Technical Note

Peak versus residual interface strengths for landfill liner and cover design

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ABSTRACT: The rationale for using peak, residual, or a combination of these shear strengths for the analysis of geosynthetic-lined slopes and design recommendations for landfill liner and cover systems is presented herein. Landfill liner systems using geosynthetics that contain sideslopes are recommended to be designed using the methodology presented by Stark and Poeppel: (1) assign residual shear strengths to the sideslopes and peak shear strengths to the base of the liner system and satisfy a factor of safety greater than 1.5; and also (2) assign residual strengths to the sideslopes and base of the liner system and satisfy a factor of safety greater than unity. The authors recommend that the stability of landfill cover systems be analysed using peak shear strengths with a factor of safety greater than 1.5 because of the absence of large detrimental shear displacement along the weakest interface. If, for some reason, the slope angle of the cover system exceeds the friction angle of the weakest interface, or large displacements such as construction-induced displacements or seismically induced displacements are expected, a residual shear strength with a factor of safety greater than unity should be used for the cover design. In both liner and cover designs a peak composite failure envelope that describes the weakest interface should be used to represent the peak shear strength, and the residual failure that corresponds to the peak composite failure envelope should be used instead of the lowest residual failure envelope.

KEYWORDS: Geosynthetics, Interface shear resistance, Design, Direct shear test, Ring shear test, Shear strength, Slope stability

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1. INTRODUCTION

The main objectives of this manuscript are to clarify the recommendations for the design of geosynthetic-lined landfill liner slopes presented by Stark and Poeppel (1994) and to present new recommendations for the design of landfill cover systems. This discussion is limited to slope instability that might occur along a soil–geosynthetic or geosynthetic–geosynthetic interface. This discussion does not relate to possible slope instability that may develop in soils underlying a waste containment facility or through the waste materials.

The selection of the interface shear strength that should be used for design of the liner and cover system is important because it affects the disposal capacity of a waste containment facility. The usual design objective for waste containment facilities is to maximise storage capacity. Thus sideslopes are designed and constructed as steeply as possible, and the waste height and slope will be as high and steep as possible, respectively. Many researchers (e.g. Martin *et al.* 1984; Saxena and Wong 1984; Koerner *et al.* 1986; Williams and Houlihan 1987; Negussey *et al.* 1989; Bove 1990; Mitchell *et al.* 1990; O'Rourke *et al.* 1990; Takasumi *et al.* 1991; Yegian and Lahlaf 1992; Stark and Poeppel 1994; Stark *et al.* 1996; Dove and Frost 1999) have shown that the residual interface shear resistance can be as much as 50–60% lower than the peak interface shear resistance. Thus use of a residual strength in design results in substantially flatter slopes, smaller disposal capacity, and decreased revenue. However, a number of case histories (Seed *et al.* 1990; Seed and Boulanger 1991; Byrne *et al.* 1992; Stark 1999) show that an overestimate of the geosynthetic–geosynthetic interface shear resistance can lead to slope instability and substantial remediation costs.

2. DESIGN OF LANDFILL LINER SYSTEMS

2.1. General

Stark and Poeppel (1994) present a design approach that uses a combination of the peak and residual for the design of landfill liner systems. This recommendation is based on the interface testing for and back-analysis of the slope instability in the Kettleman Hills Hazardous Waste Facility. Stark and Poeppel (1994) conclude that two design scenarios should be considered in stability analyses of geosynthetic liner systems. This recommendation has been verified with other case histories (e.g. Stark *et al.* 1998, 2001).

The first design scenario uses the peak interface shear resistance along the base or bottom of the landfill liner system and the residual interface shear resistance along the sideslope(s) of the liner system and satisfying a 2-D factor of safety of at least 1.5 for the final slope configuration, at least 1.3 for interim slopes, and 1.1 to 1.3 depending on the design seismic event. The second scenario involves ensuring that the 2-D factor of safety exceeds unity when the appropriate (discussed subsequently) residual interface shear resistance is applied to the base and sideslopes of the liner system. The second design scenario is considered because the peak interface strength is usually mobilised at a small laboratory displacement (Stark *et al.* 1996). Because of the uncertainty of the relationship between laboratory shear displacements and field shear displacements, the effect of progressive failure, and possible shear displacement caused by earthquake shaking, this scenario should be carefully considered. In other words, if everything goes wrong, i.e. a residual interface strength is mobilised along the weakest interface, the slope should remain stable because the 2-D factor of safety is greater than unity. If the residual interface strength is measured in a direct shear apparatus, a factor of safety greater than unity, e.g. 1.1, should be considered to compensate for the limited continuous shear displacement applied in this apparatus (Stark and Poeppel 1994; Marr and Christopher 2003).

