PERFORMANCE OF GEOSYNTHETIC REINFORCED SLOPES AT FAILURE[a]

Discussion by Dov Leshchinsky,[4] Member, ASCE

The discussed paper and its companion (Zornberg et al. 1998) provide much needed insight into limit equilibrium design of geosynthetics reinforced steep slopes. It uses well-focused experimental work. The purpose of this discussion is to add a perspective. This perspective is relevant only in the context of design.

It can be postulated that the main objective of design is to produce safe and economical structures. In light of this postulate, the following three points, all related to the discussed paper, are examined:

1. Use of peak shear strength rather than residual
2. Redistribution of reinforcement force
3. Modes of failure

PEAK VERSUS RESIDUAL STRENGTH

In their conclusions the writers state, ''The test results indicate that the stability of the reinforced slopes is governed by the peak shear strength and not by the critical state shear strength of the backfill soil.'' The writers then indicate that ''failure initiated at midheight of the slopes.'' In fact, the writers discuss in some detail the elapsed time between failure initiation and complete collapse observed in various model configurations. Clearly, this observation indicates a phenomenon of progressive failure, implying that while the soil is about to reach its peak strength along portions of the slip surface, it has already passed the peak along other portions, perhaps reaching its residual strength. The noninstantaneous development of slip surface is to be expected because of the inclusion of discrete reinforcement layers. The fact that this reinforcement tends to creep rapidly when approaching break, and the nonuniform stressing among layers just facilitates the possibility of progressive failure. There are several publications indicating directly that, for reinforced soil, failure is a progressive phenomenon [e.g., Huang et al. (1994)]. Other publications imply indirectly progressive failure by describing large deformations and bulging before ''collapse'' occurred [e.g., Al-Hussaini and Perry (1978) and Tatsuoka et al. (1989)]. The writers' conclusion (see also the companion paper) would produce a more economical structure compared with the use of residual strength. However, is it safe to conduct limit equilibrium analysis of reinforced steep slopes employing peak shear strength along a slip surface that develops progressively? Since there is no clear experimental or logical answer, it would perhaps be prudent to adopt a pessimistic design approach assuming that stability must be insured even if residual shear strength is fully reached along the slip surface. That is, the stability of steep slope hinges on the strength of the reinforcement (i.e., without reinforcement the steep slope will slide; e.g., granular slope steeper than its angle of repose).

Use of residual strength has clear cost implications in the design of reinforced steep slopes. The required strength of the reinforcement increases somewhat; however, the required length increases significantly since much deeper slip surfaces are predicted. This extra length makes construction more difficult, especially if a space constraint exists (e.g., widening existing embankment), thus rendering construction more expensive than just the cost of extra reinforcing material. Hence, this combined with what currently appears as overly conservative designed reinforced slopes create a need to introduce a less conservative design approach. The authors' experimental results may support the following proposed modified approach. The full details of the modified approach are presented by Leshchinsky (1999); it is restricted to steep slopes (and not to reinforced slopes that are stable without reinforcement, making the installed reinforcement ''dormant'').

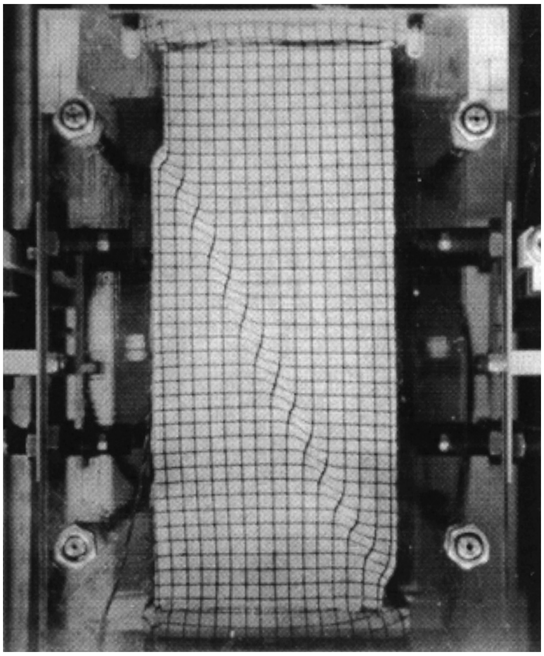

FIG. 16. Shear Band in Plane Strain Compression Test (Ticino Sand: D_r = 79%, D_{50} = 0.527 mm at σ_3 = 78 kPa and γ_{max} = 13.3%) (Photo Courtesy of Prof. F. Tatsuoka, University of Tokyo, Japan)

