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

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ABSTRACT: The behavior of strain-softening geosynthetic interfaces that can lead to progressive failures in lined containment facilities has been a source of confusion in slope stability evaluations for over 30 years. The paper presents fifteen mechanisms that can potentially induce displacements along strain-softening interfaces, along with measures that can be considered to reduce strain-softening displacement. New quantifications of shear strength variability that can be caused by manufacturing, installation, and construction practices are introduced. Guidance and recommendations are given that are applicable to numerical continuum as well as limit-equilibrium approaches to assist in selecting appropriate geosynthetic shear strength parameters for containment facilities that have strain-softening interfaces. While most of the paper focuses on deep-seated critical interfaces for high normal stress bottom liners, low normal stress veneer covers are also addressed.

1 INTRODUCTION

1.1 *General*

Slope instability of large waste or mining containment facilities presents one of the most potentially immediate and consequential impacts to the greatest spectrum of stakeholders. Often a sizable portion of the critical slip surfaces for these facilities follows a strain-softening geosynthetic interface. The present paper identifies numerous factors that can be considered when selecting appropriate long-term shear strength parameters along these interfaces.

This specific topic has been an active, and occasionally intense, subject of technical papers in the geosynthetic industry for at least 35 years. Indeed, there have been several references with nearly the same title as the present paper over the past 20 years (e.g. Gilbert 2001; Sabatini et al. 2001; Thiel 2001; Stark and Choi 2004; Eid 2011; Stark 2022). Even so, confusion still exists among design practitioners regarding the appropriate testing, interpretation, and selection of long-term shear strength parameters for geosynthetic interfaces. Undoubtedly there will be future papers with a similar title, especially as more knowledge and experience becomes available related to the effects of ageing, durability, and latent weak zones within strain-softening geosynthetic interfaces.

Due to length restrictions, the present paper is shortened from a longer companion paper version that will be published in *Geosynthetics International* under the same title. Where appropriate, the reader is directed to consult the longer version for more information. For example, definitions of several geotechnical terms as used in this paper that are in common usage by practitioners in the containment industry are provided in the *Geosynthetics International* companion paper appendices.

1.2 *Concept of Strain Softening and Progressive Failure*

Probably the greatest amount of confusion and controversy regarding the selection of appropriate shear strengths for geosynthetic interfaces occurs because many of these interfaces are characterized as 'strain-softening'. A quasi-synonymous term that is sometimes used in the literature is 'brittle.' However the connotation of brittle might imply a narrower amount of strain deformation to achieve peak and residual strength conditions than might occur, and so the term 'strain softening' is adopted in the present paper. Whatever term is preferred, the shear strength of many geosynthetic interfaces reaches their peak value with a relatively small amount of relative shear deformation, often less than 2-20 mm, and then degrades relatively rapidly with continued deformation to lower values, as illustrated in Figure 1. These lower values are variously referred to as 'post-peak' shear strength, 'large-displacement' shear strength (commonly cited as occurring at approximately 75 mm of relative displacement), ultimately reaching a constant minimum value of what is called 'residual' shear strength.

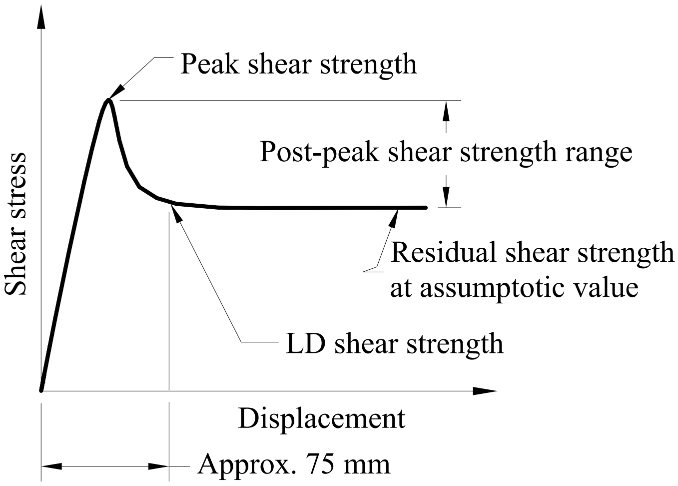

Figure 1. Schematic of shear-stress versus displacement at geosynthetic interface illustrating strain-softening behavior.

In classical geotechnical engineering, strain-softening soils have been, and continue to be some of the most difficult to evaluate for slope stability analyses. Thus, it is not surprising that confusion exists in the geosynthetics profession, considering that this complex type of shear strength interface is the norm in geosynthetics engineering.

In simplest terms, the problem with strain-softening interfaces is that even though a correctly performed limit equilibrium stability analysis based on peak strength indicates a generally accepted factor of safety (*FS*) greater than 1.5, the slope may still fail. The reason for this is a phenomenon called 'progressive failure.'

Strain-softening soils, or geosynthetic interfaces, promote the phenomenon of progressive failure, and make it impossible to count on mobilizing peak strength simultaneously at all locations along the failure surface (Duncan and Wright, 2005). Perhaps the best, and first description of progressive failure was made by Skempton (1964), and it bears repeating here, as it is directly applicable to our subject. Note that the current author has substituted the words [geosynthetic interface] where the original quote used the word 'clay':

"Irrespective of the physical explanation of the drop in strength after passing the peak, the existence of this decrease in strength must be accepted as a fact which has been well established. Thus, if for any reason a [geosynthetic interface] is forced to pass the peak at some particular point, the strength at that point will decrease. This action will throw additional stress onto the [geosynthetic interface] at some other point, causing the peak to be passed at that point also. In this way progressive failure can be initiated and, in the limit, the strength along the entire length of a slip surface will fall to the residual value. Obviously, in any given case, a slip may occur before the residual strength is attained throughout the [geosynthetic interface], but once a progressive failure has started the average strength of the [geosynthetic interface] will decrease inexorably towards the limiting residual value."

The next paragraph in Skempton's 1964 paper explains that zones of weakness and zones that have already failed past peak strength can act as stress concentrators and can then cause shear deformations to take place at an average stress that is far less than the ideal strength of the material. The current author is not aware of attempts, and has not himself attempted, to quantify the magnitude of stress concentrations within the plane of a strain-softening shear interface containing abrupt boundaries with weak zones. This concept is explored in later sections of this paper as a potential contributing factor to progressive failure, and a subject that merits further research.

1.3 *Strain-softening potential of typical geosynthetic interfaces*

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

$$R_{ss} = \frac{\tau_r}{\tau_p} \quad (1)$$

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. A synopsis of those results is provided in the *Geosynthetics International* companion paper appendices.

The author commonly specifies aggressive texturing which results in $R_{ss\text{-}LD} \approx$ 40-60% for these interfaces (e.g. see Figure 2). While aggressive texturing will provide the highest available peak shear strength, is that peak strength reliable given the high magnitude of strain-softening potential? This is a significant aspect of the subject of this paper considering that an average overall $R_{ss} \leq$ 67% could potentially cause an *FS* value of 1.5 that is based on peak strength to fall below 1.0.

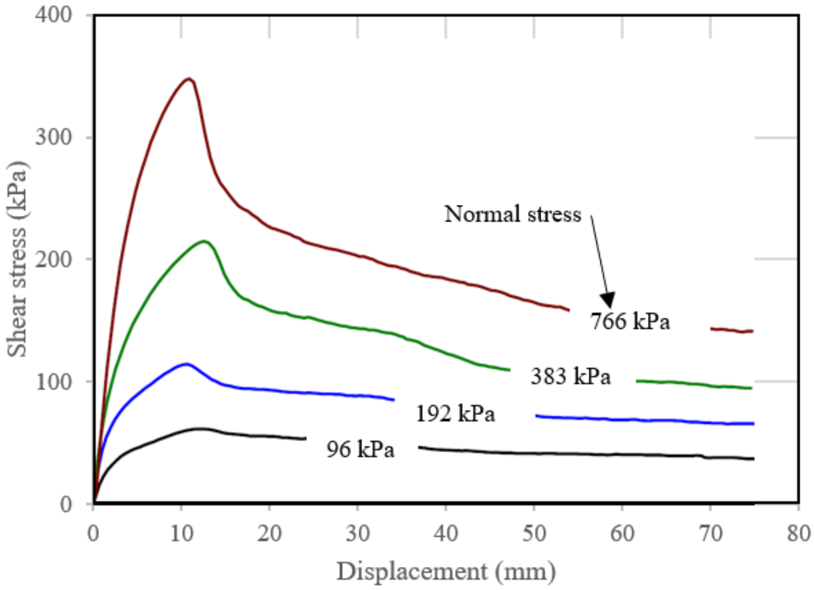

Figure 2. Shear-displacement graph for interface of aggressively textured HDPE geomembrane against geocomposite drainage layer having non-woven geotextile surfaces. Note that the magnitude of strain softening increases with normal stress, with $R_{ss\text{-}LD}$ ranging from 61% at 96 kPa to 41% at 766 kPa.

1.4 *Limit Equilibrium and Numerical Continuum Analysis Approaches*

For deep-seated failures that have strain-softening interfaces, limit equilibrium analyses that assume peak strengths can give non-realistic and non-conservative results, and also give no indication of the distribution or magnitude of mobilized shear stresses or displacements. However, the vast majority of the slope stability analyses performed in the geo-environmental containment industry are done by professionals using computer software programs based on limit-equilibrium models, such 'Spencer's method', or the 'Janbu method', or any of several other limit-equilibrium models that have been accepted and used for slope stability for many decades. Many users of these common and popular slope stability computer programs either do not realize that all of these models employ an assumption that soil blocks function as rigid bodies, or they do not understand the implications of this assumption. Because this assumption is a distinct limitation of these models, using them can lead to the following misleading or incomplete results when the critical interface is strain-softening and when peak strengths are assumed:

- The outputs of these models present a non-realistic, non-conservative uniform variation of shear stress mobilization that is proportional to normal stress, and an equally non-realistic uniform factor of safety along the entire sliding surface.
- The real-world non-uniform mobilization of stresses and strains, which are a significant driver for the inducement of displacements that lead to progressive failure in strain-softening materials, cannot be predicted by these popular programs.
- If peak strength is preserved everywhere, with no exceedance of peak strength at any point along the sliding surface, then the average factor of safety represented by limit equilibrium methods may be correct. Likewise, if residual strength exists everywhere along the sliding surface and is used in the model, then the average factor of safety represented by limit equilibrium methods may be correct.
- The results of these popular programs are non-conservative when using peak strength for strain-softening shear interfaces that would experience a drop-off in shear strength with a small amount of displacement, unless special precautions are taken to model the appropriate locations within the cross-sections being analyzed, using degraded shear strengths that are less than the peak shear strengths (e.g. residual shear strength).

Numerical analyses that employ continuum mechanics models (e.g. finite element or finite difference computer programs) are able to approximately predict the complex patterns of displacements and non-uniform mobilization of stresses along the critical interfaces. However, the greater amount of time, effort, and expense required to perform numerical analyses makes them unattractive for everyday use. Although the use of continuum analyses is generally increasing, the vast majority of slope stability analyses in engineering practice continue to be performed using limit-equilibrium programs.

The limitations of limit equilibrium methods may not be as severe in the evaluation of relatively thin soil (veneer) layers on sloped lining systems, but even in these cases the potential for degradation of geosynthetic interface shear strength is significant and should be considered.