There are uncertainties surrounding the application of these design scenarios. The main uncertainties are related to determining the residual shear resistance that should be used for the sideslopes and whether or not this recommendation is applicable to the design of the cover system. This manuscript is focused on clarifying the authors' opinion with regard to the use and applicability of these design scenarios in the design of landfill geosynthetic-lined slopes. Thiel (2001) correctly limits this recommendation to the design of liner systems, which is reflected in the title of his paper. Stark and Poeppel (1994) consider only the liner system at the Kettleman Hills Facility and other liner systems and not a final cover system.

Stark and Poeppel (1994) conclude that a residual interface shear resistance is mobilised along the sideslopes of liner systems, and the critical interface on the sideslope can differ from the base of the liner. The residual strength can be mobilised for many reasons including waste settlement or creep that leads to shear displacements along specific interfaces (Long *et al.* 1995), waste placement activities (Yazdani *et al.* 1995), lateral movement or bulging of waste (Stark *et al.* 2000), construction activity of the liner system (McKelvey 1994), thermal expansion/contraction of the geosynthetics, stress transfer between the waste on the sideslope and the landfill base that is acting as a buttress (Stark and Poeppel 1994), strain or displacement incompatibility between the waste and geosynthetic interface of interest (Eid *et al.* 2000), and/or earthquake-induced displacements. These shear displacements may lead to mobilisation of a residual strength, which can result in progressive failure effects between the sideslope and at least a portion of the base of a bottom liner system (Byrne 1994; Stark and Poeppel 1994; Gilbert and Byrne 1996; Reddy *et al.* 1996; Filz *et al.* 2001). Additional evidence of these shear displacement mechanisms has been developed since 1994 and is presented in the following sections to reinforce the recommendations in Stark and Poeppel (1994).

2.2. Development of residual interface strength condition

A residual interface shear resistance will develop in the field only if detrimental shear displacement occurs along a geosynthetic interface in the liner system. The two important factors in the above statement are: (1) detrimental or damaging shear displacement, and (2) the interface along which this detrimental shear displacement will occur. Detrimental shear displacement means that the interface shear resistance is being reduced from the peak value because shear displacement is occurring.

The two main areas for slope instability in the cross-section shown in Figure 1 are a slide mass near the slope face, i.e. toe area, and the entire waste mass sliding along a failure surface that extends along the base of the landfill and up the sideslope. The stability of the slope face area is controlled by the interface in the base liner system exhibiting the lowest peak strength and the waste strength, and is independent of the sideslope.

The stability of the entire waste mass sliding along a failure surface that extends along the base of the landfill and up the sideslope is the focus of the design recommendation of using a residual interface strength on the sideslope. The driving force or force causing instability in this scenario is the triangle of waste above the sideslope of the landfill. The stability of this triangle of waste is controlled by the interface shear resistance mobilised along the sideslope and base of the landfill. The majority of the shear resistance in this failure mode is derived from the base of the landfill because the

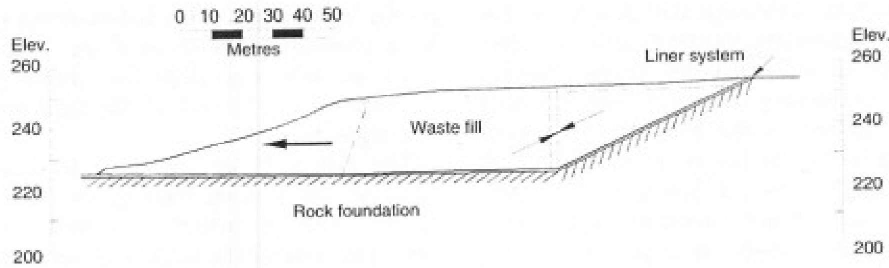

Figure 1. Schematic diagram illustrating mobilisation of buttressing effect of waste on the base of the landfill

normal stress is greatest along the base, and the failure surface is longer assuming that the same interfaces appear on the sideslope and the base of the landfill. The interface shear resistance along the base is given by $\sigma'_n \tan \delta_p$, where δ_p is the peak interface fiction angle of the weakest interface and σ'_n is the effective normal stress acting on this interface. Thus the practice of installing smooth HDPE geomembrane on the base and textured HDPE on the sideslope for value engineering and drainage layer stability purposes may have detrimental effects on stability because the smooth HDPE geomembrane will exhibit a smaller interface strength than textured HDPE.