It is an experimental fact that only one slip ''surface'' develops during the shear of granular dense soil element (e.g., unreinforced soil in triaxial or plane strain tests). In these tests, combining Mohr circle with Coulomb failure envelop produce a shear surface inclined at an initial angle of $(45° + \phi_{peak}/2)$ to σ_3. As displacement continues, a shear band forms and the residual state of strength is reached. As an example, see Fig. 16 [reproduced from Yoshida and Tatsuoka (1997)]. Observing Fig. 16 (unreinforced soil element), as well as Figs. 4, 7, and 9 (reinforced slope models) presented by the authors, one can define an average slip ''surface'' within the shear zone. That is, there are not two different slip surfaces, one attributed to ϕ_{peak} and the other to $\phi_{residual}$.

It should be noted that based on plane strain compression tests conducted on 12 different unreinforced sands, Yoshida and Tatsuoka (1997) have demonstrated that the average inclination of the shear band (i.e., the ''slip'' surface) is slightly less than $(45° + \phi_{peak}/2)$. That is, it is approximately related to ϕ_{peak}. This observation, however, is valid for medium to fine sand while the confining pressure is less than, say, 100 kPa. In their companion paper, the authors demonstrate that a single ''slip'' surface also develops in their reinforced slopes (i.e., in a sense, similar to the soil element). They show via limit equilibrium back-calculations that indeed the traced slip surfaces correspond well to ϕ_{peak}. Consequently, the following hybrid procedure is proposed for design steep slopes when the backfill is composed of medium (or less) cohesionless soil and heights do not exceed, say, 6 m:

[a]August 1998, Vol. 124, No. 8, by Jorge G. Zornberg, Nicholas Sitar, and James K. Mitchell (Paper 14816).

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1. Use ϕ_{peak} and limit equilibrium analysis to locate the critical slip surfaces. These surfaces will be used to determine the required layout of geosynthetic layers (i.e., length and spacing). Note that in reinforced slopes there can be several critical slip surfaces [i.e., surfaces producing same minimum factors of safety (Leshchinsky 1997)]. Note that ϕ_{peak} without any reduction is used [i.e., unlike conventional limit equilibrium analysis, no formal *Fs* on soil shear strength is used; this is similar to the design of reinforced soil walls—see Leshchinsky (1999) for details and definition of *Fs*].
2. Use $\phi_{residual}$ along traces of the critical slip surfaces to compute the required geosynthetic strength.

Note that this hybrid approach recognizes that slip surfaces will initiate and have a trace based on the soil peak strength. However, development of progressive failure is feasible, and at this state, the ductile reinforcement should be sufficiently strong to keep the system stable. It is entirely possible that the backfill in steep slopes will deform (during or after construction), mobilizing the soil beyond its peak strength. Consequently, the reinforcement strength becomes critical to stability in case residual strength develops. It should be pointed out that Tatsuoka et al. (1998) proposed a similar hybrid approach; however, it was limited to seismic design of reinforced walls.

In the strict sense of mechanics, use of $\phi_{residual}$ along the slip surface defined by the shear band cannot yield static equilibrium when considering an element (Fig. 16). However, in the context of limit equilibrium analysis, this is not an issue since only global equilibrium needs to be satisfied for a system comprising soil and reinforcement. Hence, the analogy to a soil element at failure is restricted to the experimental observation of the development of a singular shear ''surface.''

The proposed procedure in this discussion may result in significantly shorter reinforcement as compared to using $\phi_{residual}$. However, the required reinforcement strength will be somewhat larger than that computed using ϕ_{peak} [see Leshchinsky (1999) for results of analysis and a simplified approach].

