peak or LD shear strengths, the factors of safety with the 2(H):1(V) and 1:1 sideslopes were 10-12% lower than those obtained for the 3(H):1(V) sideslope. When peak strength was used for the base, and LD strength was used on the sideslope, the factor of safety was virtually equal for all three cases. These results from Jones and Dixon (2005) indicated that the sideslope inclination did not have a great effect on stability when the same length was used for the base. In a real-world situation, the limit of the toe of the waste and the top crest of the repository would typically be fixed as part of a defined footprint, whereby steepening the sideslope angle would cause the length of the base to increase.

The present author extended the sensitivity analysis performed by Jones and Dixon (2005), as illustrated in Figure E1. To validate the model, the present author first replicated the work of Jones and Dixon (2005) in which the length of the base was held fixed while the sideslope inclination was varied. Next, the present author again varied the sideslope inclination, but this time set a fixed distance between the landfill toe and the crest of the sideslope. The results, summarized in Table E1, indicate that the factor of safety dramatically increases as the sideslope is steepened. The present author's approach thus better reflects a real-world situation as compared to the sensitivity analysis performed by Jones and Dixon (2005). In fact, for the case that used peak strength on the base and LD strength with a 1:1 sideslope, the factor of safety was 41% higher than that of the base case using a 3(H):1(V) sideslope. Additionally, for the same project footprint, the geometry with the 1:1 sideslope would have a much greater airspace volume than the design with the 3(H):1(V) sideslope. These results are not necessarily intuitive, but they are logical, because more base area is opened up with steeper slopes to provide additional buttressing. This exercise thus proves that sideslope flattening for bottom liners may not improve slope stability and can potentially make it worse.

Although the evaluation performed by the present author shows that steepening of the sideslope could improve slope stability, it is important to note that a steeper sideslope may also increase the risk of an 'integrity' failure in bottom liners because of the greater potential displacements along the sideslope. In this case, the standard design approach is to provide a sacrificial slip layer above the critical containment liner system elements on the sideslope, and to assume residual shear strength along the sideslope.

Also of note in Table E1 are the findings that the second column (LD strengths) *FS* results are uniformly about 51% of the first column (peak strengths), and that the third column (mixed LD/peak strengths) results range from 79-93% of the first column (peak strengths). It is impossible to know from limit equilibrium analyses which column of results are the most realistic for any of these cases because the displacements required for shear strength mobilization are unknown. This is one of the key messages of the present paper.

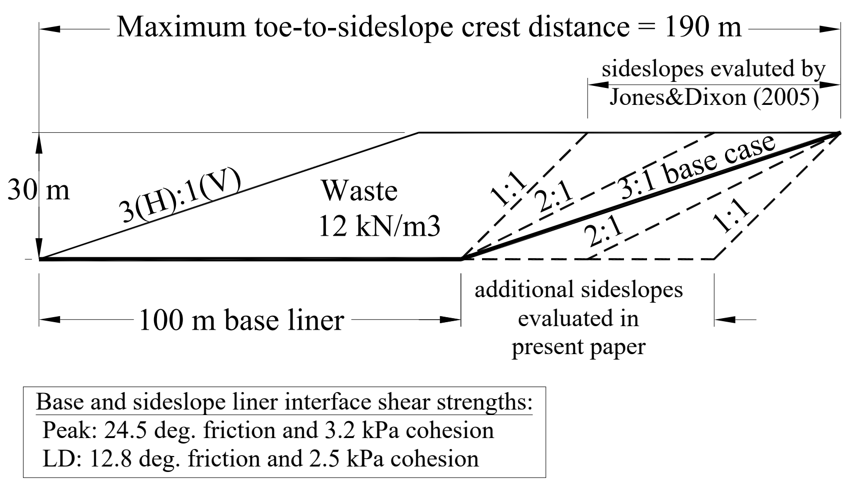

Figure E1. Geometry variations used to perform sensitivity analysis of the effect of sideslope flattening on factor of safety.