Because of the low shear resistance exhibited by geosynthetic interfaces, the triangle of waste in Figure 1 must mobilise some shear resistance along the base of the landfill to prevent instability. The shear resistance of geosynthetic interfaces along the sideslope is low because of the low σ'_n and δ_p along the sideslope. This results in shear displacement along the weakest interface in the sideslope liner system, mobilising the passive resistance of the MSW along the base of the landfill. This stress transfer mechanism is especially relevant to MSW because of the compressible nature of MSW. If the base of the landfill were filled with an incompressible material, such as concrete, the shear displacement required to mobilise the shear resistance along the base of the landfill would be smaller. However, the compressible nature of MSW results in significant deformation being required to mobilise the shear resistance along the base of the landfill, especially at the sideslope/base transition. This stress transfer phenomenon has been duplicated using numerical methods by Byrne (1994), Gilbert and Byrne (1996), and Reddy *et al.* (1996).

Byrne (1994) was the first to use numerical methods to depict the behaviour of a liner system in response to waste placement to investigate the shear strength mobilised along the base and sideslope for the Kettleman Hills slope failure. Byrne (1994) uses the finite difference computer code FLAC (Cundall 1976) to recreate the filling process and shear strength mobilised in the base and sideslope of the liner system at the Kettleman Hills facility. The initial analysis involves placement of waste to a depth that is 3 m lower than the depth at failure. The second stage corresponds to the waste depth at failure of about 30 m. The results of the first stage of waste placement indicate a stable condition, but a residual strength condition is mobilised along the sideslope and a post-peak shear strength condition is mobilised along the initial portion (about 20%) of the base of the landfill in the vicinity of the sideslope. Over the remaining 80% of the landfill base, the induced shear stress is resisted by 60% of the peak shear strength.

After placement of the second stage of waste placement, i.e. waste depth at failure, failure along the liner system is imminent. A residual strength condition is mobilised along the sideslope, and the zone of post-peak shear strength along the base of the landfill now extends about 40% of the length of the base from the sideslope. Over the remaining 60% of the landfill base the shear stress is resisted by about 90% of the available peak shear strength. Placement of another 1 m of waste is sufficient to cause slope instability (Byrne 1994).

Subsequent finite element analyses of the Kettleman Hills slope failure (e.g. Reddy *et al.* 1996; Filz *et al.* 2001) indicate similar conclusions as those reached by Byrne (1994). These conclusions are that the shear resistances mobilised along the base and sideslope of the landfill are not equal, and the use of a peak smooth geomembrane–clay interface shear resistance along the entire failure surface does not predict the failure. More importantly, use of a peak smooth geomembrane–clay interface strength overpredicts the mobilised strength, and thus a combination of peak and residual strengths should be used in 2-D limit equilibrium methods.

Progressive failure occurs in slopes in which the driving force exceeds the mobilised strength of the weakest layer, e.g. the slope angle exceeds the friction angle of the weak layer (Mesri and Shahien 2003). If this occurs, the interface at the location where the driving force exceeds the interface friction angle becomes overstressed. If this local overstressing is great enough that the interface yields and shear displacement occurs, the shear stresses applied to this location are transferred to the interface element adjacent to this overstressing because the interface is undergoing a post-peak strength loss and cannot restrain the imposed shear stresses. If the existing shear stresses and the transferred shear stresses are great enough to cause the adjacent portion of the interface to yield, the overstressing will be transferred further. This process can continue until enough of the interface is overstressed that a slope failure occurs. If the shear strength of the weakest interface increases sufficiently, the initial overstressing can be arrested and slope failure is averted. Thus the fact that a limited portion of the interface achieves a post-peak condition does not mean the entire slope should be designed using residual interface strength. Byrne (1994) shows a residual strength condition developing along the entire sideslope and transferring stresses to the base of the landfill. Gilbert and Byrne (1996), Reddy *et al.* (1996) and Filz *et al.* (2001) also suggest the possibility of progressive failure occurring along a liner interface, and thus a residual or post-peak strength, respectively, may be applicable.

In summary, these analyses support the conclusion that a residual interface strength can be mobilised along a landfill sideslope while a peak interface strength is mobilised along the base.