REDISTRIBUTION OF REINFORCEMENT FORCE

Limit equilibrium analysis can deal with the global equilibrium of reinforced slopes. As input, it requires the knowledge of the reinforcement layers' force at their intersection with the slip surface. The authors suggest a distribution that is proportional to overburden pressure above the point at which the reinforcement intersects the slip surface. As failure approaches, redistribution of force will occur leading to nearly uniform mobilization of reinforcement strength (Fig. 15). The following points are noted:

1. Force distribution in the reinforcement is mainly a function of local conditions (e.g., foundation soil), the length of the reinforcement layers, and the stiffness and ductility of the reinforcement. Conventional limit equilibrium analysis cannot account for these factors in predicting the reactive force in the reinforcement. Boedeker (1987) conducted an extensive parametric study on the effects of various assumed distributions on limit equilibrium stability of reinforced slopes. Subsequently, Leshchinsky and Boedeker (1989) suggested ''upper'' and ''lower'' values for the required force employing linear and uniform distribution with depth, respectively.
2. In the context of design, logical and consistent modification of limit equilibrium analysis can lead to a range of feasible values for the reactive force in the reinforcement layers (Leshchinsky 1992; Leshchinsky et al. 1995). One possible distribution, stemming from limit equilibrium consideration of multiple failure surfaces, is linear. The second distribution is uniform. Leshchinsky et al. (1995) concluded that the reactive force in reinforcement layers stabilizing a steep slope is probably contained between these two values. (Force in the reinforcement will also depend on its tributary area; i.e., spacing.)
3. The authors suggest that ductility of geosynthetic reinforcement will allow for load redistribution leading to a different (more efficient and therefore, less conservative) force distribution. The discusser also thinks that load shedding (or redistribution) is likely to occur as the geosynthetic deforms and, more likely, as it creeps and relaxes (Leshchinsky 1992; Leshchinsky et al. 1997). However, in the context of design, how does one define sufficient ductility? Is the proposed new distribution valid regardless of the foundation? The writers indeed point out that ''further research should be undertaken to validate the proposed distribution.''

To avoid uncertainties, it is perhaps safer to use in design a limit equilibrium analysis that considers both linear and uniform distributions [e.g., Leshchinsky (1997)]. If the reinforcement layout is wisely selected, such an approach is not overly conservative when compared with the authors' proposed distribution. However, it covers a wide range of possible realities, insuring both local and global stability (i.e., linear and uniform distributions, respectively). The end result is not necessarily more reinforcement but rather adequate reinforcement ''density'' along the height to insure safe structure. For realistic length of geosynthetic, this approach shows good agreement with relevant field data recorded by Fannin and Herman (1990) [see comparison by Leshchinsky et al. (1995)].

It is interesting to note that the authors' suggested distributions, which are based on overburden pressure, imply nonuniform mobilization of reinforcement layers' tensile strength. When the geosynthetic layers approach a break, such nonuniform reactive force will lead to different rates of creep in each layer. This, in turn, will result in different soil movement around each layer, thus producing progressive failure.

MODES OF FAILURE

It should be stated up front that massive failure of reinforced steep slopes has rarely occurred (surficial failures are common). The failure mechanism observed by the authors provides an insight about one important mode of failure (i.e., ''internal'' failure). However, this is only one possible mechanism.

In all tests, the reinforcement layers were long relative to the height of the slope (L/H = 0.9). It is clear that the combination of reinforcement layout and geotextile strength ''allowed'' for the development of only one failure mechanism: rotational surface passing through all layers (i.e., made all other potential modes of failure less critical). In reality, however, shorter reinforcement is used. Also, due to large strength reduction factors (i.e., for creep, installation damage, and aging), the strength of specified geosynthetics is much larger than the value needed for short-term stability. Consequently, other possible failure mechanisms are feasible (possibly more critical) and must be checked in design. For example, direct sliding is feasible, especially in slopes flatter than the one experimented by the writers (i.e., flatter than 1H:2V). Furthermore, compound rotational failure (i.e., failure extending through the reinforced soil into the retained soil) is feasible and likely. Such mechanisms were discussed, formulated, and computerized, for example, by Leshchinsky (1997).