1.5 *Goals of the Present Paper*

The present paper has three goals: (1) provide a contextual history of our understanding of progressive failure along strain-softening geosynthetic interfaces for lined containment, (2) describe mechanisms that could lead to progressive exceedance of peak strength along strain-softening interfaces, and (3) describe measures that can be used to allow for or mitigate strain softening mechanisms.

Due to space limitations for the present paper, the significant discussion of the history and literature related to the issue of strain softening and progressive failure along geosynthetic interfaces in lined containment facilities is provided in the *Geosynthetics International* companion paper appendices, which includes a rather lengthy list of references, and which serves as a resource for anyone interested in conducting their own research or evaluation of this topic. In conjunction with the extensive reference material, the *Geosynthetics International* companion paper appendices also present a historic review of bottom liner slope stability failures that involved geosynthetic or geosynthetic-like interfaces.

1.6 *Organization of the Present Paper*

The present paper is organized as follows:

1. Introduction and background.
2. Review of mechanisms that can potentially induce displacements along strain-softening geosynthetic interfaces.
3. Review of measures that can be implemented to reduce or mitigate displacements that can lead to progressive failure along critical strain-softening interfaces.
4. Discussion of risk and other considerations when considering peak strength.
5. Conclusions and recommendations.

2 MECHANISMS AFFECTING DISPLACEMENT ALONG STRAIN SOFTENING INTERFACES

This section of the paper presents a list of 15 Mechanisms that can potentially induce displacements along strain-softening interfaces, and thus contribute to a progressive failure mechanism. This is the most extensive such list that has been published to date. All 15 of the potential Mechanisms discussed in this section are relevant to bottom liner (high normal stress) situations, while 9 of these mechanisms are also deemed to be potentially relevant to veneer (low normal stress) situations. Some of the Mechanisms suggested by the author may be considered speculative, unproven, or theoretical. Several of the Mechanisms that affect displacements as discussed in this paper are interrelated and may seem redundant from a purely technical point of view. Even so, it is worthwhile discussing them separately to develop an awareness of the range of influences and their nuances that might affect the exceedance of peak strength, and hence the propagation of displacements, at the interfaces.

The Mechanisms that could lead to progressive displacements are grouped into five categories based on the nature of their causes.

- Category 1 includes five Mechanisms that are related to static stress and strain mobilization due to planned geometry, gravity, and constitutive material properties such as density, stiffnesses, compressibility, and Poisson's ratio. For a given geometry and material properties, the strain-softening effects from this set of Mechanisms are unavoidable and can best be estimated using numerical analyses.
    - Mechanism #1: Non-uniform mobilization of shear stresses.
    - Mechanism #2: Waste settlement, stiffness, compressibility, creep, and degradation.
    - Mechanism #3: Static lateral spreading of the waste.
    - Mechanism #4: Foundation settlement.
    - Mechanism #5: Strain/deformation incompatibility between overlying waste/ore material and the liner system interface.
- Category 2 includes four Mechanisms that are related to the aggravation of stresses and strains caused by construction and operational activities.
    - Mechanism #6: Waste placement activities.
    - Mechanism #7: Slope overfilling/oversteepening.
    - Mechanism #8: Toe excavation.
    - Mechanism #9: Construction-induced shear strength degradation.
- Category 3 includes three Mechanisms that are related to different types of short- or long-term transient influences that are not due to construction and operational forces, and that could affect mobilized shear stresses.
    - Mechanism #10: Pore pressures.
    - Mechanism #11: Seismic loading.
    - Mechanism #12: Increased operating temperature.
- Category 4 includes one Mechanism related to the factor of time.
    - Mechanism #13: Long-term ageing and creep of geosynthetics.
- Category 5 includes two Mechanisms related to variability.
    - Mechanism #14: Variability of material manufacturing.
    - Mechanism #15: Variability of installation practices.

2.1 *Mechanism #1. Non-uniform mobilization of shear stresses.*

This Mechanism, which was previously described in Section 1.4, is reiterated here for the sake of completeness. Byrne (1994) was the first landmark paper to clearly demonstrate, using numerical analyses, that the non-uniform mobilization of shear stresses is a significant cause of the initiation and progression of failures in strain-softening materials for bottom liner systems. The reason for this is that with deformable materials some areas will be stressed much higher than the average, which could cause those areas to exceed their peak strength. Such a situation can be further exacerbated if those areas have weak spots (e.g., localized delamination) or other conditions (e.g., pore pressures, downdrag) that increase the shear demand on adjacent areas, as discussed later as contributory mechanisms. In the present paper we list this Mechanism first in our list of reasons why peak strength could be exceeded, as we highlighted more than 20 years ago in Thiel (2001). Acknowledgment of this fact established our acknowledgment that waste and soil materials do not act like rigid bodies, and that limit equilibrium analyses are only approximate tools that do not accurately represent physical reality.

We also list this as the #1 Mechanism because (a) this mechanism will co-exist with all other mechanisms that may exist, and (b) this mechanism is generally not intuitive and often needs to be explained and emphasized to practitioners. It could be true that many, perhaps even the majority of designers who perform limit equilibrium slope analyses, have not studied the implications of the numerical analyses presented in the literature cited herein. In the evaluation of slopes that involve strain-softening interfaces, which includes most lined containment facilities, limit equilibrium analyses on bottom liners should be overseen by trained geotechnical professionals who have studied these principles and can exercise appropriate engineering judgement.

While the principle of the static non-uniform distribution of shear stresses is general to all geotechnical structures, its effect is generally less pronounced for veneer liner systems due to the quasi planar load distribution Considerations of non-uniform distribution of shear stresses in veneer systems is typically ascribed to other mechanisms such as differential settlement below the liner, localized pore pressures, construction activities, or variabilities in materials or installation as described below.

2.2 *Mechanism #2. Waste settlement, stiffness, compressibility, creep, and degradation.*

Localized strength degradation of liner interfaces due to waste settlement that results in significant displacements adjacent to the lining system is probably the most commonly cited reason for the promotion of progressive failure (Stark and Poeppel, 1994; Long et al., 1995; Gilbert and Byrne, 1996; Richardson and Thiel, 2001; Thiel, 2001; Sabatini et al., 2002; Kavazanjian et al., 2006; Stark, 2022).

Slope stability studies based on continuum numerical approaches described in the literature references (e.g. Byrne, 1994) generally model waste settlement that is due only to the elastic compression of the waste. The development of relative displacements occurs more readily and to a greater extent on sideslopes as compared to the flatter base of a deep-seated critical surface. Reddy et al. (1996) and Gilbert et al. (1996) both mention that waste stiffness is a major factor affecting mobilized shear stresses and displacement distribution. More compressible, less stiff waste results in more accumulation of strain along the base, especially towards the toe, due to the transfer of stresses from the waste above the sideslope to the waste on the base that acts as a buttress.

Note on the concept of liner-system integrity related to downdrag. Downdrag of the waste along a liner system due to settlement also affects another aspect of liner-system performance, which is generally termed 'integrity.' This concept regards the issue of strains in the liner system that might ultimately cause a tensile failure in the liner, particularly at the crests of slopes at intermediate benches, and the top anchor trench where strains are the greatest. The issue of 'integrity' is not to be confused with the issue of slope stability, though they are related to each other by the causal mechanism of waste settlement and by the nature of the interface shear strengths. The numerical analyses that are used to evaluate both issues are quite similar. Thus, discussion of the integrity issue is relevant to the subject of the present paper. The subject of liner integrity being compromised due to waste settlement was apparently first discussed by VonPein and Lewis (1991), with the first numerical solutions being proposed by Long et al. (1995). Yazdani et al.

(1995) measured smooth HDPE geomembrane strains on a 2.1 m high 3(H):1(V) slope in a California landfill to confirm design assumptions and confirmed that the maximum strains occurred near the top of the slope. Villard et al. (1999) measured strains and displacements on a slope in a full-scale field experiment and correlated their results with a numerical analysis that showed, interestingly but not surprisingly, that the maximum strains in tension and compression occurred where there was the least relative displacement (at the toe and crest of the slope), with the minimum strain (zero) occurring where there was the maximum relative displacement in the middle of the slope. Another numerical simulation of a failure of integrity was presented by Jones and Dixon (2005), a work that was carried forward by the same group in the UK as published in a series of papers related to doctoral work presented in Fowmes (2007). The subject of integrity has received much additional attention in the past decade as well, including publications by Thiel et al. (2014), Yu and Rowe (2018), and Gao et al. (2022). The focus of these studies has been the impact of waste settlement on strains in the geomembrane, rather than relative displacements along the geomembrane. The assumptions in these cited papers from this past decade did not model the strain-softening characteristics of the interfaces but assumed the most conservative case, from the point of view of the criterion of integrity failure, namely that of peak strengths being maintained, to illustrate the worst-case strains at the top of the slope. The important lessons learned from these papers, relative to the subject matter of the present paper, are:

- The length of a sideslope, sideslope inclination, waste loading and settlement, and relative interface shear strengths above and below a geomembrane all influence the maximum geomembrane strain (Yu and Rowe, 2018). Similar conclusions were drawn by Gao et al. (2022), and those same factors would also be relevant to the amount of potential displacement on strain-softening interfaces.
- Sideslope flattening provides the most significant mitigation against downdrag and sideslope displacement problems. VonPein and Lewis (1991) state that “there does not appear to be any problems where the slopes are 3(H):1(V) or flatter”, based on their experience and field observations. Yu and Rowe (2018) state that numerical analyses, for the conditions they evaluated, indicate that without geosynthetic reinforcement, slopes steeper than 3(H):1(V) display long-term problematic strains, while the 3(H):1(V) slopes had acceptable strains. Similar conclusions were later expressed by Gao et al. (2022). The penalty for flatter slopes, of course, is reduced airspace for waste or mining ore.
- Increasing the number of intermediate benches on a sideslope is very beneficial in that it reduces the maximum liner strains (Breitenbach and Athanassopoulos, 2013; Thiel et al., 2014; Yu and Rowe, 2018; and Gao et al., 2022), and is also a means of improving the overall slope stability in the case of weak and strain-softening interfaces, as will be discussed later in Section 3.5.
- Gao et al. (2022), who employed the most sophisticated constitutive model for the waste fill, were able to show that mechanical creep and biodegradation can be significant factors in the development of tensile strains in a liner system and can lead to a continuing increase in maximum tensile strains (and displacements) after capping of the landfill.
- The benefit of introducing a stiff, strong geosynthetic reinforcing layer (e.g., a geogrid or high-strength geotextile) to reduce strains in the sideslope geomembrane was emphasized as a possible solution by Long et al. (1995), Thiel et al. (2014), and Yu and Rowe (2018).

This Mechanism is not considered applicable for veneer systems. Related mechanisms that would be applicable to veneer systems would include foundation settlement and construction activities, as discussed below.

2.3 *Mechanism #3. Static lateral spreading of waste.*

The issue of static out-of-slope lateral spreading of waste causing displacements along a bottom liner appears to have been raised first by VonPein and Lewis (1991), who suggested that “it is probably reasonable to assume that the toe of a large canyon landfill will move 60 cm or more during filling.” Stark et al. (2000) presents inclinometer data taken at the bottom of a 16.5 m high waste fill into the underlying native clay, installed adjacent to, and immediately after, the 1996 Rumpke landfill slope failure, which showed significant out-of-slope movements of the waste that extended into the native clay. They suggest that this phenomenon could have caused excessive shear displacements in the weak base layer that contributed to the progressive failure mechanism of strain softening in the native clay that resulted in the Rumpke slope failure. The reason for static lateral spreading is described by Duplancic (1990), who presents landfill inclinometer data similar to that of Stark et al. (2000), and states that: "Fills on slopes commonly experience lateral deformation due to the lateral force component imposed by the slope." This mechanism is generally not considered applicable for veneer systems.