2.3. Composite failure envelope design for bottom liner system

The interface along which detrimental shear displacement may occur is the interface that exhibits the lowest peak interface shear resistance in the bottom liner system regardless of the value of the residual interface shear resistance. For example, if the interface with the lowest peak interface shear resistance exhibits the highest residual interface shear resistance, the detrimental shear displacement may still occur along this interface but the resulting stability will be controlled by the residual interface shear resistance along this interface and not the lowest residual interface strength, e.g. a GCL. The reason for not mobilising the lowest residual interface shear resistance is that detrimental shear displacement will not occur along an interface with a higher peak strength before movement is initiated along the interface with the lowest peak interface strength. If detrimental shear displacement does not initiate along an interface, the shear resistance cannot drop to the residual value. In other words, there is no evidence that an interface can somehow end up at a residual strength condition if it is not subjected to detrimental shear displacement.

The failure envelope that corresponds to the lowest peak interface strength may correspond to the strength of one or more interfaces because geosynthetic interface strength is stress-dependent and non-linear (Stark and Poeppel 1994; Stark *et al.* 1996; Fox *et al.* 1998; Dove and Frost 1999). If more than one interface is used to develop the failure envelope for the interface with the lowest peak strength, the failure envelope is referred to as a composite failure envelope. The selection of a composite failure envelope for a multi-layer liner system is discussed at the end of the liner and cover system discussion.

The proper failure envelope for use in the design scenarios for bottom liner systems is reviewed in this section. This procedure is primarily for liner system design because of the large range in normal stress along the liner system. However, this procedure can be used for a cover system too. The range of normal stress is usually small in a cover system, i.e. 2.5–20.0 kPa, so the weakest peak interface strength usually does not change over this range in normal stress. However, if the weakest peak interface strength does change over this small normal stress range, a composite failure envelope should be developed for cover design purposes using the procedure described below.

The procedure for constructing a peak composite failure envelope uses the following three steps:

1. Determine the interface(s) or material(s) in the composite liner system that exhibit(s) the lowest peak strength for the full range of normal stresses encountered along the bottom liner system.
2. Determine the peak composite failure envelope for the weakest interface(s) or material(s) in the composite liner system for the full range of effective normal stresses encountered along the liner system.
3. Determine the residual composite failure envelope that corresponds to the peak composite failure envelope in Step 2.

The resulting peak and residual composite failure envelopes are used in the two design scenarios presented by Stark and Poeppel (1994) and discussed in Section 2.1. An example of developing a peak composite failure envelope is presented in Figures 2–5. Figure 2 presents the peak failure envelopes for the following interfaces measured using a torsional ring shear device (Stark and Poeppel 1994):

- nonwoven geotextile–smooth HDPE geomembrane (GM);
- clay–smooth GM; and
- geonet–smooth GM.

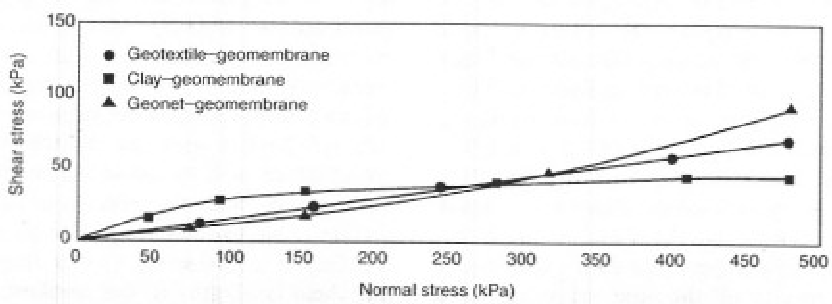

Figure 2. Peak failure envelopes for three components of the composite liner system at Kettleman Hills Waste Repository (Stark and Poeppel 1994)

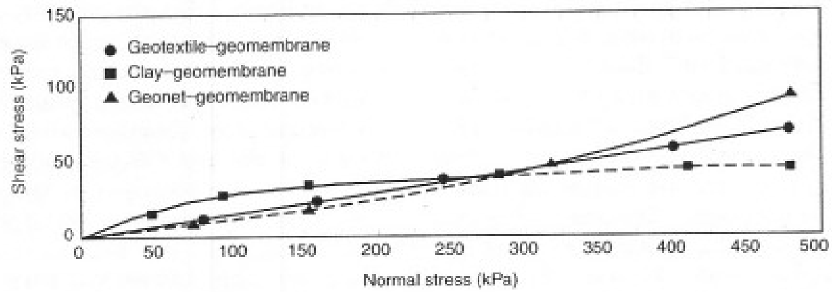

Figure 3. Peak composite failure envelope for three components of the composite base liner system at Kettleman Hills Waste Repository

For $\sigma'_n \leq 280$ kPa, the geonet–smooth GM interface exhibits the lowest peak shear strength and is the critical or weakest peak interface strength. However, the clay–smooth GM interface is critical for $\sigma'_n > 280$ kPa. Therefore a composite failure envelope, illustrated by the dashed line in Figure 3, should be used to represent the peak interface strength of the liner system. In other words, the peak composite failure envelope represents the weakest composite interface, and this shear displacement will occur along this composite interface before some other interfaces. Therefore this composite interface is the interface along which a residual strength condition could develop.