Table E1. Factors of safety (*FS*) calculated for various sideslope inclinations relative to Jones and Dixon (2005) base case geometry.

Case	Notes	Calculated *FS* (Spencer's method)		
		Base peak, Sideslope peak	Base LD, Sideslope LD	Base peak, Sideslope LD
Base case 3(H):1(V) sideslope	100 m base Toe-crest dist. =190 m	3.36	1.72	2.65
J&D (2005) 2(H):1(V) sideslope	Hold base dist.=100 m Toe-crest dist.=160 m	3.02	1.55	2.53
J&D (2005) 1(H):1(V) sideslope	Hold base dist.=100 m Toe-crest dist.=130 m	2.97	1.53	2.65
New evaluation 2(H):1(V) sideslope	Base increases to 130 m Hold toe-crest dist.=190 m	4.03	2.06	3.53
New evaluation 1(H):1(V) sideslope	Base increases to 160 m Hold toe-crest dist.=190 m	5.09	2.59	4.74

Appendix F – Risk, Reliability, and Consequences

The standard accepted geotechnical factor of safety (*FS*) for slope stability is 1.5 as applied to shear strength and mobilized shear stresses. Silva et al. (2008) show that this value was based on the historical performance of earth dams designed and constructed using conservative engineering practices, having an estimated probability of failure, P_f, on the order of 10^{-4}.

One of the distinguishing aspects of the geotechnical profession has been that there are differing states of practice related to how engineering calculations should be performed, as compared to, say, the structural engineering profession. As regards the slope stability of lined containment facilities, the most common regulations for landfills in the USA simply state that the minimum static factor of safety should be greater than 1.5. With few exceptions, the regulations typically offer no guidance as to how shear strength values should be determined for various materials or interfaces, or the way the analyses should be performed.

Duncan (2000) observed that applying the same value of *FS* to conditions that involve widely varying degrees of uncertainty is illogical and suggested an approach whereby reliability analyses could be used in concert with an approach that considers the factor of safety. The two approaches would then complement each other. Acknowledging that neither the reliability analysis nor the *FS* are usually calculated with precision, both still have value and enhance one another. The reliability analysis approach proposed by Duncan (2000) can be easily accomplished even by civil engineers with a very limited background in statistics, and the method helps to expose any aspect of the stability analysis to which the results are the most sensitive. The result is an approximate estimate of the reliability of the project slope stability, the inverse of which is P_f.

Silva et al. (2008) presented a general graph of P_f versus *FS* with correlations shown for four categories of projects representing different magnitudes of consequences. They suggested a design approach whereby a designer could use their graph, along with the level of design, construction, and operational measures appropriate for the category that is representative of the project consequences to achieve the desired P_f, using "good conservative engineering practices." Onto that graph Bonaparte et al. (2020) plotted the estimated landfill stability performance using data from the past several decades in the USA. Their results estimated that an average P_f for the containment industry is on the order of 10^{-3} assuming a design basis of *FS* greater than 1.5. Given the potential consequences of bottom liner landfill failures, Bonaparte et al. (2020) advocated that the landfill industry should strive for a lower P_f on the order of 10^{-4} along with an *FS* value greater than 1.5, which are the estimated average values used for earth dams.

Presumably either of the approaches promoted by Duncan (2000) or Silva et al. (2008) could be employed to help make design decisions regarding the stability of a lined containment facility.

The chief difficulty in performing the analyses would be the determination of the appropriate shear strength for the strain-softening interface dynamics that would take place in the field. For example, the current author performed a reliability calculation for a lined containment project having a significant 3(H):1(V) backslope lined with relatively strong, but strain-softening interfaces using limit equilibrium stability analyses. When peak strength was assumed, the results indicated FS = 1.8 with a very low P_f (less than 1×10^{-7}). When LD strength was assumed, the results indicated FS = 1.3 with a P_f value of 4.2%. The former results indicate a highly reliable project. The latter results would be unacceptable and indicate a high risk for a project for which failure would entail severe monetary and environmental consequences. On the other hand, if a project is temporary in nature, with relatively small to modest consequences of failure, and if the cost of reducing the P_f value is high, the latter results could be acceptable as long as the owner of the project was well informed of the risk. Without the insights provided by numerical continuum methods, it is difficult to say which shear strength assumptions would most correctly represent the shear strengths that might be mobilized in the field. In such a case, experienced judgement and/or detailed numerical analyses can be helpful. By using the risk analysis as a diagnostic tool one gains insight into the relative importance of the issue of strain-softening as applicable to a particular project.