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Discussion by Bruno T. Dantas[5] and Mauricio Ehrlich,[6] Member, ASCE

A parametric numerical study has been performed to provide a basis for the extension of Ehrlich and Mitchell's (1994) analytical working stress method for reinforced vertical walls to generic reinforced soil slopes (Dantas 1998). The following factors were analyzed in this study: the influence of slope height and inclination, the reinforcement stiffness and spacing, the soil friction angle, and compaction-induced stresses on the soil on reinforcement tension. A modified version of the CRISP92 (Britto and gunn 1990) finite-element program with Soil Compaction (Iturri 1996) was used for the analyses.

Taking into account the above mentioned finite-element analyses (Dantas 1998), some points of the authors' conclusions are addressed as follows.

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STRESS PATH AND CONSTRUCTION PROCEDURE

The prototype construction sequence usually involves the placement and compaction of backfill until the slope crest is achieved. In centrifuge tests the stress increase due to acceleration to obtain the corresponding prototype stress levels does not represent the actual stress path in the field. The soil is subjected to only one straightforward loading to the stress state at the end of construction, and the induced stress due to soil compaction is not represented. As part of Danta's (1998) studies, the effect of the stress path and construction procedure on the reinforcement mobilized tension was evaluated.

Fig. 17 shows finite-element method results for two hypothetical 5 m high reinforced soil slopes, one with geosynthetic reinforcement and the other with metallic reinforcement, both with face inclination equal to 60°. In the analyses it was assumed that the soil friction angle, ϕ, was equal to 35° and the soil unit weight, γ, equal to 19.6 kN/m^3. The relationship between the reinforcement maximum tensile stresses, T, and depth, z, in a normalized form, determined for the geosynthetic reinforced soil slope are shown in Fig. 17(a) and for the metallic reinforced soil slope in Fig. 17(b).

The relative soil reinforcement stiffness index, S_i, is defined by Ehrlich and Mitchell (1994) as

$$S_i = (E_r A_r)/(\kappa P_a S_v S_h) \tag{14}$$

where $(E_r A_r)$ = reinforcement stiffness; κ = Duncan et al.'s (1980) modulus number for loading; P_a = atmospheric pressure (a units constant); and S_v and S_h = vertical and horizontal spacing between adjacent reinforcements, respectively.

Three construction procedures were considered: a multilayer sequence, a single-layer sequence, and a multilayer sequence with soil compaction (only available for the metallic reinforced soil slope). The compaction equipment was equivalent to the