2.4 *Mechanism #4. Foundation settlement.*

Gilbert and Byrne (1996) refer to foundation settlement as another mechanism that could cause localized deformations that lead to the exceedance of peak strength of strain-softening geosynthetic interfaces. This factor might be especially prevalent in the case of piggyback liner systems installed atop old waste. USEPA (2004) describes potential magnitudes and effects of waste settlement on cover systems. In general, total settlement below veneer systems would be expected to have negligible impact on displacements, while differential settlements will locally increase shear stresses in a manner that would promote progressive displacements on the critical interface.

2.5 *Mechanism #5. Strain/deformation incompatibility between overlying waste/ore material and the liner system interface.*

A situation where the critical failure surface is along the base of a bottom liner system, and then daylights up through the waste, illustrates the concept of 'strain incompatibility' between a strain-softening interface and the overlying waste. This has been emphasized by Stark et al. (2000) as being a mechanism that can promote progressive failure. It is a dynamic of 'incompatibility' which occurs because as the waste/ore mobilizes shear stresses in order to resist collapse, the degree of strain needed to mobilize peak stresses on the strain-softening interface is quite low as compared to the degree of strain needed to mobilize significant shear stresses in the waste/ore material. The result is an unbalanced development of shear stresses mobilized 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 was identified by Stark et al. (2000) as a contributing factor in the Rumpke failure. This mechanism is generally not considered significant for veneer systems.

2.6 *Mechanism #6. Waste placement activities.*

Stark and Choi (2004) mentioned waste placement activities as a contributory factor in promoting progressive failure, referencing Yazdani et al. (1995) as a source. As discussed in Mechanism #2 above, the field measurements presented by Yazdani et al. (1995) represent the strain that would accumulate from waste settlement, and the data collected in that study would be more germane to the issue of downdrag as related to 'integrity' as discussed above. However, one could also imagine that the effects of waste or mine ore operational placement activities could be similar to those of construction activities, as described in Mechanism #9 (Section 2.9), where the presence of heavy equipment in proximity to the liner interface could potentially induce additional temporary dynamic forces that might cause some localized displacement, especially where there are weak spots that would act as stress concentrators that could contribute to progressive failure. In the context of veneer systems, this mechanism would be related to construction activities as described for Mechanism #9.

2.7 *Mechanism #7. Slope overfilling/oversteepening.*

Normal waste filling and its concomitant settlement have already been mentioned as being the most commonly recognized mechanism causing liner displacement (Mechanism #2). Stark et al. (2000) mention *over*-filling as an exacerbating factor when the filling exceeds the approved design/operations plan. This often happens when a new cell is not ready in time and the existing capacity of the landfill is overextended. Overfilling then creates additional shear stress and deformations that are often beyond the design limitations. This mechanism is really an extension of the mechanisms of non-uniform mobilization of shear stresses (Mechanism #1) and waste or mine ore settlement (Mechanism #2) but is listed as a separate mechanism that is a result of operational decisions that can promote progressive failure. This mechanism was identified by Stark et al. (2000) as a contributing factor in the Rumpke failure.

2.8 *Mechanism #8. Toe excavation.*

Though the removal of a small amount of toe buttressing may seem innocuous, it can actually be quite devastating due to the initiation of non-uniform mobilization of shear stresses and the static lateral spreading of waste. The plots of the numerical analyses performed by Byrne (1994, Fig. 10 of that paper) for mobilized friction angle, and by Filz et al. (2001, Fig. 6 of that paper) of the mobilized shear stresses clearly indicate that the real-world phenomenon of non-uniform mobilization of shear stresses (Mechanism #1) concentrates shear stresses at the toe of the fill on the base. Stark et al. (2000) point to the excavation at the toe of the Rumpke landfill as being not only a contributing factor to the failure, but also a factor that allowed the runout of the translational landslide to extend further than it otherwise might have.

What is particularly pernicious about a toe excavation is that it reduces buttressing, thus inviting progressive lateral displacements, and could be the triggering mechanism for a failure. Given that real-world mobilization of shear stresses favors increased stresses at the toe, as demonstrated by Byrne (1994), a small displacement that exceeds the peak strength at the toe makes that zone weaker, which means it can carry less of the lateral load. That in turn puts more of the load on zones adjacent to the critical plane, which may in turn cause another part of that plane to become overstressed and exceed its peak strength (which is the classical description of progressive failure). This mechanism is also applicable to veneer situations. The full-scale field study by Villard et al. (1999) showed how removal of the toe at the base of a veneer fill significantly increased the geosynthetic strains. Stark et al. (2012) describes the value of a toe buttress when constructing veneer layers.

2.9 *Mechanism #9. Construction-induced shear strength degradation.*

Concern over the effects of deformations and displacements on the long-term operational peak strength of interfaces that occur during construction has been previously expressed by Gilbert and Byrne (1996), Thiel (2001), Sabatini et al. (2002), and Stark and Choi (2004). None of these sources, however, specifically addressed exactly how construction activities could impact liner system interfaces. The present paper provides updates regarding this issue.

The construction of geosynthetic-lined containment facilities commonly involves a relatively thin layer of soil to be spread over one or more geosynthetic layers; this applies to both bottom liner and final veneer liner systems. A key point here is that the *localized* shear stresses caused by a soil-spreading operation using a dozer are significantly higher than the *average* shear stresses that are assumed to be distributed over the entire slope length. This is due to forces needed to overcome the friction at the base of the soil pile being spread, the weight of the dozer, and any acceleration/deceleration of the dozer.

We would note that this consideration is completely different from the usual consideration of the stability of equipment operation on slopes that is most commonly cited from sources such as Koerner and Soong (1998), Qian et al. (2001), Druschel and Underwood (1993), McKelvey (1994), and USEPA (2004). These references only consider an entire slope reach, and equipment stresses are assumed to be distributed over the entire slope length, often with the objective of calculating the anchorage strength required to secure the geosynthetics, thereby preserving slope stability during this type of construction. If the veneer stability of the entire slope length is at issue, then the references cited in this paragraph can be used, and the calculation is straightforward.

A much more pernicious situation, which is dangerous from the point of view of progressive failure, is if the peak shear strength of any of the interfaces is exceeded by the construction-induced *localized* shear stresses. In this case the shear resistance of these interfaces will be degraded little by little as they experience relative displacements during construction. Such localized relative displacements, and the resulting localized shear strength degradation, may or may not be apparent as construction proceeds. Obvious failures that the author has seen in this regard include a case history presented in Thiel and Narejo (2005), and another confidential case history used as an example in Thiel and Giroud (2023). Localized track spinning, which would cause relative displacements of a geotextile to a textured geomembrane on the order of one to twenty centimeters, would be deleterious to the integrity of the interface's peak shear strength at all locations where that occurred. These types of small but impactful slippages could occur over and over without attracting the attention of the dozer operator or the construction observer. The cumulative effect of such localized shear strength degradation events over the course of construction of an entire slope can thus be seen as detrimental to the slope's static and dynamic stability in the long term, especially in light of a progressive failure.

There are five references that suggest methods to quantify the elevated localized shear stresses below the dozer tracks could cause localized exceedance of shear strength: Paruvakat and Richardson (1999), Kerkes (1999), Jones et al. (2000), Thiel and Narejo (2005), and Thiel and Giroud (2023). Each of these references either adds to or improves upon the work presented in the other references, and taken as a whole, they provide useful approaches to quantification of the problem, as well as suggestions for construction specifications and construction quality assurance (CQA) that can mitigate the problem.

To the author's knowledge, the effect of interface deformation at low normal stresses on the subsequent shear strength at higher normal stresses has only been documented in one study, that of Esterhuizen et al. (2001). They showed that for a particular smooth geomembrane/clay interface, deformations at low normal stresses would reduce the peak strength of the interface at higher normal stresses. They present results showing that the peak shear strength at 345 kPa normal stress was reduced by approximately 13% due to pre-shearing at 35 kPa normal stress. They provided an interesting "work-softening" model to describe this behavior in a manner that can be used in a finite-element analysis. Although their model fits the data very well, it is only applicable to the specific clay and geomembrane used for their study.

Limited testing was performed for the present paper in order to provide some insight into this issue for a textured geomembrane/geocomposite interface, where the geocomposite surface was a nonwoven geotextile that was heat-bonded to a geonet. Two cases were checked: one for high normal stress (bottom liner) situations and one for long-term low-normal stress (veneer) situations. For the high-normal stress situation, this particular interface was pre-sheared at a low normal stress of 24 kPa, representative of dozer loading, and then final-sheared at a higher normal stress of 192 kPa. The results, presented in Figure 3, indicated that the peak strength at the high normal stress was reduced by approximately 13% due to pre-shearing at the dozer construction stress, as compared to shearing a virgin sample at the high normal stress. For the low-normal stress situation, this particular interface was pre-sheared at a construction normal stress of 24 kPa to represent the dozer loading, and then final-sheared at a lower normal stress of 10 kPa, representative of the typical long-term loading of a cover system. The results, presented in Figure 4, indicated that the dozer-induced pre-shearing resulted in LD shear strength under the design normal stress of 10 kPa to be approximately 42% lower than the peak strength that would typically be obtained by shearing a virgin sample.

Note related to the expansion/contraction of exposed geosynthetics. Stark and Poeppel (1994), Stark and Choi (2004), and Zamara et al. (2014) mention thermal expansion/contraction of exposed geosynthetics as being a possible cause that peak strength could be reduced at strain-softening interfaces. The present author has commissioned testing on two separate projects that used different types of textured geomembranes that had been dragged over non-woven geotextile based GCLs during deployment to determine if the dragging caused any degradation of shear strength as compared to virgin materials tested at 400 kPa normal stress. The result was that no perceptible differences were noted. The author is not aware of any other such zero-normal-load shear-degradation studies that have been conducted. This cause for possible peak strength degradation is included here as a subset of potential construction-induced strength degradation, a possibility that remains to be verified by further testing.

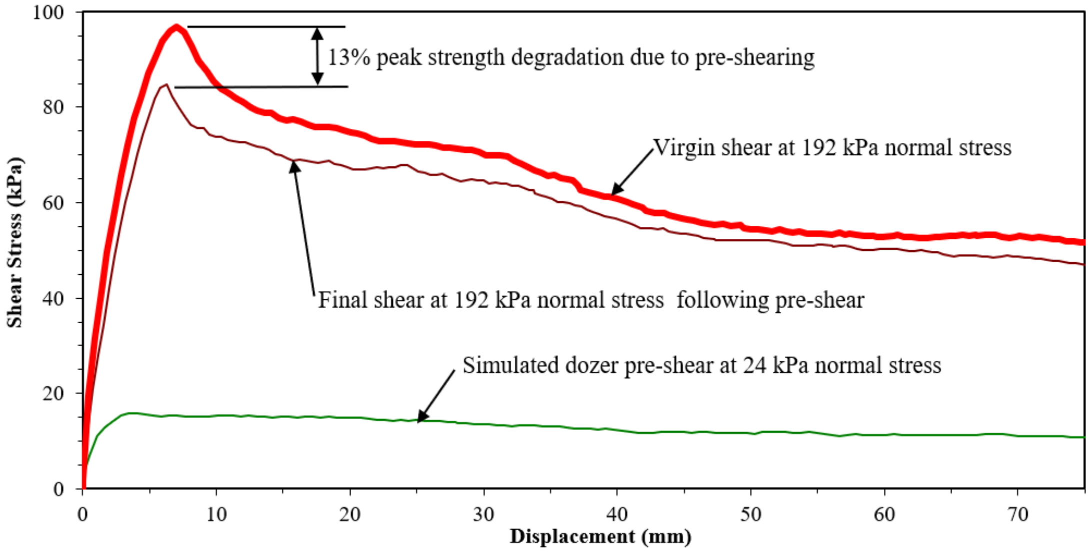

Figure 3. Test results showing effect of pre-shearing at low normal stress on peak strength at high normal stress for an interface of a textured HDPE geomembrane against the nonwoven geotextile surface of a drainage geocomposite.