Risk, R, is commonly defined as the product of the probability of failure, P_f, and the consequence of failure, C:

$$R = P_f \times C \tag{2}$$

Therefore, hand in hand with the calculation of FS and P_f would be a consideration of C for a given project. Many bottom liner containment projects have a consequence of failure that could lead to large cleanup and remediation costs, and even potential fatalities. Hence the advocacy of Bonaparte et al. (2020) for using a low P_f value on the order of 10^{-4} along with an FS value greater than 1.5. If a designer follows the rule-based analysis of Stark and Choi (2004) for bottom liner systems, the design will intrinsically be substantially safe. Part of the purpose of this paper was to set forth all the Mechanisms that a designer should consider that might lead to progressive failure, and thereby establishing a defendable justification for this type of rule-based design basis.

For veneer lining systems, where the consequences of failure are not as great, lower acceptable values of FS and higher values of P_f could be considered, depending upon the regulations and the degree of risk tolerance that would be acceptable to stakeholders. It is incumbent upon engineers to present the relative degrees of risk to their clients, and upon their clients to then make informed decisions.

REFERENCES

Bonaparte, R., Daniel, D.E., Koerner, R.M. 2002. *Assessment and Recommendations for Improving the Performance of Waste Containment Systems, EPA/600/R-02/099*. Cincinnati, OH: USEPA.

Bonaparte, R., Bacchus, R., and Gross, B. 2020. Geotechnical Stability of Waste Fills: Lessons Learned and Continuing Challenges. *J. Geotech. Geoenviron. Eng*., 146(11): 05020010, 24 pp.

Breitenbach, A. 1997. Overview study of several geomembrane liner failures under high fill load conditions. *Proceedings of Geosynthetics 1997*, March 11-13 Long Beach, CA: 1045-1061.

Byrne, R. J. 1994. Design issues with strain softening interfaces in landfill liners. *Proceedings of Waste Tech'94 Conference, Charleston, SC, January 1994.* National Solid Waste Management Association.

Byrne, R. J., Kendall, J. & Brown, S. 1992. Cause and mechanism of failure, Kettleman Hills Landfill B-19, Unit IA. *Proceedings of ASCE Specialty Conference on Performance and Stability of Slopes and Embankments—II*. New York: ASCE, 1188–1215.

Daniel, D.E. and Scranton, H.B. 1996. *Report of 1995 workshop on geosynthetic clay liners, EPA/600/R-96/149*. Cincinnati, OH: USEPA.

Dixon, N., and Jones, D.R.V. 2003. *Stability of landfill lining systems: Report no. 2, guidance*. Environment Agency, United Kingdom, R&D Technical Report P1-385/TR2.

Driller, M. 2022. Personal communications with Mike Driller, senior engineer with the California DWR regarding the history of guidance given for landfill liner stability analyses.

Drushel, S.J. and Underwood, E.R. 1993. Design of Lining and Cover System Side Slopes. *Proc. Of Geosynthetics '93*, March 30-April 1, 1993, Vancouver B.C.: 1341-1355

Duncan, J.M. 1996. State of the art: limit equilibrium and finite-element analyses of slopes. *ASCE J. of Geotechnical Engineering*, May 1996, 122(7): 577-596.