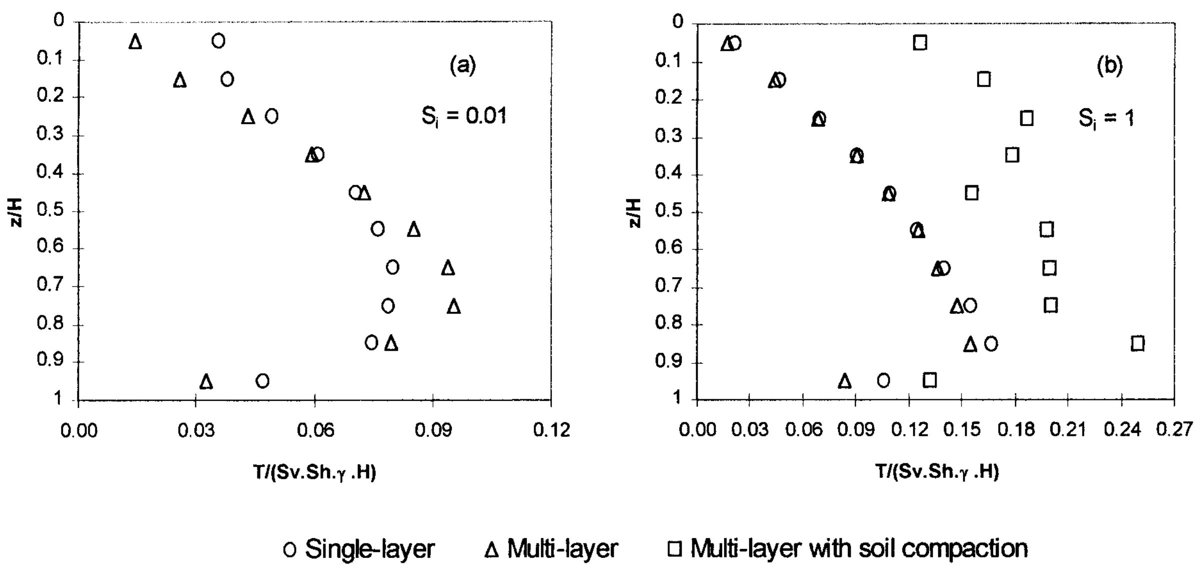

FIG. 17. Effect of Stress Path and Construction Procedure on Reinforcements Maximum Tension for 60° Slope Reinforced by (a) Geosynthetic (S_i = 0.01) and (b) Metallic (S_i = 1) Inclusions

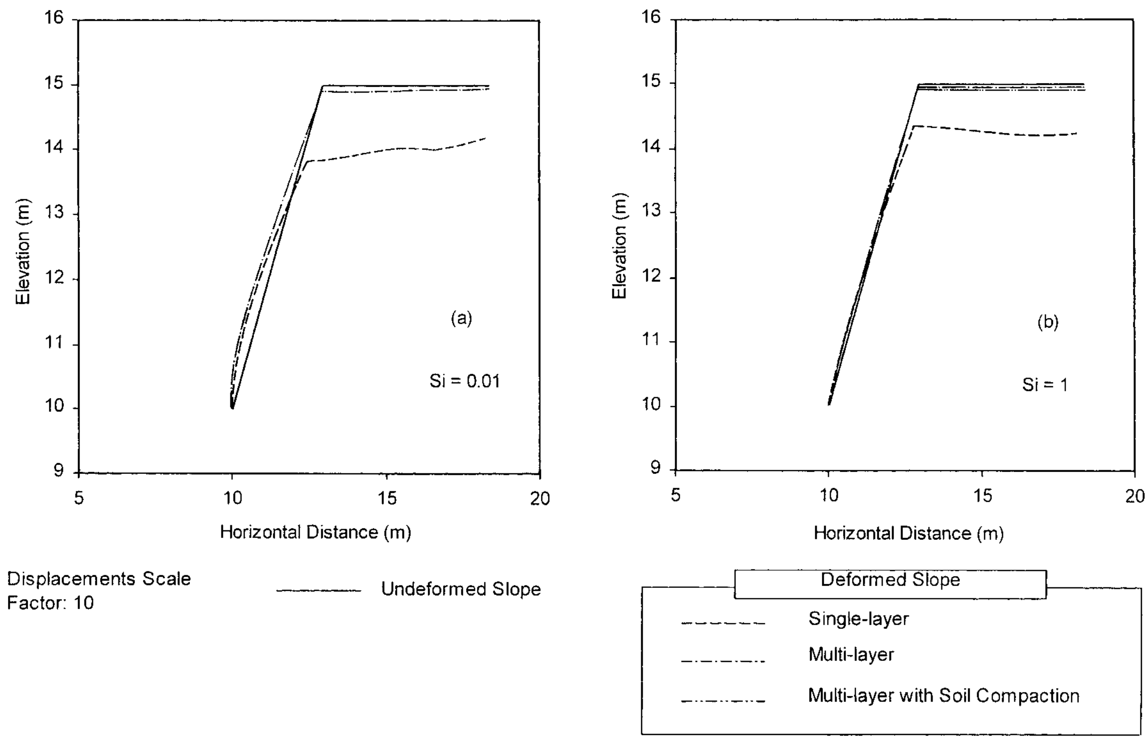

FIG. 18. Effect of Stress Path and Construction Procedure on Deformability of 60° Slope Reinforced by (a) Geosynthetic (S_i = 0.01) and (b) Metallic (S_i = 1) Inclusions