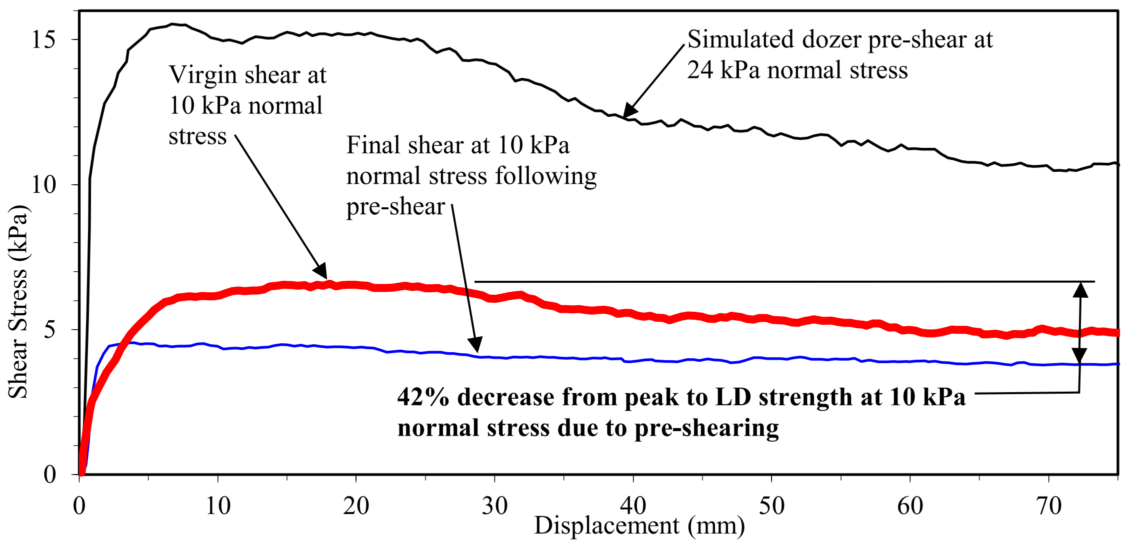

Figure 4. Test results showing effects of pre-shearing at dozer construction normal stress on peak strength at low veneer normal stress for an interface of a textured HDPE geomembrane against the non-woven geotextile surface of a drainage geocomposite.

2.10 *Mechanism #10. Pore pressures.*

Apart from gravity, pore pressures (most pervasively those caused by liquid, but also possibly caused by gas) are the single most prevalent factor contributing to slope stability failures. The reason for this is that pore pressures reduce the effective normal stress that gives interfaces their shear strength, but the applied shear stress is often unaffected by pore pressures. Examples of significant excess pore pressure buildups in bottom liner systems that have been observed are failure or malfunction of pumps, power, or controls; leachate collection systems that experience collapsed pipes (observed by author via in-pipe cameras and exhumations); and leachate collection systems that clog due to poor design (Koerner et al., 1994) and/or long-term inorganic precipitation where biological clogging is typically a catalyst (Rowe and VanGulck, 2004). Localized waste saturation can create conditions for the development of significant pore-gas pressures which have been measured in excess of 200 kPa that are suspected of causing a stability failure in a high-food-waste-content MSW landfill (Ma et al., 2019). In final cover systems the overloading of the drainage layer on top of the barrier layer is the most common cause of the many veneer-cover failures caused by inadequate transmissivity, susceptibility to being overloaded with water during construction, and inadequate or blocked outlets. Even if pore pressure buildups are just temporary, such as during a control system failure in a storm event, progressive failure can be triggered.

Pore pressures on the underside of the liner system could manifest as a result of saturated high-plasticity clays in the subgrade that do not have adequate time to dissipate relative to the rate of loading for bottom liner systems, or landfill gas pressures for final cover systems.

2.11 *Mechanism #11. Seismic loading.*

Seismic loading increases the potential for displacements to occur along the critical failure plane which, in conjunction with the non-uniform mobilization of shear stresses, can locally (or globally) cause displacements that reduce the strength of the critical interface below its peak strength, thus leading to progressive failure. In this regard the design practitioner should assess the potential for this type of deformation.

When performing calculations to determine if seismic deformations due to the design earthquake will be within acceptable limits (as defined by standard practice or the regulations), LD or residual values for the strengths should be assumed along the entire critical interface for purposes of those calculations, even if peak strengths have been determined to be acceptable for purposes of the static stability (Kavazanjian, 1999; Kavazanjian, 2023). The design should then be checked to meet other project-specific standards or regulatory requirements separately for static FS, and for the maximum estimated displacement due to the design earthquake.

2.12 *Mechanism #12. Variations in operating temperature.*

Hanson et al. (2015) performed laboratory testing to determine the effects of temperature on the shear strength of the interface between a textured HDPE geomembrane (asperity height of 0.45 mm) and a needle-punched GCL having a woven geotextile interface with the geomembrane. Tests were performed for low-normal load (cover system) applications at an average normal stress of 15 kPa, and high-normal load (bottom liner) applications at an average normal stress of 150 kPa. The temperatures for the low-normal load testing were 2, 20, and 40°C. The temperatures for the high-normal load testing were 20 and 40°C. Their research suggests that the design values of shear strengths for interfaces between textured geomembranes and a non-woven geotextile-based product could potentially be prorated downwards by as much as 15-20% for cover systems, and 10-15% for bottom liner systems, to account for the effects of temperature.

Karademir and Frost (2021) performed an extensive interface shear program to evaluate the effects of temperature on interface shear strength between several different types of geomembranes (smooth PVC and HDPE, as well as three different types of textured HDPE) and needle-punched nonwoven (NPNW) polypropylene (PP) geotextiles. Tests were performed at normal stresses of 10, 100, and 400 kPa, and at temperatures of 21, 26. 30. 35. 40, and 50°C. In all cases their results indicated that the interface shear strength increased with temperature. The increases in strength going from 21 to 50°C ranged from 14-23% for peak strength and from 14-18% for large displacement strength for the various interfaces tested. They concluded that for the range of normal stresses and materials tested that interface shear behavior determined at room temperature yields interface friction values that are conservative.

Given the differing conclusions between the two studies mentioned above, the present author does not recommend adjustments to interface shear strength due to temperature effects, and that the standard factor of safety of 1.5 would be appropriate to account for variations in this regard.

2.13 *Mechanism #13. Long-term ageing and creep of the geosynthetics.*

Several authors have pointed to long-term ageing and creep as being potentially significant contributors to a mechanism of progressive failure along strain-softening interfaces for both high- and low-normal-load situations (e.g., Skempton, 1964; Byrne, 1994; Gilbert and Byrne, 1996;

Breitenbach, 1997; Thiel, 2001; Sabatini et al., 2002; Zanzinger and Alexiew, 2002; Jones and Dixon, 2003; Zanzinger and Saathoff, 2012).

Trauger et al. (1997) performed long-term internal shear testing of soaked reinforced GCL specimens at both low (24 kPa) and high (97-389 kPa) normal stresses with applied shear stresses equivalent to 26.6° and 19.3° friction, respectively, for up to 10,000 hours without shear failure. Zanzinger & Alexiew (2002) performed long-term internal shear testing on GCL specimens at low normal stresses with applied shear stress ratios of up to 90% of the short-term internal shear strength on reinforced GCLs for up to 5,000 hours without shear failure. These studies indicate good long-term durability for GCL reinforcement exclusive of ageing of the geotextile fibers.

Marr and Christopher (2003) considered long-term ageing and creep of the internal reinforced needle-punched fibers of GCLs. This is important because if the internal shear strength of the GCL exceeds its peak then the remaining residual strength will be that of hydrated bentonite, which can be as low as 4° friction. Most designs provide another interface that is weaker than the peak internal strength of the GCL to 'ensure' that the peak internal GCL strength never fails (this is known as the 'fuse' concept, where the 'critical interface' is defined as the one that has the lowest peak strength, even if it does not have the lowest residual strength). Marr and Christopher (2003) cautioned that long-term creep could challenge this design concept, and recommended that the following reduction factors (R_f) be applied to the *difference between* the peak and residual internal shear strength of the project-specific GCL: $R_{f\text{-}cr} = 3$ to account for long-term creep; and $R_{f\text{-}age} = 1.1$ (100-year life) or 2.0 (300-year life) to account for ageing. The two values of R_f would be multiplied by each other to yield a total R_f ranging from 3.3 to 6.0. The resulting value would be *added back* to the residual value of the GCL internal shear strength to obtain the maximum allowable long-term internal design strength, $\delta_{GCL\text{-}all}$, of the project-specific GCL. Marr and Christopher (2003) further suggested that to prevent failure from occurring inside the GCL, another interface should be provided in a layer above the GCL that has a short-term peak interface strength less than $\delta_{GCL\text{-}all}$. This latter goal cannot always be achieved within the constraints of the available materials and design goals, and the present author suggests that the same design intent could also be met by verifying, through analyses, that the long-term mobilized shear stress of the design is less than $\delta_{GCL\text{-}all}$. Note that this type of calculation can only be quasi-reliable when numerical analyses are used, since limit equilibrium methods do not provide an accurate picture of the true mobilization of stresses or strains of deformable bodies. The present author believes that this approach is conservatively biased because the high peak internal shear strength at high normal stresses for needle-punched reinforced GCLs, even under fully hydrated conditions, is much greater than the sum of the bentonite shear strength and geotextile tensile strength, due to some mechanism that is not fully understood at this time (Thiel and Maubeuge, 2002). The R_f values suggested by Marr and Christopher (2003), therefore, might not need to be applied to the entire difference between the peak and residual internal shear strength of the GCL. More research is needed in this regard.

Abdelaal and Solanki (2022) performed laboratory testing to investigate, among other things, the effect of geotextile ageing on the interface shear behavior, using a 2 mm thick blown-film textured HDPE geomembrane having an average asperity height of 0.45 mm. Three different single-layer non-woven needle-punched staple fiber geotextiles with mass per unit areas of 200, 580, and 1500 g/m^2 were tested at normal stresses of 250, 700 and 1000 kPa. The results showed that for the interfaces that involved geotextiles that were aged prior to the shear box experiments for up to 2 years at 85°C, all the highly aged single-layered geotextiles showed an increase in the peak interface friction angles as their ageing increased. For these single-layered geotextiles, the results suggest that assessing the interface friction angles using unaged geotextiles for a stability analysis is a conservative practice as long as the geotextile remains intact in the field. This is a welcome finding in light of the previous ambiguity that dogged the question of long-term ageing relative to shear strength, at least for interfaces involving nonwoven geotextiles set against textured HDPE geomembranes.

2.14 *Mechanism #14. Variability of material manufacturing.*

Consideration of material variability is endemic to geotechnical engineering in which the accurate characterization of soil properties is often a statistical endeavor. One of the often-touted benefits of geosynthetics is their relative uniformity, compared to soils, due to their being manufactured under controlled conditions. While there is merit to this perception, casual acceptance of this as a fact has led to abuse in the adoption of geosynthetics via the assumption that single tests, especially those related to shear strength, can be taken as representative for entire projects. Even worse is when published values of shear strength are blindly accepted as a design basis with no further qualification.