Duncan, J.M. 2000. Factors of safety and reliability in geotechnical engineering. *J. of Geotechnical and Geoenvironmental Engineering*, ASCE, 126(4): 307-316

Duncan, J.M. and Dunlop, P. 1968. *Slopes in stiff fissured clays and shales.* Contract Report S-68-4 for the US Army Corps Engineers, Waterways Experiment Station, Vicksburg, Mississippi.

Eid, H.T., Stark, T.D., Evans, W.D., and Sherry, P.E. 2000. Municipal solid waste slope failure. I: Waste and foundation soil properties. *Journal of Geotechnical and Geoenvironmental Engineering*, 126: 397–407

Filz, G.M., Esterhuizen, J.B., and Duncan, J.M. 2001. Progressive Failure of Lined Waste Impoundments. *J. of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 127, No. 10, Oct., pp. 841-848.

Gilbert, R.B. 2001. Peak Versus Residual Strength for Waste Containment Systems. *Proceedings of the 15th Annual GRI Conference Hot Topics in Geosynthetics – II, Houston, TX, Dec. 13, 2001*. Folsom, PA: Geosynthetics Institute, 29-39.

Gilbert, R. B. & Byrne, R. J. 1996. Strain-softening behavior of waste containment system interfaces. *Geosynthetics International*, 3(2): 181–203.

Gilbert R.B., Long J.H., and Moses B.E. 1996. Analytical model of progressive slope failure in waste containment systems. *International Journal of for Numerical and Analytical Method in Geomechanics*, 20(1): 35-56.

Hillman, R.P. and Stark, T.D. 2001. Shear strength characteristics of PVC geomembrane-geosynthetic interfaces. *Geosynthetics International*,8(2):135-162.

Jones, D.R.V. and Dixon, N. 2003. *Stability of landfill lining systems: Report no. 1, literature review*. Environment Agency, United Kingdom, R&D Technical Report P1-385/TR1.

Jones, D.R.V. and Dixon, N. 2005. Landfill lining system stability and integrity: the role of waste settlement. *Geotextiles and Geomembranes* 23(12): 27-53.

Kavazanjian, E., Dixon, N., Katsumi, T., Kortegast, A., Legg, P., and Zanzinger, H. 2006. Geosynthetic barriers for environmental protection at landfills. *8th International Conference on Geosynthetics*, Yokohama, Japan, September 18-22, 2006.

Koerner, R. M. 2003. A recommendation to use peak shear strengths for geosynthetic interface design. GFR Magazine 21(3) April 2003: 28-30.

Koerner, R.M. and Narejo, D. 2005. Direct shear database of geosynthetic-to-geosynthetic and geosynthetic-to-soil interfaces. *GRI Report #30, Geosynthetics Research Institute, Folsom, PA.*

Koerner, R.M. and Wong, W.K. 2011. GRI Report # GRI-41 – Analysis and critique of twenty large solid *waste landfill failures*. Geosynthetic Research Institute, Folsom, PA. Also presented as a webinar by R.M. Koerner on February 12, 2014, under the auspices of the Geosynthetics Institute under the title of "Behavior and Analysis of 20 Solid Waste Landfill Failures."

LaRochelle, P. 1989. Problems of stability: progress and hopes. *The Art and Science of Geotechnical Engineering*. Ed. by Corning et al., Prentice Hall, N.J.: 269-290.

Lopes, R.F., Smolkin, P.A., and Lefebvre, P.J. 1993. Geosynthetic interface friction: a challenge for generic design and specification. *Proceedings for Geosynthetics '93,* held in Vancouver, B.C. in April 1993: 1259-1272.

Ohio EPA. 2004. *Geotechnical and stability analyses for Ohio waste containment facilities*. Geotechnical Resource Group, State of Ohio EPA. Sept 14, 2004.

Qian, X. and Koerner, R.M. 2010. Modification to translational failure analysis of landfills incorporating seismicity. *ASCE Journal of Geotechnical and Geoenvironmental Engineering* 136(5): 718–727.

Reddy, K.R., Kosgi, S. and Motan, S. 1996. Interface Shear Behavior of Landfill Composite Liner Systems: A Finite Element Analysis. *Geosynthetics International*, 3(2): 247-275.