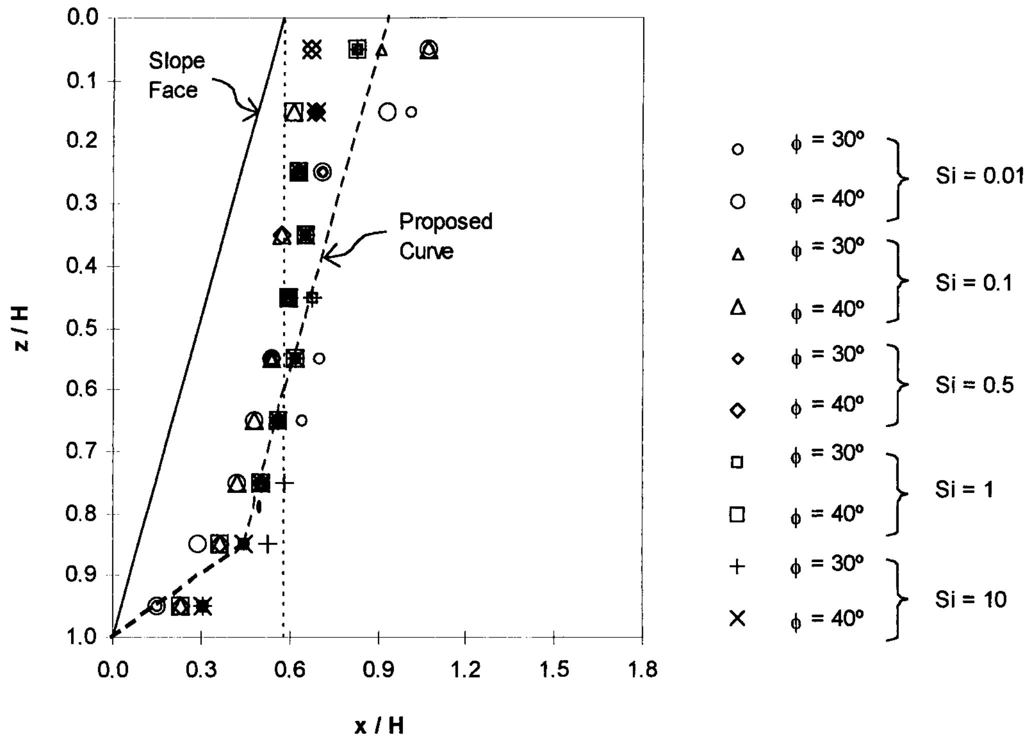

FIG. 19. Location of Maximum Tension Point along Reinforcement for Slopes with Face Inclination of 60° and Different Soil Friction Angles and S_i Values

Dynapac CA25 vibratory roller with maximum drum force of 160 kN.

The results in Fig. 17 show that, under working stress conditions, the construction sequence is not a major influence factor on the mobilized reinforcement tension, but soil compaction may not be neglected. Basically, the same conclusions were also obtained for all other slope inclinations and reinforcement types studied. Adib (1988) and Ehrlich and Mitchell (1994) had also shown for reinforced soil walls that soil compaction is an important influence factor on reinforcement tensile stresses. Ehrlich and Mitchell (1994) have shown that for typical condition the compaction may be the major contributor to reinforcement tension to depths of more than 6 m. Therefore, as the authors' centrifuge models were not concerned with soil compaction, there may not be complete equivalence between model and prototype, unless the induced compaction soil stress could be neglected in the field. The discussers believe that the effect of soil compaction on the performance of geosynthetic soil slopes at failure is still to be addressed.

Another aspect to be pointed out is the stress path's influence on the slope deformability. Dantas's (1998) numerical results can also be used to show that different deformation patterns are associated with different stress paths, particularly when vertical displacements are focused (Fig. 18). The numerical studies show that the horizontal displacements are not significantly affected by the construction sequence. Considering these results, the monitoring of horizontal displacements may be the best procedure for evaluation of deformability of prototype slopes trough centrifuge studies.

