Discussion of "Modification to Translational Failure Analysis of Landfills Incorporating Seismicity" by X. Qian and R.M. Koerner

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The authors have emphasized the importance of fill sequencing as it relates to slope stability during operations of a lined repository. The neglect of changes in stability during operations have resulted in several large landfill slope stability failures, with one of the most notable and earliest of these being the Kettleman Hills hazardous waste landfill slide in 1987.

The authors' approach promotes a two-wedge analysis based on limit-equilibrium. Certainly the uses of limit-equilibrium techniques are the most common methods used to evaluate slope stability. Few design practitioners use finite-element techniques for performing slope stability, except for very complex situations or forensic analyses. Thus, refinements to approaches promoting limit-equilibrium stability methods are of continued value.

While limit-equilibrium techniques are useful tools for analysing slope stability, it is important that practitioners recognize inherent limiting assumptions. Fundamentally, limit-equilibrium analyses assume rigid-body mechanics, and assume that the factor-of-safety is uniformly distributed along the presumed failure surface. Except in the case where residual strengths are used along the entire slip surface and the factor of safety is less or equal to unity, this modelling limitation does not represent reality, and its use can significantly distort the true understanding of the distribution of the shear stresses along any presumed failure plane. This modelling limitation is likely one of the obscure and half-understood reasons that the historic use of a factor of safety of 1.5 has been required and largely successful for static slope stability analyses. The non-uniform mobilization of shear stresses may result in certain portions of a failure surface reaching, and exceeding, their peak strengths before others. This point was the essential lesson to be taken from the Filz et al. (2001) publication. Previous to Filz et al. (2001) there was a similar publication by Reddy et al. (1996) that made the same point. These are landmark publications that should be studied by any practitioner involved in slope stability analyses, especially those involving geosynthetic lining systems.

The Filz et al. (2001) and Reddy et al. (1996) papers indicate that shear stresses in real deformable bodies will initially begin to approach, and eventually may exceed, peak strengths at the outer slope toes of fills, and at the base of the back slopes of lined repositories under static filling conditions. The practical recognition of this phenomenon and associated design recommendations relative to evaluating the stability of lined repositories was first widely promoted and published by Stark and Poeppel (1994). Their paper should be recognized as the first for the general design recommendation that post-peak shear strength be used along lined backslopes, while peak strength can be used along the base of lined repositories.

An important concept that has been overlooked by the authors is that “a chain is only as strong as its weakest link”. Relative to performing slope stability analyses for multi-interface lining systems, this adage would be translated as “the critical interface will be the one with the lowest peak shear strength”. Figure 1 of this discussion shows a graph of the peak shear strength envelopes for all 8 interfaces considered by the authors in their example. It is clear from this graph that there are two interface strength envelopes that would control the slope stability of any configuration related to their design example: Interfaces II and VI (identical interfaces comprised of a geocomposite vs textured HDPE) have the lowest peak strength at effective normal stresses below approximately 100 kPa, and Interface III (textured HDPE vs woven side of GCL) has the lowest peak shear strength at effective normal stresses above approximately 100 kPa. Thus, these are the two shear strength envelopes that really should be used in evaluating the slope stability of the example problem. The designer would have to define the zones where the effective normal stresses are below 100 kPa, and use either the peak or post-peak strength parameters, whichever is deemed appropriate, of Interface II or VI for those zones. For the zones where the normal stresses are above 100 kPa the designer would use either the peak or post-peak strength parameters, whichever is deemed appropriate, of Interface III. In this particular example, the peak shear strength parameters of Interface VII are so close to those of Interface III that, for the zones where the designer wished to use post-peak strength parameters, it would be prudent to check the post-peak strength parameters from both those two interfaces and use the one that is the lower of the two. In this case it would still be Interface III that would control at effective normal stresses above 100 kPa.

Given the logic described in the preceding paragraph, the need to study the 64 options presented in the authors' Table 2 becomes obviated. The most critical interfaces could be chosen based on inspection of the graph of all peak strength envelopes superimposed on one graph, as shown in Figure 1, and only one analysis for the critical k_y would need to be run.

Furthermore, using this same logic, the authors' selections of the critical interfaces in their design example would be judged incorrect on two counts. First, the selection of Interface IV (internal shear strength of GCL) as the critical post-peak interface on the backslope, as indicated in some of the authors' examples, would not be correct because its peak shear strength is greater than that of either Interface III or VII for all effective normal stresses below 300 kPa. Thus Interface IV would <u>not</u> be expected to exceed its peak because either Interface III, or Interface II or VI, would fail first (depending on the effective normal stress) and never allow the GCL to mobilize its full peak internal strength. Second, along the base of the landfill, where the authors have chosen to use peak strength parameters, the critical interface would be different where the normal force was less than 100 kPa (in which case Interface II or VI would govern) compared to where the normal force was greater than 100 kPa (in which case Interface III would govern). The authors' examples do not indicate the selection of different critical interfaces for different effective normal stresses. Standard industry practice would take this into consideration.