The idea that facilities constructed without the benefit of project-specific testing can be less reliable than those verified with testing is generally well accepted and espoused in the literature (e.g., Richardson et al., 1998; Thiel, 2001; Sabatini et al., 2002; McCartney et al., 2004; Dixon et al., 2006). Construction conformance (verification) testing of shear strength is standard practice for lined containment facilities where slope stability is important.

Often it can be time consuming and costly to conduct numerous interface shear performance tests during construction. For that reason, index tests are often performed, where it is presumed that attainment of certain minimum index values will infer that the shear strengths that had been verified by performance tests will be achieved. Example index tests that might be relevant to shear strength for soils could include grain size distribution, Atterberg limits, clay fraction, moisture, and density. Example index tests that might be relevant to shear strength for geosynthetics include asperity height for geomembranes, and peel strength for GCLs and geocomposites. Although precise correlations between the results for these various index tests and shear strength might not be available, engineering judgement indicates that replication of benchmark values that have been previously demonstrated to be satisfactory should result in acceptable performance.

However, even when conformance testing of materials supplied to a job site is performed, weak locations can exist that are not representative of the average strength may become host to impactful stress concentrations. This fact, when combined with the fact of non-uniform mobilization of shear stresses (Mechanism #1 above), could result in a localized progressive exceedance of peak strength that could contribute to a stability failure. Examples of geosynthetic manufacturing variability that have been observed to create weak zones are non-uniform texturization of HDPE geomembranes (e.g. 'tiger striping'), variation of peel strength across the roll width of GCLs, and variations of peel strength of GCLs from beginning to end of needle-board changes during the manufacturing run. Perhaps one of the most highly variable interfaces that currently exists for geosynthetic shear interfaces is the heat bonding of a geotextile to a geonet that is commonly performed to create a geocomposite drainage layer. While the relationship between peel strength and (internal) interface shear strength of the geotextile/geonet products is not well understood, an attempt to study this and demonstrate such a relationship exists was published by Thiel and Narejo (2005) as a result of an investigation of a field failure of this interface. Standard testing for geocomposite peel strength, which is an index of shear strength, is almost always performed in the USA according to ASTM D7005, Standard Test Method for Determining the Bond Strength (Ply Adhesion) of Geocomposites. This test only requires reporting of the results based on the average of five 100 mm wide specimens across the panel width. Because there is a natural laboratory bias towards cutting specimens from the sample that do not fall apart, zero-strength specimens are almost never taken, even though they commonly exist due to manufacturing limitations. Thiel and Gatrell (2019) tested the variability of the peel strength of samples in which contiguous 100 mm wide specimens were cut in a checkerboard pattern across the panel width. Figure 5 presents the results for a sample that yielded 38 specimens, which resulted in an average peel strength of 290 N/m with a standard deviation (assuming normal distribution of data) of 230 N/m, not counting the unbonded edges of the panel. It is noteworthy that even though the average value was soundly above the target specification of 175 N/m and 'passed' the conformance testing requirements, 12 of the 38 specimens (32%) were below the target specification. Of these 12, 7 of the values (18.4%) were less than one-fourth (25%) of the target specification, which is very low (< 44 N/m). The test results presented in Figure 5 may represent the lower end of quality that can be achieved for geocomposite bonding, but such results have been qualitatively reported to the author by others and experienced by Thiel and Narejo (2005).

The study concluded that the current practice of taking only five specimens across the panel width is inadequate to verify the variability of bonding across geocomposite drainage products that are created by heat bonding. In addition, even with contiguous specimen testing across the panel width the question remains as to which value would be representative for design purposes, considering the strain-softening nature of the interface.

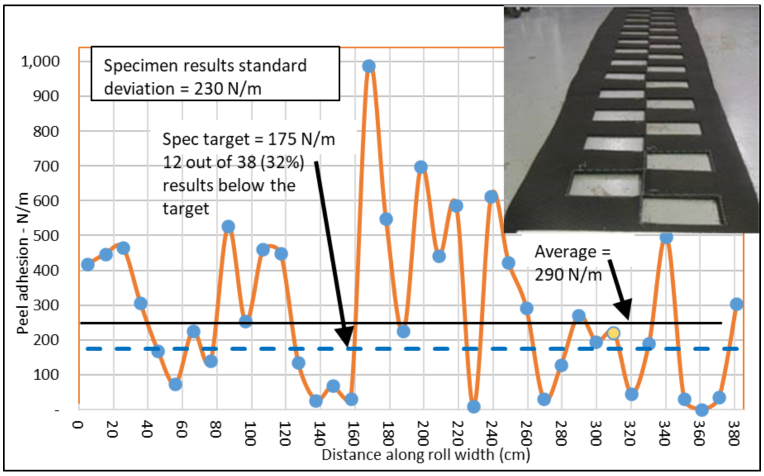

Figure 5. Results of contiguous peel tests across geocomposite panel width (from Thiel and Gatrell, 2019).

2.15 *Mechanism # 15: Variability in the installation of geosynthetic interfaces*

The present paper introduces the idea that variability in installation practices is a potentially significant contributor to the introduction of repetitive weak zones in geosynthetic installations, as related to the seaming of geosynthetics. Examples of this apply both to high- and low-normal-load situations and include:

- Smooth edges on otherwise textured geomembranes that are intentionally manufactured to improve seam quality. The width of the smooth edges is commonly 0.15 m on both sides of a 6.86 m wide panel. Considering that seam overlaps are commonly 0.1 m, this leaves 0.2 m of smooth surface for every 6.76 m, or about 3.0% of the area.
- Geocomposite drainage layers comprised of geonets with geotextiles heat-bonded to one or both sides typically have the geotextiles unbonded along each edge to allow for seaming. The width of the unbonded edges is commonly 0.3 m on both sides of a 4.42 m wide panel. Considering that seam overlaps are commonly 0.1 m, this leaves 0.5 m of unbonded surface for every 4.32 m, or about 11.6% of the area.
- GCLs are commonly overlapped with an approximately 0.08 m wide ribbon of free bentonite applied within the overlaps. The width of the overlap is commonly 0.15 m on both sides of a 4.42 m wide panel. This leaves 0.08 m of potentially hydrated loose bentonite for every 4.27 m, or about 1.9% of the area.

The shear strengths of each of the seam zones for each of these materials will generally be substantially weaker than those of the non-seam zones, depending on which type of materials are placed against these interfaces. It is possible that some designers have taken some of these considerations of installation variability into account and have prorated the design shear strength accordingly. However, even simple proration of shear strengths may not be a completely adequate response to this issue with strain-softening materials in bottom liner situations because of the potential consequences of shear stress concentrations that would likely occur at the edges of these weak inclusions.

The relative significance of this issue depends upon not only the pervasiveness of the weak zones and the degree of their weakness, but also the configuration of the site-specific lining system. Consider the example of a liner system that was used in a design example presented by Qian and Koerner (2010), shown in Figure 6.

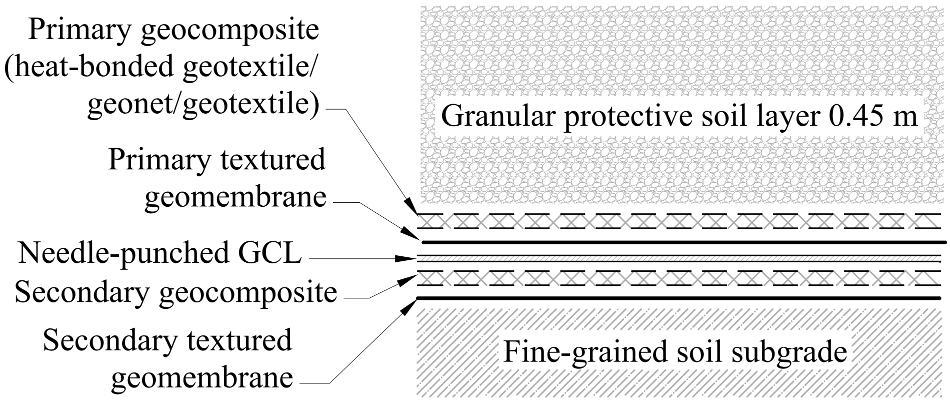

Figure 6. Detail of liner system that was used in a design example presented by Qian and Koerner (2010).

The detail shows that there are two layers of geomembrane, two layers of geocomposite, and one layer of GCL, all in close proximity to one another (<15 mm). The shear strengths of each of the seam zones for this cross-section could be evaluated as follows:

- The smooth edges of both geomembranes would have one side against the geotextile surface of a geocomposite. Stark and Richardson (2000) report secant peak and residual smooth geomembrane/geotextile shear strengths as being 9° and 5°, respectively, with peak strength mobilization occurring at 2 mm of displacement, for a normal stress of 400 kPa. This compares to peak and residual secant strengths of 30° and 15°, respectively, for textured interfaces, with peak strength mobilization occurring at approximately 6 mm of displacement.
- Regarding the geocomposite panel edges with the unbonded geotextile, limited testing was performed for purposes of the present paper in order to provide information regarding the interface strength between a loose geotextile and a geonet with the shear taking place *parallel to the geonet rails*. Whereas testing the shear strength of this interface in either the machine- or transverse-panel-direction of the geonet will deliver apparently high friction values, the shear strength parallel to the geonet rails is very low, as can be experienced by simply walking around construction sites and stepping on the edge of an unbonded geocomposite. The test results indicated peak and large-displacement geonet/geotextile shear strengths of 12.5° and 9.7°, respectively, over a normal stress range of 50-200 kPa, with peak strength mobilization occurring at approximately 5 mm of displacement.
- Since in this case the GCL is designed to be installed in the dry secondary layer, bentonite hydration in the seam is ignored in this exercise.

Consider the installation of this liner system on a 3(H):1(V) sideslope. Typically, the geosynthetic materials would be deployed with their machine-direction going downhill in the direction of the slope. Since the unbonded geonet edge strength was measured parallel to the geonet ribs, this is the orientation that should be considered. Figure 7 is a photograph of the sample testing in the laboratory showing a 25° angle of the geonet ribs relative to the machine direction of the geonet. (This value will be specific to the product being tested.) To account for this orientation when considering the shear stresses and potential for progressive failure on a 3(H):1(V) (18.4°) slope, it can be calculated that the angle of the sideslope at a 25° skew is 16.7°. Adding up the tributary areas of the weak seam zones for the two layers of geomembranes and the two layers of geocomposites yields a result of $2\times(3\%+11.6\%) = 29.2\%$ of the area having a shear strength of less than 10° friction at the critical inclination of 16.7°. This indicates a very significant proportion of weak areas could exist that have a shear strength substantially less than the critical slope inclination, and which would be significantly less than either the peak or residual strength of a textured geomembrane/geotextile interface.

