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Peak versus residual shear strength in geosynthetic-reinforced soil design

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The writer would like to state first that he has responded with a discussion to a previous publication by the author (Leshchinsky 2000). That discussion was followed by a comprehensive paper (Leshchinsky 2001) introducing a possible procedure to account for peak strength of soil in the framework of limit equilibrium analysis. The author's current paper reaches the same conclusion as his initial paper regarding the use of peak strength in design. However, the writer does not see further convincing evidence about the universal applicability of the author's conclusion. This discussion shows that the recommendation of using the peak strength should be carefully considered before its application to design.

Before discussing whether the peak strength recommendation is universally valid, the following comments are made for the record:

- The author refers to the paper by Leshchinsky and Boedeker (1989) as a source recommending the use of residual shear strength in the design of reinforced soil structures (e.g. Table 1). The writer cannot find such a recommendation in the referred (his) paper; however, Leshchinsky has made such recommendation in another paper, including rational justification, and it is likely that the author confuses the papers.
- The stability chart by Leshchinsky and Boedeker (1989) provides the total (sum) reinforcement force required for a limit equilibrium state. As this analysis deals effectively only with global stability, Leshchinsky and Boedeker (1989) illustrate the effects of assumed distribution of maximum force among the reinforcing geosynthetic layers (see p. 1476). This illustration is based on a reference where an in-depth and rigorous analysis was conducted. The author's commentary states that Leshchinsky and Boedeker (1989) use linear force distribution (see response to Dr Giroud, and reference to Figure 10 on p. 384), but the fact is that they recommend assessing two possible extreme distributions in design—linear and uniform—and use the synergistic results. This recommendation should be viewed in the perspective of 1987 when the referenced paper was written; today preference would likely be to use uniform mobilization (as commonly used in reinforced slope stability analysis and as was used for verification by the author in his response to Dr Giroud).
- The author states that Jewell (1991) recommends using residual shear strength. Without describing Jewell's rationale, this statement is simply out of context. Such background is available in Jewell's (1990) keynote paper. Based on extensive study of shear test results on compacted granular soils, Jewell concluded that the residual strength of such soils is, at most, 1.3 times smaller than their peak strength. Therefore Jewell recommends either using the residual strength as is or using the peak strength reduced by (a factor of safety of) 1.3. His suggested residual strength relates to design, and is practically compatible with most current design methods for reinforced slopes that recommend the use of peak strength combined with a minimum global factor of safety of 1.30. Recognizing the limitations of limit equilibrium analysis, the writer thinks that Jewell's approach is logical.

In supporting the use of peak strength, the author refers to existing design methods. The author correctly states that most design methods recommend the use of peak shear strength in design. However, in slopes these methods also recommend a minimum global factor of safety of 1.3 to 1.5 which, *de facto*, reduces the peak strength to below its residual value. Hence the existing design methods for reinforced slopes support, *de facto*, the residual strength approach ...

Having said the above, one must also state that, typically, lateral earth pressures are used in the design of reinforced walls (to be distinguished from reinforced slopes; the common distinction between walls and slopes in practice is 70°). This is unlike the approach by

Leshchinsky and Boedeker (1989) or Jewell (1991), which is based on slope stability concepts. Unfortunately the lateral earth pressure approach adds to the confusion when compared with the conventional slope stability-based approach. Although the slope stability approach is equally applicable for reinforced 'walls' and 'slopes' (the differentiation between the two is more semantic and arbitrary than physical), the tradition of using lateral earth pressures prevails in most design methods for 'walls'. Hence the argument by the author for using peak strength in calculating lateral earth pressures (and the size of the active wedge) might be relevant and thus worthwhile reviewing.

Consequently, in the context of walls, one can now refer back to design methods using peak strength and review whether this practice indeed supports the author's argument. First, limit equilibrium is valid (i.e. can be verified) only when the factor of safety is 1; rarely would a structure be 'designed' for such a factor of safety. Furthermore, the writer is not aware of practitioners who use the plane strain shear test to determine the peak shear strength of the soil (even the author in his own research did not use such a test; it requires special lab equipment). Subsequently, *if* laboratory data are available, they are at best related to direct shear or triaxial tests, thus rendering wall designs that are based on lower peak strength than in plane strain. Furthermore, plane strain conditions rarely prevail in reality; any 3D effects just increase stability relative to the assumed pure plane strain conditions in design. Most importantly, lateral earth pressures for walls are conservative, as they assume linearly increasing distribution of lateral pressure with depth (proportional to overburden). This results in a required tensile strength that is about twice as much when compared with the slope stability approach (i.e. uniform mobilization of strength). This large required strength is further increased by reduction factors (e.g. creep) that can be overwhelming in the short term. Can the satisfactory performance of walls be linked to the use of peak strength in current designs? The good performance of walls, even if designed for low factors of safety, may be attributed to other factors than the use of peak strength. The writer fails to see a direct relationship between the current 'use' of peak strength in design and the good performance of walls; the performance can be attributed to much more influential factors.