In zones where a post-peak shear strength condition is assumed, the authors' approach for selecting the critical interface is equivalent to selecting the interface having the lowest post-peak shear strength along that interface. If this extreme assumption is to be maintained, it has severe implications for the geosynthetics industry. For example, this assumption would presume that regardless of how well a GCL is reinforced, the designer would ignore this internal reinforcement for zones where a post-peak condition is expected. This assumption would defeat a currently accepted practice of designing in a "slip surface" whereby another interface with a lower peak strength would absorb any deformations in the liner system due to construction, downdrag settlement, or seismic forces.

It is also worth mentioning that a larger discussion is required regarding the selection of peak vs residual shear strengths when performing analyses to determine the yield acceleration, k_y. The approach mentioned in the authors' paper of assuming the post-peak strength along the backslope and peak strength along the base really has its origins in static stability analyses. The landmark papers by Stark and Poeppel (1994), Reddy et al. (1996), and Filz et al. (2001) suggest that this condition represents the consideration of the selection of shear strength parameters in a *static* condition. The consideration of *seismic* conditions for determination of k_y adds a greater complexity to the selection of the appropriate shear strength parameters, and would generally involve a consideration of the expected *amount of deformation along the critical interfaces*. In practice this is often an iterative process, and readers are recommended to read Matasovic et al. (1997) for a broader discussion of this subject.

References

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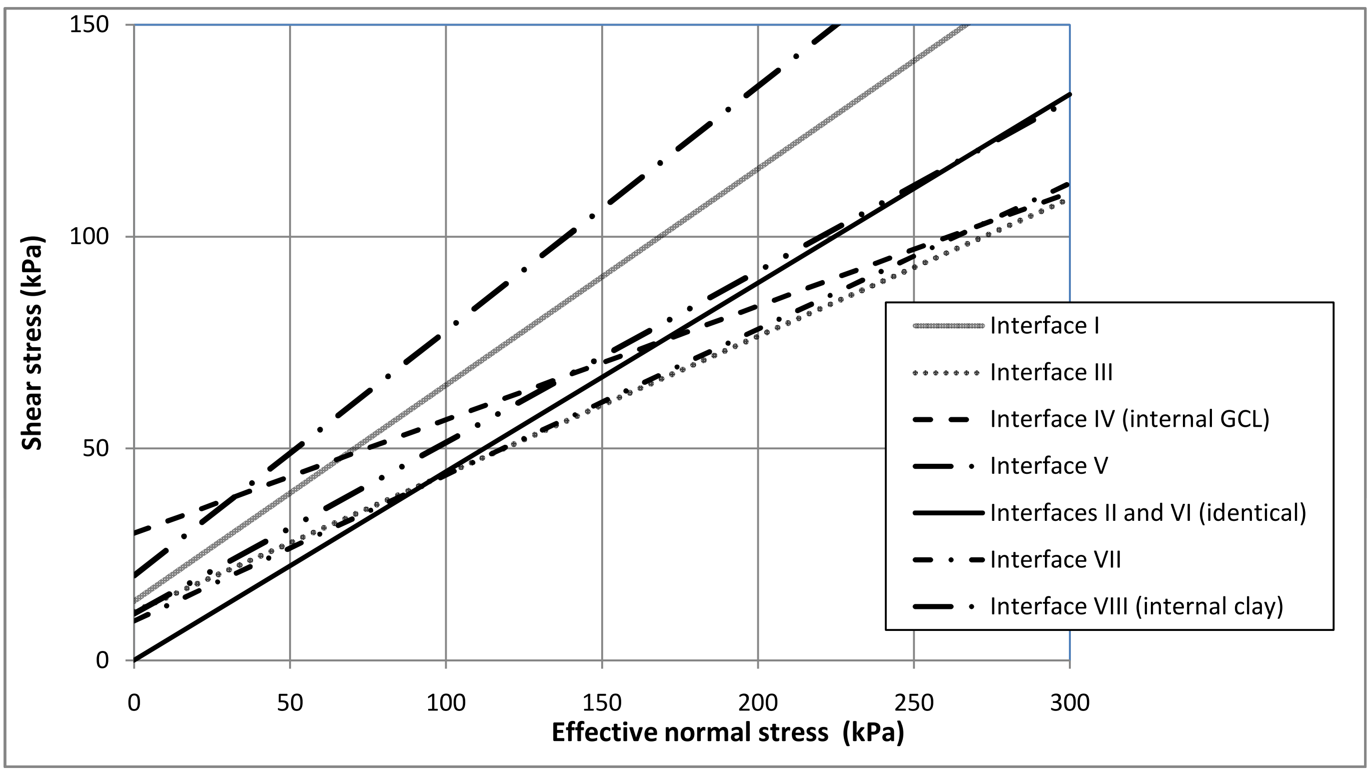
FIGURES:

Fig. 1. Peak shear strength envelopes for all interfaces from authors' example



150
100
50
0
0
50
100
150
200
250
300
Shear stress (kPa)
Effective normal stress (kPa)
Interface I
Interface III
Interface IV (internal GCL)
Interface V
Interfaces II and VI (identical)
Interface VII
Interface VIII (internal clay)