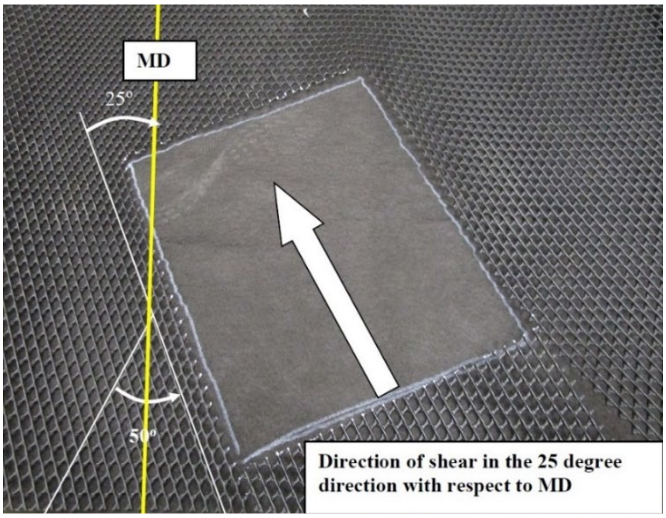

Figure 7. Photograph of test setup with geotextile on top of geonet for shear parallel to the geonet ribs. (Courtesy of SGI Testing, Atlanta, GA)

The conclusion we reach from this discussion of Mechanism #15 is that the seaming mechanics required for each of the different types of geosynthetics can introduce regularly spaced weak areas in the liner system that provide sites for the initiation and promotion of progressive failure. The detrimental effects of these weak zones may also be exacerbated by shear stress concentrations that induced by the sudden changes in shear strength at their edges. A related weak-zone issue can also be created during installation by leaving excess geosynthetic scraps and rubsheet materials below geomembranes rather than collecting these materials and throwing them away, a practice which should be addressed by vigilant specifications and CQA.

2.16 *Note regarding the combined material and installation variability of drainage geocomposites.*

This discussion of the material and installation shear strength variability that is potentially introduced by the use of drainage geocomposites should be cause for designers and owners to exercise a high level of scrutiny when using those materials where slope stability is important. Consider the large variability of material with regard to geocomposite peel strength described in Section 2.14, where test data indicated that 18.4% of the 'bonded' area was less than one-quarter of the specified valued of 175 N/m peel (i.e. < 44 kN/m). A reasonable and prudent assumption is that areas with bonding of less than 44 N/m (0.25 pounds per inch) will become fully unbonded during construction and in-service conditions. For a net installed panel width of 4.32 m this would represent 16.2% of the total installed area for that side of the geocomposite having a low shear strength (< 10° friction) in the direction parallel to the geonet rails. (Although the other side had similar bonding problems, the geonet rails on the other side run in a different direction and so only one side would be counted.) To this could be added the unbonded edge zones that represent 11.6% of the installed area, as described in Section 2.15, now producing a total unbonded area on the order of 27.8% of the total planimetric area that could have a low shear strength (< 10° friction) in the direction parallel to the geonet rails. If two layers of geocomposite are used in a liner system, such as the one depicted in Figure 6, then a value potentially greater than 50% of the lined area in service could have this low degree of shear strength in the direction of the geonet rails due to the poor bonding, depending on how much overlap of the poorly bonded zones occurred between the two geocomposite layers from a planimetric perspective. The debilitating nature of the high frequency of both known and random poorly bonded areas in a geocomposite can play an outsized role in reducing the dependable shear strength. It is highly recommended then, to give special attention to specifications for the bonding between geotextiles and geonets when using drainage geocomposites. Where slope stability is critical, specifications for these materials should be written that require minimal widths of unbonded edge zones, and higher average peel strengths to compensate for the very high standard deviations in manufacturing that seem endemic to these manufactured products. In addition, increased conformance (verification) testing frequency of peel strength would be advised, requiring that more specimens be tested across the panel width, and perhaps even contiguous specimens for CQA, as illustrated in Figure 5. The examples of materials and variability in installation presented herein are for specific products and situations that have been encountered by the author. The frequency, magnitude, and distribution of defects could therefore be quite different for other products in other regions, and for different installation practices. While the author regularly specifies these products with confidence, it is always done with these considerations.

3 THIRTEEN MEASURES THAT CAN BE ADOPTED TO REDUCE OR MITIGATE DISPLACEMENTS THAT CAN LEAD TO PROGRESSIVE FAILURE

There are measures that can be adopted to reduce the tendency for shear displacement along a critical strain-softening interface, thus improving the liner's reliability against failure. The development of relative displacements occurs more readily and to a greater extent on sideslopes as compared to at the base of deep-seated critical surfaces. The target zones for application of the measures described below would have to be evaluated on a case-by-case basis. For example, while it may be advantageous to pursue bolstering of the peak strength of a liner system along the base, to attempt to do so along a steep sideslope could prove detrimental due to the threat of high-strain integrity (i.e. ripping) failures on the sideslope.

This section presents a list of thirteen Measures that can reduce or mitigate the tendency for displacements that might lead to progressive failure in the presence of strain-softening interfaces in high normal load bottom liners, of which ten also apply to veneer situations. These Measures have been grouped into five categories depending upon the treatment mechanism, and can generally be targeted to benefit the base, sideslope, or both areas of a given design.

- Category 1 includes five Measures that are related to geometric modifications that would increase the factor of safety and reduce the magnitude of relative displacements that could occur.
  - Measure #1: Fill slope flattening and avoidance of over-filling.
  - Measure #2: Sideslope flattening.
  - Measure #3: Longer base design.
  - Measure #4: Buttressing of the toe and avoidance of excavation at the toe.
  - Measure #5: Geometric interruptions in the subgrade.
- Category 2 includes five Measures that are related to the attempt to preserve peak strength to the extent possible. Preserving peak strength can be useful for one or more phases of the project, which include construction, operations, and final build-out stability. These measures could target the base liner, or the sideslope liner, or both.
  - Measure #6: Increase peak strength.
  - Measure #7: Reduce weak spots resulting from variabilities in materials and installation practices.
  - Measure #8: Minimize any significant interface damage resulting from construction or waste filling.
  - Measure #9: Mitigate foundation settlement.
  - Measure #10: Implement high strength reinforcement along sideslopes.
- Category 3 includes one Measure that is related to avoiding the transient destabilizing influence of pore pressures.
  - Measure #11: Mitigate potential against high pore pressures.
- Category 4 includes one Measure that is related to the assumption that residual strength will develop along the geosynthetic interfaces.
  - Measure #12: Increase the residual strength of the interface with the lowest peak.
- Category 5 includes one Measure related to adaptive management:
  - Measure #13: Long-term instrumentation and monitoring.

3.1 *Measure #1: Fill slope flattening and avoidance of over-filling.*

Fill slope flattening is a common geotechnical measure taken to improve slope stability and reduce the risk of failure. Since the only reason slope stability is an issue is the presence of a slope, reducing the severity of the slope quite naturally reduces the risk of instability. The penalty for fill slope flattening, of course, is less capacity for waste or mining ore. This Measure has a significant beneficial influence on veneer stability.

3.2 *Measure #2: Sideslope flattening (discussion for bottom liner situations only; see Measure #1 for veneer situations)*

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. A proof of this is provided in the *Geosynthetics International* companion paper appendices.

3.3 *Measure #3: Longer base.*

Having a greater ratio of base length to sideslope length is a proven means of improving slope stability (Reddy et al., 1996; Stark, 2022). This fact is also demonstrated in the example proof described above in Measure #2, which is published in the *Geosynthetics International* companion paper appendices. Even so, a very long base is not a panacea against displacements along a liner with low-stiffness waste materials that can experience significant static lateral spreading. This measure is not applicable to veneer systems.

3.4 *Measure #4: Construct toe buttress and avoid excavation at the toe.*

Constructing a buttress at the toe of a slope is a common geotechnical solution employed to improve the stability of slopes in general. This technique is commonly used in canyon landfills and in valley-fill mining leach pads. The size and resistance of a toe buttress can be designed to allow the containment facility to safely mobilize residual shear strength conditions along the lined interface. A toe buttress is essentially a dam that provides resistance to prevent the waste or mining ore body from sliding down the canyon or valley, or from spreading laterally. Providing a buttress at the toe greatly reduces the likelihood that the toe of the fill will displace, which is important, because numerical analyses have shown that this can be the location that triggers a progressive failure. The obverse of a toe berm is excavation at the toe, which can be especially debilitating and is suspected of having played a role in the Rumpke failure (Stark et al., 2000).

USEPA (2004) shows examples of toe buttresses for veneer situations that not only support the toe but also allow general slope flattening. A variation of a toe buttress is a tapered thickness cover soil (Koerner and Soong, 1998). Designing shorter slope lengths between benches on a veneer slope is another way to increase the effect of intrinsic toe buttressing by limiting the finite slope length of any given slope reach. For practical engineering purposes, when the ratio of the slope length to veneer soil thickness exceeds 10-20, the slope will act like an ‘infinite slope’, and the benefits of toe resistance to slope instability then provide diminishing returns as the slope length increases.

3.5 *Measure #5: Geometric interruptions in subgrade.*

Among the most effective and reliable methods of improving the stability of a containment facility that has strain-softening liner interfaces is to create non-planar interruptions along the critical slip interface that will force the critical failure surface to intermittently pass through stronger materials that are above and/or below the geosynthetic interfaces. This Measure generally applies to bottom liner systems rather than veneer systems. The best overall description of this approach is provided by Breitenbach and Athanassopoulos (2013), who describe three types of geometrical subgrade interruptions that improve stability:

1. Stability Berms. Also referred to as “speed bumps”, stability berms can be constructed in the subgrade before the liner is placed. They cause the critical slip plane to either pass along a non-planar surface, or to pass through higher strength materials. Depending upon the details of the geometry it is likely that both phenomena will occur, and overall stability will be improved as compared to having a planar surface lined with geosynthetics. Breitenbach and Athanassopoulos (2013) provide an example sensitivity analysis which considers the effects of the number, width, height, spacing, and subgrade shear strength of stability berms upon the slope stability factor of safety.
2. Stability Trenches. The geometrical inverse of a berm is a trench, and in this regard a stability trench can provide an effect similar to the that of a stability berm. Trenches can be excavated into the subgrade, lined, and backfilled with the waste, mining ore, or other materials in order to cause the critical slip plane to either pass along a non-planar surface, or to pass through higher strength materials. Breitenbach and Athanassopoulos (2013) note that trenches can create more complications when implemented with gravity drainage systems on the liner than would be the case with stability berms.
3. Stability Benches. The use of benches on slopes to improve stability and reduce downdrag liner strains, a practice that functions similarly to the way stability berms function on the base, has also been emphasized by Thiel et al. (2014), Yu and Rowe (2018), and Gao et al. (2022).

While the implementation of these various techniques to interrupt the subgrade geometry all involve extra earthworks, drainage, and lining complexities, they are viable means of increasing the stability factors of safety, even as some displacement does occur along the strain-softening geosynthetic interfaces.

3.6 *Measure #6: Increase peak strength.*

If the design basis is to preserve peak strength along a portion of the liner system (e.g., along the base) then specifying materials that increase the peak interface strength will enhance that goal. Examples of methods to accomplish this include more aggressive texturing of geomembranes, and the incorporation of high-friction (granular) soil layers between geosynthetics. Where GCLs are used, enhanced needle punching should be employed to ensure that the internal shear strength of the GCL is not the weak link, and to ensure its strength by a confidence-inducing margin, as discussed in Section 2.13.

3.7 *Measure #7: Reduce weak spots due to variabilities in materials and installation practices.*

This Measure can be considered a direct countermeasure to Mechanism Nos. 14 (material variability) and 15 (installation variability). Tools for the practical implementation of a reduction in the negative effects caused by the variability of material and installation interface strengths are: (a) an understanding of the limitations of manufacturing and installation, (b) strict specifications regarding what is required as related to material manufacturing and installation, and (c) diligent CQA to verify that what is specified is actually provided. There are limits as to what can be provided through manufacturing and installation, which is why it is important for the designer to be very familiar with those limitations.