The author makes a strong design recommendation based also on tests using centrifugal models. The following comments are made with regard to these tests:

- The author's centrifugal models do not simulate construction (whereby layers of soil are placed and compacted, thus allowing for deformations during construction to occur). His model simulates, at best, a propped reinforced steep slope. How does this affect the displacement of soil possibly reaching its residual strength along a potential slip surface? The tested models do not provide an answer.
- The author used sand in modeling a prototype. In centrifugal modeling, all linear dimensions are increased by the ratio of centrifugal acceleration to gravity (to render the same stresses and strains as in the prototype). At the accelerations used, the average sand particle simulated a prototype's average particle size of about 30 mm. That is, the model particles were not scaled, thus rendering a prototype composed of gravel (with some particles nearly as large as boulders). Does this represent a realistic prototype? Would large particles produce stronger interaction between reinforcing layers and reinforced soil? As a minimum, the author would have to test a 'model of a model' showing that not scaling the particles does not affect the simulated problem, thus making his modeling relevant to a desired prototype.
- The close spacing of reinforcement layers in the model combined with the same sand as in the prototype produces a much stronger interaction between the reinforcement layers and the soil. Furthermore, the thickness of the failure surface (i.e. the shear band) could play a role in increasing stability in small-scale tests, whereas in a full-scale structure it is typically negligible (see references in Leshchinsky 2002). The end result is potentially overpredicting stability. That is, the results could be on the unsafe side when compared with the prototype. In this connection, it is interesting to note the centrifuge model testing by Law *et al.* (1992) and its unconservative correspondence to a prototype.
- The author states that reaching residual strength has to do with extensibility of the reinforcement and its possible deformation during construction. Actually, movement of soil during construction can occur with nearly no deformation of the reinforcement (movement of soil is not necessarily deformation, or strain, in the reinforcement). Soil can move outwards and down owing to compaction, sliding a little over the reinforcement, thus mobilizing soil strength and rendering a residual strength at some locations along a potential slip surface. The author's work does not verify construction effects.
- In addition to movements during construction, progressive failure is another possible reason for the approaching residual strength. Progressive failure is related to the displacement field within the soil mass, and it may develop over time within the reinforced mass. It is not too difficult to measure the displacement field in centrifugal models (e.g. Leshchinsky *et al.* 1982). Such data could provide a tool to judge whether progressive failure occurred in the author's experiments. However, it seems that the author did not conduct such measurements. What is the basis then for the author's statement that progressive failure was not observed? Was there any time lag between reaching the target acceleration and the 'sudden' collapse? Figure 4 (and the original paper by the author) indicates that there was such a lag; in the context of progressive failure, what does such a lag mean? Could it be redistribution of loads as progressive failure develops? Experimental work by others clearly shows

the phenomenon of progressive failure in reinforced soil (see references in Leshchinsky 2001).

- The conclusion that peak strength should be used is based on the apparent observation that there is no progressive failure, and on a numerical comparison with limit equilibrium predictions for $F_s = 1.00$ (i.e. at failure when limit equilibrium analysis is physically meaningful). Although such a comparison is indeed required, it is incomplete. Limit equilibrium analysis yields coupled results: minimum factor of safety *and* its associated critical failure surface. The author checks the factor of safety (see response to Dr Giroud) but does not show the agreement of the one element he could measure directly in his experiments: that is, the trace of the observed critical surface. What is the value of the results if the calculated factor of safety is 1, but the critical trace of the failure surface is different? In fact, the writer sees the most important contribution of the author's work in identifying the failure mechanism. Leshchinsky and Lambert (1991) used an alternative modeling technique to verify this mechanism. Centrifugal models are more realistic for such a study. The writer is eagerly looking forward to a parametric study showing the measured and predicted trace of critical slip surfaces. Such a parametric study could be very meaningful if used in conjunction with a safety map such as that proposed by Baker and Leshchinsky (2001).

Finally, the concept of residual and peak shear strength in design should also be discussed at the philosophical, though practical, level. Based on experimental evidence at the elemental level, Leshchinsky (2001) suggested that location of the critical slip surface is dictated by the peak strength. However, Leshchinsky (2001) also suggests that the required reinforcement strength be determined based on the drop of soil shear strength along this surface, which, for simplicity (and lack of precise knowledge), is assumed conservatively to reach the residual strength, thus recognizing the potential for the development of progressive failure. This approach was termed the hybrid approach, and was properly cited by the author. Compared with the exclusive use of residual strength, it yields significantly shorter reinforcement, whereas the reinforcement required strength is somewhat higher than for the exclusive use of peak strength. The hybrid approach is applicable to reinforced walls and slopes. However, the author concludes in his paper that progressive failure does not develop in geosynthetic-reinforced slopes, thus justifying the use of peak strength only. The writer thinks that very detailed experimental modeling is needed before reaching such a firm conclusion, especially when related to design using a limit equilibrium analysis (i.e. stability assessment). The reported experimental modeling may indicate whether the location of the trace of the critical surface is controlled by the peak or residual strength (or by neither). Such a conclusion could be significant (even beyond the question of residual or peak strength), thus making the author's work valuable. Contrary to the author, the writer argues, that in a limit state, progressive failure *must* exist in geosynthetic-reinforced structures regardless of a particular experimental verification. The following explains this argument.