Figure 4 shows the individual residual strength failure envelopes for the same liner interfaces shown in Figure 2, and Figure 5 shows the design residual failure envelope (dashed) for the liner system. The design residual failure envelope corresponds to the peak composite failure envelope and does not simply represent the lowest residual composite failure envelope. The geotextile–smooth GM interface exhibits the lowest residual shear strength, but this residual envelope is not used for design because the peak strength of the geotextile–smooth GM interface will not be exceeded (see Figure 2) before the peak composite failure envelope is exceeded. Thus a residual strength condition will not be mobilised along the geotextile–smooth GM interface because detrimental shear displacement will occur on the geonet–GM and/or the clay–GM interface before it occurs on the geotextile–GM interface. Thus the residual composite failure envelope is between the highest and lowest residual failure envelopes.

In this example, there is not a large difference between the peak failure envelope of the geotextile–GM and geonet–GM interfaces at $\sigma'_n \leq 280$ kPa, so it may be prudent in this case to design for both of these interfaces at $\sigma'_n \leq 280$ kPa, which would involve checking to ensure the factor of safety is also greater than unity if the residual failure of the geotextile–GM interface is used for $\sigma'_n \leq 280$ kPa.

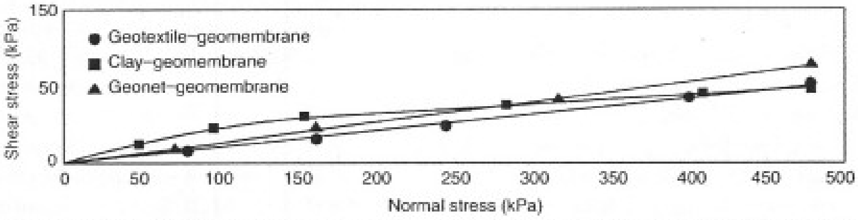

Figure 4. Residual failure envelopes for three components of the composite base liner system at Kettleman Hills Waste Repository (Stark and Poeppel 1994)

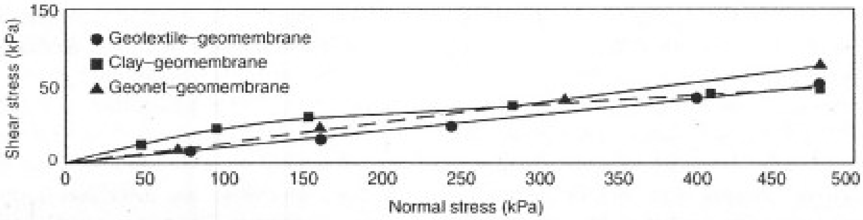

Figure 5. Design residual failure envelope for three components of the composite base liner system at Kettleman Hills Waste Repository

The proper selection of a composite failure envelope for design is especially important when a bottom liner system contains a reinforced GCL because of the high peak strength and low residual strength of hydrated bentonite (Stark and Eid 1996; Fox *et al.* 1998). With a reinforced GCL, other interfaces that exhibit a large post-peak strength loss, e.g. nonwoven geotextile–textured GM or double-sided drainage composite-textured GM, should also be evaluated to develop a representative composite failure envelope. The residual failure envelope for any hydrated GCL will plot well below the geotextile–smooth GM residual failure envelope in Figure 4 because it corresponds to the shear strength of hydrated bentonite. However, the peak strength envelope for an encapsulated unreinforced GCL and a reinforced GCL is likely to be significantly higher than that for many other typical interfaces in the liner system. If so, the GCL peak shear strength will not be exceeded, and the GCL will not reach a residual shear condition. Thus the GCL internal residual envelope should not be used for the sideslopes (design steps 1 and 2) or the base liner (design step 2). Use of the GCL internal residual failure envelope under these conditions would be unnecessarily conservative. This emphasises that the main design issue surrounding GCLs is not shear strength but hydraulic equivalence (Stark *et al.* 2004).