Richardson, G.N. 2002. Slope stability considerations for lined landfills. *Geotechnical Fabrics Report* May: 12-16.

Richardson, G., Kavazanjian, E., and Matasovic N. 1995. *RCRA Subtitle D Seismic Design Guidance for Municipal Solid Waste Landfills*. USEPA Office of Research and Development, Publication no. EPA/600/R-95/051, April 1995.

Richardson, G.N. and Thiel, R.S. 2001(a). Interface shear strength: Part 1 –geomembrane considerations. *Geotechnical Fabrics Report* Jun/Jul, 19(4): 14-19.

Richardson, G.N. and Thiel, R.S. 2001(b). Interface shear strength: Part 2 – design considerations. *Geotechnical Fabrics Report* Aug, 19(5): 16-19.

Richardson, G.N., Thiel, R.S., and Mackey, R. 1998. Designing with needlepunched reinforced GCLs: stability fundamentals. *Geotechnical Fabrics Report* Oct/Nov 1998: 22-27.

Sabatini, P.J., Griffin, L.M., Bonaparte, R., Espinoza, R.D., and Giroud, J.P. 2002. Reliability of state of practice for selection of shear strength parameters for waste containment system stability analyses. *Geotextiles and Geomembranes*, 20(4): 241-262.

Seed, R.B., Mitchell, J.K., and Seed, H.B. 1990. Kettleman Hills Waste Landfill Slope Failure. II: Stability Analysis. *J. of Geotechnical Engineering*, ASCE, 116(4): 669-690.

Schmucker, B.O. and Hendron, D.M. 1998. Forensic Analysis of the 9 March 1996 Landslide at the Rumpke Sanitary Landfill, Hamilton County, Ohio. *Proc. Of the 12th GRI Conference, Lessons Learned from Geosynthetic Case Histories*. Folsom, PA: Geosynthetic Institute, 269-295.

Silva, F., T. W. Lambe, and W. A. Marr. 2008. Probability and risk of slope failure. *J. Geotech. Geoenvironmental Eng.* 134 (12):1691–1699.

Somasundaram, S., Khilnani, K. 1991. Stability of High Refuse Slopes on Synthetic Lining Systems at the Bee Canyon Landfill. *Proc. Of Geo '91 Conference, Atlanta, GA, 1991*: 145-158.

Stark, T.D. 1999. Stability of waste containment facilities. *Municipal and industrial solid waste disposal technology, Waste-Tech '99*, Feb 1-3, New Orleans, LA.

Stark, T.D. 2022. *FGI Webinar: Mobilized interface strengths on geosynthetic lined slopes.* Fabricated Geomembrane Institute, October 18, 2022.

Stark, T.D. and Poeppel. 1994. Landfill liner interface strengths from torsional-ring-shear tests. *ASCE Journal of Geotechnical Engineering* 120(3): 597–615.

Stark, T.D. and Richardson, G. 2000. Flexible geomembrane interface strengths. *Geotechnical Fabrics Report, April:* 22-26.

Stark, T.D. and Choi, H. 2004. Peak versus residual interface strengths for landfill liner and cover design. *Geosyntethics International* 11(6): 491–498.

Stark, T.D., Eid, H.T., Evans, W.D., and Sherry, P.E. 2000. Municipal solid waste slope failure. II: Stability analyses. *ASCE Journal of Geotechnical and Geoenvironmental Engineering* 126: 408–419.

Thiel, R.S. 2001. Peak vs. residual shear strength for landfill bottom liner stability analyses. *Proceedings of the 15th Annual GRI Conference Hot Topics in Geosynthetics – II, Houston, TX, Dec. 13, 2001*. Geosynthetics Institute, Folsom, PA: 40-70.

Thiel, R., Erickson, R., and Richardson, G.N. 2002. GCL design guidance series, part 2: GCL design for slope stability. *GFR Magazine,* 20(6)