Polymeric material exhibits time-dependent behavior. For example, conducting the wide-width ultimate strength test on a geosynthetic will yield different ultimate strength values at different rates of loading: that is, the creep component effect on the measured ultimate strength will become more pronounced as the test is performed at a slower rate. In fact, this is the main reason for this test to be considered an index test. In general, when a slope, reinforced or not, possesses a factor of safety of about, say, 1.1, deformations are visible. The embedded reinforcement must be stressed with deformations (it is no longer dormant), and, depending on its level of stress, it will progressively deform with time (i.e. creep), rendering further deformation in its vicinity within the soil mass. This may result in sections along the *potential* slip surface exceeding the peak strength, possibly reaching the residual value. As creep strain progresses, the soil deforms and the resistance contribution by the soil progressively drops further, resulting in further decrease of the factor of safety, increase in reinforcement load, further deformations, even more sections approaching the residual strength, and so on until creep rupture occurs. The process for a reinforced slope possessing a factor of safety of, say, 1.1 may take time t. When the factor of safety is 1.01 it may take, say, $t/1000$: that is, even when the factor of safety is very near 1.0, there is an element of progressive failure that is introduced by creeping geosynthetic. To state *a priori* that progressive failure does not exist is not prudent in design when examining a limit state; it is conceptually incompatible with the time-dependent behavior of the reinforcing element. Indeed, this scenario is not seen during the life of reinforced structures when the design calls for proper safety and reduction factors (rendering, *de facto*, 'working conditions'); however, limit equilibrium analysis is physically valid only when the system is at the verge of failure, meaning that the factor of safety approaches unity.

In view of the above argument, one may revisit the author's work and realize that the loading rate is not reported as a major variable. In fact, it would be a major experimental effort to carry out various centrifugal tests, each conducted at a different sustained level of loading until failure occurs and the reinforcement ruptures. Such a test is analogous to the conventional creep tests; however, it uses a centrifuge testing facility for many hours/days/months to induce in-soil creep in geosynthetic reinforcement.

Concluding this discussion on a practical note, the writer would like to point out that geosynthetic reinforcement is increasingly being used with local backfill. Unlike metal, its durability (corrosion) is less of an issue: thus the level of 'select fill' is limited by constructability and drainage. In most places, lower-quality backfill is substantially cheaper than sandy backfill, making it economically attractive and likely to make the requirement for high-quality backfill obsolete in a few years. Lower-quality backfills do not possess much of a strain-softening, and therefore render the argument of residual versus peak strength of academic interest. In fact, the lack of proper soil testing in practice already makes this discussion academic.

REPLY BY THE AUTHOR

The author thanks the discusser for his interest in the paper and for the arresting discussion. The author agrees with many of the observations posed by the discusser with one notable exception: the experimental results presented by the author indicate that the soil peak shear strength governs the stability of geosynthetic-reinforced slope models.

The discusser addresses several different issues regarding geosynthetic-reinforced soil design. They are all relevant issues. However, several of them are beyond the scope of the author's paper (e.g. the use of local backfills). Nonetheless, the comments below address important issues raised by the discusser:

- The discusser is correct in pointing out that the paper by Leshchinsky and Boedeker (1989) does not provide recommendations on the selection of residual shear strength. While the author considers the work by Leshchinksy and Boedeker (1989) to be the seminal reference of the discusser's methodology, the recommendation for using residual shear strength in design is only explicitly stated in later contributions by the discusser (e.g. Leshchinsky 1999; Leshchinsky *et al.* 1995).
- Many of the issues raised by the discusser regarding the use of centrifuge modeling of geosynthetic-reinforced soil structures are addressed by Zornberg *et al.* (1997).
- The author agrees with the discusser on the need to check the agreement between the estimated location of the critical failure surface and the experimentally observed trace. However, this evaluation has been already conducted and, as reported by Zornberg *et al.* (1998b), the agreement is very good.
- The testing program suggested by the discusser involving carrying out various centrifugal tests, each conducted at a different sustained level of loading until failure occurs, is certainly appropriate. Indeed, a testing program of this nature has been recently conducted by the author's research group, though the results have not been reported yet.

Partly based on the centrifuge tests reported by the author (Zornberg *et al.* 1998a), the discusser has recommended a hybrid approach (Leshchinsky 2001) in which the peak soil shear strength is used to locate the critical slip surface, while the residual soil shear strength is subsequently used along the located slip surface to compute the reinforcement requirements. Although this is a thoughtful approach, one of the arguments offered by the discusser on the need to use the residual soil shear strength is for added conservatism. The author believes that conservatism in design should be quantified by identifying the shear strength governing failure (peak shear strength in the case of the reported centrifuge models) and selecting an appropriate factor of safety. This approach appears more consistent than the discusser's proposal of identifying a lower shear strength value (e.g. residual shear strength) and selecting a factor of safety that may no longer be a good indicator of the actual conservatism in design.

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