Section 2.16 directly addressed specific issues related to the variability of geocomposites, which typically present a greater degree of variability in terms of both manufacturing and installation than do other geosynthetic materials, at least in the USA. Possible approaches to address these variabilities include:

- The geocomposite drainage layer could potentially be replaced with a granular drainage layer which has a very dependable internal and interface shear strength. The emphasis of concern would then switch to the potential for damage during construction resulting from the placement of a thin granular layer.
- Geocomposites are typically manufactured with a certain unbonded distance near the edge to allow for seaming by overlap and zip-tying of the geonet cores. The manufacturer could be requested to manufacture the material with the minimum amount of unbonded distance from the edge. If the design does not depend on the transverse-direction transmissivity, then the material could perhaps be manufactured without unbonded edges.

- To compensate for the weak spots caused by the high standard deviation of peel strength associated with geocomposite drainage materials, a higher average peel strength may need to be specified. For products created by heat-bonding, this will usually require a thicker geonet core because of the reduction in transmissivity that goes with obtaining a higher peel strength. It is also good to be aware that a high average peel strength requirement can make it very difficult to peel the geotextile back at the ends of the panels where butt seams need to be performed.

Regardless of the approach taken, a high level of project-specific preparation of the specifications, conformance testing, and CQA construction enforcement is required to reduce the incidence and degree of variability, which the author has found to be a successful approach.

3.8 *Measure #8: Minimize any significant interface damage due to construction or waste filling.*

As discussed in Section 2.9, the main cause of construction damage to the geosynthetic interface is excessive shear stresses and displacements induced by construction equipment. The most common issue in this regard is the shear forces induced by dozer tracks when drivers attempt to push too large of a soil pile upslope, or often worse, downslope. Other conditions of excessive shear stress can occur when construction equipment applies braking forces when moving in a downslope direction. Solutions to these issues require clearly defined enforceable constraints in the specifications that require review by the responsible engineer for all proposed equipment operations on thin (veneer) soil layers being placed on lined slopes. Approaches to the required calculations are presented in Paruvakat and Richardson (1999), Kerkes (1999), Jones et al. (2000), Thiel and Giroud (2023), and Thiel and Giroud (2024).

3.9 *Measure #9: Mitigate foundation settlement.*

To the extent that foundation settlement might introduce additional undesirable relative displacements along the liner system, foundation improvements such as dynamic compaction, preloading, or bridging weak zones to the extent feasible with geosynthetic reinforcement, can help mitigate this issue.

3.10 *Measure #10: High strength reinforcement along sideslopes.*

Insertion of a stiff reinforcement layer within a sideslope liner system to carry a large portion of the tangential stresses, and thus reduce displacements and strains along all the other interfaces, is a well-known design approach. This concept is especially relevant for thin layer cover systems in general, especially during construction (e.g. Koerner and Soong, 1998; Druschel and Underwood, 1993; McKelvey, 1994; and USEPA, 2004), but is also applicable to bottom liner systems (Long et al., 1995; Thiel et al., 2014).

3.11 *Measure #11: Mitigate the potential for high pore pressures.*

Control of, and proper accounting for pore pressures is a fundamental geotechnical design requirement that is not unique to lined containment facilities. What can be particularly problematic for lined containment facilities, though, is the fact that excessive pore pressures generated at the lined interface can cause a localized exceedance of the peak shear strength, which in strain-softening materials can promote a progressive failure mechanism. Such situations can arise due to equipment malfunctions, power failures, failures of backup systems, leachate mounding within the waste mass above fouled or crushed leachate/liquid collection systems, long-term reductions in drainage infrastructure capacities, and the low permeability of high-organic waste leading to waste saturation and localized high gas pressures. Redundancy and robust reduction factors in drainage infrastructure can improve reliability.

Design of bottom liner leachate or solution collection systems should account for long-term clogging mechanisms appropriate for the site-specific hydraulic and chemical loading, drainage layer and pipe layout design, and filtration. A good review of these mechanisms and design approaches to address site-specific issues is presented by Rowe and Yu (2010), and an interesting design case history is presented by Yu and Rowe (2016).

Proper design of reliable lateral drainage layers above and/or below the barrier layers of veneer lining systems, and providing robust drainage outlets, cannot be over emphasized due to the sensitivity of veneer stability to relatively small increases in pore pressures. When designing a lateral drainage layer above a veneer liner system in order to control pore pressures that would be caused by meteoric water infiltration from above, it is advisable to use the unit-gradient technique as recommended by Thiel and Stewart (1993) along with conservative long-term hydraulic conductivity estimates of the cover soil to estimate the amount of water coming into the drainage layer. NRC (2011) suggests that regardless of climate, cover profile, or placement condition, the saturated hydraulic conductivity of most cover soils will increase over time until it is in the range of approximately $8x10^{-8}$ to $6x10^{-6}$ m/s. The typical lateral drainage factor of safety related to pore pressure relief should be greater than 2 after applying appropriate reduction factors (Giroud et al., 2000). When designing a lateral drainage layer below a veneer liner system in order to control gas pressures it is recommended to follow the methods outlined by Thiel (1998). A special need for caution is noted here since veneer slopes are especially vulnerable to failure during an intermediate stage of the construction process when the drainage layer may be exposed to direct precipitation before topsoil placement is completed, making it vulnerable to washout (a type of stability failure).

3.12 *Measure No.12: Increase the residual strength of the interface with the lowest peak.*

The critical interface within a liner system is the one that has the lowest peak strength. Once the peak strength of that interface is exceeded, stability will depend upon the residual strength of that particular interface. It is common for designs to introduce a weak interface, sometimes referred to as a 'fuse layer', so that the location of slippage can be controlled above the critical containment geosynthetics in such a way as to avoid damaging the containment integrity of the liner system. This concept was mentioned in Section 2.13 with regard to protecting the internal shear strength of the GCL, as recommended by Marr and Christopher (2003). Within limits, certain geosynthetic materials can potentially be selected for the 'fuse layer' that will have higher residual shear strengths than other options. While this is an excellent goal to pursue, it may be difficult to achieve with reliability and precision, especially considering manufacturing variability. Brown et al. (1999) describes a case history for a California steep sideslope canyon landfill where selection of the geosynthetic interfaces was carefully tested and specified in an attempt to 'dial in' an optimal combination of (a) protection of the critical containment geosynthetics against an integrity failure via the selection of a reliable 'fuse layer', and (b) maintaining as high a degree of a residual strength as possible along the 'fuse layer.'

3.13 *Measure No.13: Adaptive management; Long-term instrumentation and monitoring.*

Considering that intensive use of geosynthetics has only occurred since the 1980s, we could say that we really don't have enough data on the long-term performance of geosynthetics used in liner systems to make accurate estimates of performance regarding ageing and creep of geosynthetics, considering that structures such as landfills may need to remain stable for hundreds of years (i.e., until they become inert). This situation mandates either incorporating extra conservatism in design decisions, or long-term monitoring.

A design that includes reliable methods of monitoring the system performance, and adapting fill plan operations to the results of instrumented feedback, could be a component of risk management that supports a slope stability design basis. The ideal is that real-time monitoring can aid in addressing uncertainties in the analysis by allowing for a comparison between the system's actual performance and its predicted performance, thus allowing for corrective measures if needed.

An excellent review of the approaches, value, and technology for instrumentation and monitoring of slope stability is provided by Marr (2013). Advances in communication and data management technology over the past 30 years have made real-time monitoring and data evaluation a practical reality that is within reach of projects of all sizes. Instrumentation can be used to monitor performance during all phases of a project, including construction, operations, and post-closure.

Instrumentation that could be considered include various types of inclinometers and extensometers to keep track of settlement and lateral movements along a vertical profile; survey monuments to keep track of settlement and lateral movements on the surface; piezometers and pressure transducers to keep track of fluid pressures at specified point locations; pressure cells to keep track of actual normal pressures at various locations; temperature sensors installed during construction or in boreholes, pipes, wells, or sumps; different types of strain or deformation gauges to measure elongation, contraction, or relative movement (i.e. slippage) at an interface; load cells to measure total or effective normal stress; and accelerometers to measure dynamic forces from equipment, blasting, or earthquakes. See Marr (2013) for further discussion.

The measurement of strains or deformations along a liner system are possible in concept (e.g. Yazdani et al., 1995; Daniel and Scranton, 1996; Villard et al., 1999; Fowmes, 2007; Zamara et al., 2014), but the long-term viability of instrumentation of this type, especially at significant distances or depths, has not been fully proven along liner system interfaces. Though a handful of field-scale instrumented studies have been performed, and the results of numerical analyses have compared favorably with limited observed failures, the validation of numerical models via a comparison with the results of field measurements is needed (Fowmes, 2007; Kavazanjian et al., 2018). The reporting of full-scale case studies of such monitoring would be a great contribution to the profession.

In concert with any monitoring plan there should be a response-action plan. For example, if cracks are observed near slope crests, or lateral spreading of the waste toe is observed, or tension-thinning of a geomembrane at a slope crest is observed, etc., what should be done? A typical hierarchy of responses could include, for example, immediate increased/expanded monitoring of movements with survey points and inclinometers, engagement of qualified geotechnical professionals to assess the situation, cessation of any continued slope loading or toe excavation, aggressive removal of any sources of pore pressures (liquid or gas), consideration of toe buttressing with earthworks, and consideration of crest unloading. One of the greatest lessons learned in past failures as related to the designer's limitation of liability is to provide operational plans to owners as part of their scope of work. Important elements of such plans, as related to the subject of the current paper, would be fill sequencing plans, periodic inspections and monitoring, and basic response-action plans.

4 DISCUSSION OF RISK AND OTHER CONSIDERATIONS IN THE USE OF PEAK STRENGTH

Duncan (1996) writes that the only fully reliable design in the presence of a strain-softening interface is the use of residual strengths. Admittedly, there are situations where designers believe that the geometry, high peak shear strengths, control of pore pressures, and lack of a significant seismic threat should allow peak strengths to be used. As described by Baecher (2023) geotechnical uncertainties are generally epistemic in nature, meaning that selected parameters and approaches are often subjectively based on our experience, and invoke what is called 'engineering judgement'. Designers should be sufficiently experienced and qualified to make such judgements and should provide well documented justification for their decisions. Such determinations should also include consideration of the level of risk and the consequences of failure. A discussion of risk, reliability, and consequences as relates to this paper is provided in the *Geosynthetics International* companion paper appendices.

4.1 *Designs for bottom liners based on peak strength.*

Recognizing that all designs will have higher reliability when using LD or residual shear strengths, there are possible project-specific scenarios in which the use of appropriately modified peak strengths is viable. In such cases the following considerations might be taken into account for bottom liner systems:

1. The first element that should be considered is the geometry and the presence of slopes. Industry field experience, and numerical analyses, have confirmed that slope inclinations

approaching and exceeding 3(H):1(V) have a high susceptibility to experiencing displacements caused by the non-uniform mobilization of shear stresses and settlement of the overlying fill, and the resulting stress and strain distributions on the slopes can also affect the flatter areas. Influential in this regard will be the relative stiffness of the contained waste or mining ore, and its long-term compressibility and settlement. Estimations of the propensity for displacements to cause exceedance of peak strength can be approximated using examples of numerical analyses previously published in the literature. Otherwise, project-specific numerical analyses can be performed.