TENSION DISTRIBUTION WITH DEPTH

The authors have pointed out that failure initiated at midheight of the slopes, contradicting assumptions in current design methods that failure should develop from the toe of the reinforced slopes. These experimental results are consistent with finite-element analysis results performed by Dantas (1998) that show a nonlinear reinforcement tension distribution with depth under working stress (in Fig. 17 these results are exemplified). For different slopes inclinations and reinforcement stiffnesses, Dantas's (1998) results show that the layer at which the highest reinforcement tension value occurs varies over a depth of 70 to 80% from the top of the slope.

Nevertheless, it is important to note that the authors' experimental results may have been influenced by the additional reinforcement at the toe of the slope provided by the overlap of the inclusions. As pointed out by the authors, the uppermost reinforcements may have been overloaded compared to the ones located near the bottom of the wall.

LOCATION OF POTENTIAL RUPTURE SURFACE

The results shown in Fig. 19 support the authors' statement that the potential failure surfaces may be independent of the reinforcement stiffness and spacing. The location of the potential rupture surface may be interpreted as the location of the maximum tension point along the reinforcement. The results show that under working stress conditions, this location is not significantly affected by the soil friction angle or by the reinforcement stiffness and spacing, but it may be considered only a function of the slope geometry.

SOIL-REINFORCEMENT STRESS-STRAIN COMPATIBILITY

The authors' results show that even for geosynthetic reinforcement systems (extensible reinforcements) the stability is governed by the peak soil shear strength. Thus, at the failure surface, i.e., at the reinforcements' maximum tension point, reinforcement deformations are much lower than expected, based on results of unconfined laboratory test. Therefore under working stress the soil may not be fully plastified. This conclusion supports the discussion addressed in the companion paper.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

A_r = reinforcement transverse area;
E_r = reinforcement modulus;
H = slope height;
P_a = atmospheric pressure;
S_h = horizontal spacing between adjacent reinforcements;
S_i = relative soil reinforcement stiffness index;
S_v = vertical spacing between adjacent reinforcements;
T = reinforcement maximum tension;
x = horizontal distance from toe of slope;
z = depth from top of slope;
γ = soil unit weight;
κ = modulus number (hyperbolic stress-strain curve model); and
ϕ = soil friction angle.

Closure by Jorge G. Zornberg,[7] Nicholas Sitar,[8] Members, ASCE, and James K. Mitchell,[9] Honorary Member, ASCE

The writers thank Prof. Leshchinsky for his interest in the paper. Based partly on interpretation of the centrifuge test results discussed by the writers, the discusser suggests a new approach for designing geosynthetic reinforced steep slopes. Central to his approach is the use of a hybrid design procedure in which peak soil shear strength properties would be used to locate the critical slip surface, while the residual soil shear strength properties would subsequently be used along the located slip surface to compute the reinforcement requirements.

Even though use of the residual shear strength in design represents a conservative approach, this conservatism is not supported by the centrifuge data presented by the writers in the paper, which clearly showed that failure of the geosynthetic reinforced slope models was governed by the peak shear strength of the backfill. The perceived conservatism in design is also not supported by the generally observed good performance of monitored reinforced soil structures. The controversial issue regarding selection of soil shear strength properties was extensively discussed, though no consensus was reached, among practitioners and researchers attending the 6th International Conference on Geosynthetics (Panel Session: Reinforcement Applications—User's Questions—Industry's Response, Atlanta, March 1998). Although the hybrid approach proposed by the discusser is a feasible design method that departs from the strict use of residual shear strength properties [e.g., Leshchinsky and Boedeker (1989)], the writers believe that a hybrid approach would not represent the failure observed in the centrifuge tests and would lead to a still conservative method for the design of reinforced soil slopes.