In summary, designers should not simply use the minimum residual failure envelope for design, but should determine which materials will reach a residual shear condition and then use the corresponding residual composite failure envelope for design. This is accomplished by first establishing the minimum peak composite failure envelope.

3. DESIGN OF LANDFILL COVER SYSTEMS

The proper methodology for selection of the design failure envelope for a cover system differs from the liner system design because of differences in the expected detrimental shear displacements. In particular, the design scenarios presented by Stark and Poeppel (1994) are not applicable to cover systems. Unpublished two- and three-dimensional back-analyses of cover failure studies by the first author show that peak interface strengths are mobilised throughout a cover system. This results for a number of reasons, including the presence of low shear stresses, low normal stresses (which limit detrimental, i.e. damage-inducing, shear displacements to a geosynthetic interface), smaller shear displacements required for stress transfer in soil cover than in MSW, and smaller settlements of the compacted soil veneer as compared with MSW. Although there is an opportunity for considerable construction-induced shear displacements to occur in cover systems, these displacements can be minimised by placing cover materials from bottom to the top of the sideslopes or by including tensile reinforcement (Koerner and Soong 1998). Therefore it is recommended that the stability of cover systems be analysed using the peak shear strength of the weakest interface, or if applicable the weakest composite interface, with a factor of safety greater than 1.5.

There are some situations where a residual interface shear resistance with a factor of safety greater than unity should be used in cover system design. If the slope angle of the final cover system is greater than a peak interface shear strength of the weakest interface, progressive failure can occur (Gilbert and Byrne 1996). As denoted previously, progressive failure occurs in slopes in which the driving force exceeds the mobilised strength of the weak layer, i.e. the slope angle exceeds the friction angle of the weak layer. Also, when large displacements such as construction-induced displacements or seismically induced displacements can be expected, the use of residual shear strength is recommended.

Thus, if the average slope angle of the cover system is greater than the lowest peak interface friction angle, a residual interface friction angle should be used for design. However, cover systems reinforced with tensile members can limit the progressive displacement on the weakest layer, and thus a residual interface shear strength will not fully mobilise. In such a case, the stability of a cover system can be analysed using the peak shear strength of the weakest interface with the factor of safety greater than 1.5.

4. RECOMMENDATIONS

The following recommendations can be discerned from information presented in this paper.

1. Detrimental, or damaging, shear displacement may occur within geosynthetic-lined landfill liner sideslopes owing to construction activities, thermal expansion/contraction, large displacements needed to mobilise the passive resistance of a waste buttresses on the base liner, strain or displacement incompatibility between the waste and geosynthetic interfaces, earthquake-induced displacement, lateral waste movement, waste placement procedures, or waste settlement. These shear displacements can lead to mobilisation of a post-peak strength and/or progressive failure effects between the sideslopes and base of a bottom liner system.
2. The failure envelope that corresponds to the lowest peak interface strength may correspond to one or more geosynthetic interfaces because geosynthetic interface strength is stress-dependent. If more than one interface is used to develop the failure envelope for the interface with the lowest peak strength, the envelope is referred to as a composite failure envelope. The procedure for constructing a peak composite failure envelope for multi-layer liner and cover systems uses the following three steps:
   (a) Determine the interface(s) or material(s) in the composite liner system exhibiting the lowest peak strength for the full range of normal stresses encountered along the bottom liner system.

(b) Determine the peak composite failure envelope for the weakest interface(s) or material(s) in the composite liner system for the full range of effective normal stresses encountered along the liner system.

(c) Determine the residual composite failure envelope that corresponds to the peak composite failure envelope in Step (b).

3. Utilising the peak and residual composite failure envelopes obtained above, the two design scenarios for the bottom liner systems with a sideslope presented by Stark and Poeppel (1994) can be used:
   (a) assign residual shear strengths to the sideslopes and peak shear strengths to the base of the liner system and satisfy a factor of safety greater than 1.5; and
   (b) assign residual strengths to the sideslopes and base of the liner system and satisfy a factor of safety greater than 1.0 or 1.1 if direct shear data are used.
4. The stability of geosynthetic cover systems can be analysed using the peak shear strength of the weakest interface, or if necessary the weakest composite interface, with the factor of safety greater than 1.5. The use of a peak interface strength is recommended for the cover system because of the lack of or limited amount of detrimental shear displacement along the weakest interface in a cover system compared with a liner sideslope. However, if the average slope angle of the cover system is greater than the lowest peak interface friction angle, or large displacements such as construction-induced displacements or seismically induced displacements are expected, a residual interface friction angle should be used for design.

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