2. If pore pressures or seismic factors have significant potential to cause liner displacements, then residual strengths should be used.
3. In all cases where 'peak' strengths are considered they should be appropriately modified or adjusted to account for construction impacts, spatial variabilities due materials manufacturing and installation, ageing, and any other factors deemed relevant.
4. The designer should consider preparing an operational plan that illustrates safe parameters for the fill sequencing, maximum fill limits, and perimeter buttressing.
5. An adaptive approach to management can be implemented through the use of instrumentation and monitoring. Such a program is only useful in conjunction with a reliable response action plan, and when early warning signs are not ignored.
6. Designing around an appropriately adjusted peak strength for the early phases of a project life may be feasible. Settlement strain values during this period will be less than the long-term post-closure values. Thus, while the stability analysis of the final geometry of a filled facility can be based on rule-based approaches such as those recommended by Stark and Choi (2004), it may be legitimate to count on peak strength, with caution, during certain early operations of facilities when a well-engineered filling plan is provided and followed.

4.2 *Designs for veneer liner systems based on peak strength.*

Stark and Choi (2004) present recommendations that landfill cover systems (which represent a major category of veneer liner systems) can be designed using the peak strength of the weakest interface with a factor of safety greater than 1.5. Stark and Choi (2004) mention three situations in which residual shear strength with a factor of safety greater than 1.0 should be considered: (1) if the slope angle of the final cover system is greater than the peak strength of the weakest interface, (2) if large construction-induced displacements are expected, and (3) if seismically induced displacements can be expected. Stark and Choi (2004) do not describe how the peak strength would be measured and evaluated. A standard design practice is to obtain manufactured samples for laboratory shear strength testing in order to determine the peak interface strengths. Those results are then commonly used in the slope stability analyses without further modification. For the reasons described previously, the present paper recommends that the measured peak strength of any veneer system interface should be modified on a project-specific basis from the test results normally obtained in the laboratory. While thoughtful specifications and a high-level of CQA can help mitigate the need for some conservatism, there are other factors that require engineering judgement related to peak shear strength adjustments from laboratory-measured values.

Based on the above discussion, the present paper recommends that, while the use of 'peak' strength can be appropriate for veneer lining systems, the peak strength be selected such that it accounts for conditions that include variabilities in materials and installation practices, construction damage, seismic displacements, foundation settlement, potential effects of long-term ageing and creep, and the potential for any of these factors to cause stress concentrations at the boundaries of a change shear strength (e.g. going from an unbonded to a bonded condition of a geotextile lamination to a geonet.) Having considered these possible adjustments, the design factor of safety is typically recommended to be greater than 1.5 to account for geotechnical variabilities, unknowns, and simplifying assumptions, as is standard in the geotechnical profession.

5 CONCLUSIONS AND RECOMMENDATIONS

The present paper has identified that:

1. Limit equilibrium stability analyses are based on the fictitious assumption that the soil or waste blocks above a liner system function as rigid bodies. This could result in an unrealistic uniform variation of shear stress mobilization and FS along the critical surface that could be highly non-conservative for strain-softening interfaces if the design is based on peak strengths.
2. Limit equilibrium stability analyses provide no feedback on the amount of displacement that may occur. Thus, the available shear strength of strain-softening materials in the field is unknown.
3. Limit equilibrium analyses are the most common type of slope stability analyses used in general.
4. Most geosynthetic interfaces are strain-softening.
5. Numerical continuum modeling can roughly predict the non-uniform distribution of mobilized shear stress and displacements along the critical surface, considering strain-softening behavior.
6. Only a handful of numerical modeling studies of geosynthetic lined containment facilities have been published. The use of numerical analyses in practice is relatively limited due to the time they require and the expense of performing them, although their use is becoming slightly more prevalent as time goes on.
7. The number of bottom liner failures that have occurred in the containment industry over the past 35 years that can be attributed to progressive failure along strain-softening geosynthetic interfaces is relatively small. They have definitely occurred though, and when they have, they have been large, costly, and consequential. The failures that have occurred represent conditions and circumstances ranging from base liners to sideslopes, and from smooth interfaces to textured.
8. For containment facilities containing strain-softening interfaces there are potentially: 15 Mechanisms that could promote displacement that could lead to progressive failure initiation and propagation in bottom liner systems, and 9 such Mechanisms in veneer systems; and 13 Measures that can be taken to reduce or mitigate the development of displacements in bottom liner systems, and 10 such Measures in veneer systems.
9. For bottom liner systems the likelihood that significant interface displacements will be experienced is project-specific, and dependent upon the complex interaction of all of the considerations discussed in the present paper. A responsible evaluation can be made based on a combination of numerical analyses, engineering judgement that is based on a review of case histories and inductive reasoning, sensitivity studies, reliability analyses, and heedfulness of the Mechanisms and Measures described in Sections 2 and 3.
10. Bottom liner systems with sideslopes approaching and steeper than 3(H):1(V) have a high probability of experiencing significant interface displacements that will lead to strain softening. Inclusion of a sacrificial slip layer above the critical containment liner element on slopes, with the assumption of residual strength along this interface, is a common design remedy that could be considered to protect the liner's integrity.
11. It may be legitimate to count on appropriately adjusted peak strength, with caution, during early operations of bottom liner facilities as long as well-engineered construction and filling plans are prepared with good construction monitoring.
12. The state of our understanding of the specific causation and propagation of progressive failure along strain-softening geosynthetic interfaces is, overall, in a semi-quantitative and semi-empirical phase. It is simply not easy to accurately deduce the mobilized stresses and displacements, and the available shear strength, in the field at all points along a strain-softening bottom-liner interface. Detailed numerical continuum analyses can be of great help in this regard. And even with such analyses, there are numerous stress concentration possibilities (localized variabilities in materials, installation, or construction damage) that have not been captured by, or incorporated into, such models to date. Also, many of the factors that potentially affect displacement along interfaces are not fully understood. Though a handful of field-scale instrumented studies have been performed, and

the results of numerical analyses have compared favorably with the limited observed failures, we still await validation of numerical models as compared with field measurements (Fowmes, 2007; Kavazanjian et al., 2018).

13. Most slope stability designs are driven by rules that were formulated by a combination of inductively obtained conclusions from past failures, along with approximate engineering models for slope stability. Forensic studies have attempted to apply numerical analyses in a deductive manner with the goal of determining the root cause of slippages, but there are significant nuances to these dynamics that have yet to be completely modeled.
14. Over-reliance on a small number of laboratory shear and conformance testing results, and ignorance of variabilities in the installation process, remain a large area of concern.
15. The factor of pore pressures has been mentioned in the literature as one of the contributors to many of the documented (and undocumented) bottom liner and veneer system failures in the containment industry. Some of the cases reported pore pressures due to head buildup above the liner, and some due to saturated non-consolidated clays or gas pressures below the liner.
16. If the use of peak strength values is relevant to the slope stability design, the basis of the definition of peak strength should be documented. It may not be appropriate to adopt the peak strength results measured from a factory sample that is tested in the laboratory. Modifications to the peak strength should be considered based on the several factors described in this paper.
17. It is the responsibility of the design engineer to communicate relative degrees of risk concerning slope stability to owners so that owners can make informed decisions.
18. Simple reliability analyses are helpful because they can highlight where small variations in assumptions can have a significant impact on the probability of failure.
19. If a designer follows the rule-based analysis of Stark and Choi (2004) for bottom liner systems, the design will intrinsically be substantially safe.

Based on these findings the following recommendations are suggested:

1. Designers should be concerned about activities that may affect strain-softening interfaces after they issue a design for construction. This would include the construction, operational, and post-closure phases. They should also consider the possibility of changes in project ownership. Designers should convey their expectations clearly in the project documentation, which would include the construction specifications, construction inspection requirements, facility operations, and post-closure monitoring expectations.
2. Project-specific testing is recommended in order to determine the peak and LD (or residual) shear strengths that are representative of the actual materials being used for construction and representative of field conditions (e.g. spraying all interfaces being tested with water during the shear test assembly process to mimic the condensation that occurs in the field, in addition to being flooded during testing).
3. In the evaluation of slopes that involve strain-softening interfaces, which includes most lined containment facilities, limit equilibrium analyses of deep-seated failure surfaces (e.g., bottom liners) should be overseen by trained and experienced geotechnical professionals who have studied the principles described in the present paper and can exercise appropriate engineering judgement.
4. When peak strengths are being assumed in stability analyses of bottom liner or veneer lining systems, consideration should be given to modification of the peak strengths to account for variabilities in materials and installation processes, construction damage, seismic displacements, foundation settlement, potential effects of long-term ageing and creep, and the potential for any of these factors to cause stress concentrations at the boundaries of a change in shear strength, such as can be found in the transition from an unbonded to a bonded condition of a geotextile lamination to a geonet. Following these adjustments, the design factor of safety should be greater than 1.5 to account for geotechnical variabilities, unknowns, and simplifying assumptions, as is standard in the geotechnical profession.
5. It is recommended to give special attention to specifications for the bonding between geotextiles and geonets when using drainage geocomposites. Where slope stability is critical, specifications for these materials should be written that require minimal widths

of unbonded edge zones, and higher average peel strengths to compensate for the very high standard deviations in manufacturing that seem endemic to these manufactured products. In addition, increased conformance (verification) testing frequency of these parameters would be advised, requiring that more specimens be tested across the panel width than typically suggested by ASTM D7005.

6. Sensitivity and probabilistic studies can provide insight into determining which elements of the design are the most critical so that design efforts can be focused. An evaluation of the failure risk, especially as regards potential consequences, should be considered as part of this type of evaluation.
7. A degree of uncertainty can be addressed by implementing a long-term program of instrumentation and monitoring, combined with a response action plan.
8. When considering the use of peak strengths, the consequences of failure should be weighed against the uncertainties in the design.
9. Where designers wish to minimize uncertainties and follow a safe defensible standard practice for bottom liner static stability without the use of sophisticated numerical analyses, the rule-based recommendations of Stark and Choi (2004) are probably the most pragmatic and straightforward. This is the case because they address regulatory concerns (FS > 1.5), as well as the work of Gilbert and Byrne (1996), which seeks to achieve FS > 1.0 under residual strength conditions. The Stark and Choi (2004) rules can be summarized as follows:
    - For landfill bottom liners, assign residual shear strengths to the sideslopes, peak shear strengths to the base of the liner system, and satisfy a factor of safety greater than 1.5.
    - Assign residual strengths to the sideslopes and base of the liner system, and satisfy a factor of safety greater than unity.
10. When performing calculations to determine if seismic deformations due to the design earthquake will be within acceptable limits (as defined by standard practice or the regulations), LD or residual strengths should be assumed along the entire critical interface for purposes of those calculations, even if peak strengths have been determined to be acceptable all parts or all of the critical interface for purposes of the static stability. The design should then be checked to meet other project-specific standards or regulatory requirements separately for static FS, and for the maximum estimated displacement due to the design earthquake.
11. Designers should attempt to position the critical slip plane above the primary geomembrane to the extent feasible for a given project.
12. The stability of veneer liner systems can be based on peak strength, but consideration should be given to modify the peak strength to take into account the factors described in Section 4.
13. Recommendations for future studies include: (1) the effects of regularly or irregularly spaced weak zones in the plane of geosynthetic strain-softening interfaces, such as those created by commonly found manufacturing weaknesses or installation seaming practices, and how those might initiate interface displacements that could contribute to progressive failure; (2) the potential for weak zones to act as stress concentrators within the plane of the interface, and thus enhance the tendency for progressive interface displacement initiation and propagation; and (3) the long-term ageing, durability, and creep performance of geosynthetic interface shear strength.

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