The discusser's remarks regarding progressive failure are based on the premise that the soil will attain its full (peak)

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shear strength before reinforcement rupture, which is not supported by the reported experimental centrifuge data. No shear displacements were clearly observed along the ultimate slip surface before reinforcement rupture. In contrast to the discusser's view that the stability of steep slopes would eventually hinge upon the strength of the reinforcement, failure of the centrifuge models was triggered by the rupture of geotextile reinforcements. The discusser is correct in pointing out that some time elapsed between failure initiation and final collapse. However, the elapsed time occurred without any increase in the g-level. Consequently, final collapse after failure initiation is attributed to the loss of reinforcement tensile strength due to breakage, rather than to the decrease in soil shear strength from peak to residual values due to shear displacements.

The writers believe that design approaches for geosynthetic reinforced slopes should use peak soil shear strength for both defining the location of the slip surface and calculating the reinforcement requirements. This approach would be consistent not only with the observed experimental centrifuge results, but also with the U.S. practice of using peak shear strength in the design of unreinforced slopes. In fact, the use of residual shear strength values in the design of geosynthetic reinforced slopes while still using peak shear strength in the design of unreinforced embankments could lead to illogical comparisons of alternatives for embankment design. For example, an unreinforced slope that satisfies stability criteria based on a factor of safety calculated using peak strength, would become unacceptable if reinforced using inclusions of small (or negligible, for the purposes of this example) tensile strength because stability would be evaluated in this case using residual soil shear strength values.

The writers agree with the discusser that the tension distribution among the reinforcements is also a function of conditions not represented in the centrifuge models, namely, foundation conditions. However, and even though further investigation should be pursued to identify the actual reinforcement tension distribution, it is apparent that a linear distribution (i.e., triangular with maximum tension toward the base of the slope) is not consistent with the centrifuge experimental results or with observations from monitored full-scale structures [e.g., Adib (1988) and Christopher et al. (1992)]. Consequently, a clear message should be conveyed to designers that the critical elevation in terms of reinforcement requirements is a function of the slope inclination and that, in contrast with the case of vertical walls, this critical zone may not be located toward the base of the structure.

Finally, the writers fully agree with the discusser in that the centrifuge study focused only on one of the several failure modes that should be contemplated in the design of geosynthetic reinforced slopes (i.e., internal failure due to the breakage of the reinforcements). As stated by the writers in the paper, the use of comparatively long reinforcements was deliberate since the focus of the study was on the evaluation of this one particular failure mode. External and compound failure mechanisms were not expected to develop during testing, but they should certainly be accounted for in design.

The input provided by Dantas and Ehrlich is appreciated. The writers are pleased to learn that the numerical simulations performed by the discussers validate the experimental results in relation to the reinforcement tension distribution with depth, the location of the potential rupture surface, and soil-reinforcement stress-strain compatibility. The discussers indicated concern that the centrifuge simulations do not reproduce compaction-induced stresses that may occur in actual prototype structures. The limitations of centrifuge modeling that lead to differences in the behavior between reinforced slope models and prototypes are extensively discussed by Zornberg et al. (1997a). As reported by the writers, compaction effects cannot be replicated in the model, which is constructed at 1 g prior to centrifuging. More importantly, while placement of a compacted soil layer in a prototype induces deformations on the layers underneath the one being placed, the preconstructed centrifuge model responds in its entirety as it is brought up to scale speed. However, the primary objective of the centrifuge investigation was validation of the analytical tool used for analysis and design of reinforced slopes (i.e., limit equilibrium), which is also blind to compaction-induced stresses. Numerical simulations presented by the discussers illustrate the effect of compaction-induced stresses in a metallic-reinforced slope under working stress conditions. The writers believe that the effect of compaction-induced stresses would be significantly smaller for the case of geosynthetic-reinforced slopes under failure conditions, which was the focus of the investigation performed by the writers.

Limit Equilibrium as Basis for Design of Geosynthetic Reinforced Slopes[a]

Discussion by Mauricio Ehrlich,[4] Member, ASCE, and Bruno T. Dantas[5]

Limit equilibrium methods have been extensively used as a basis for design of reinforced soil structures. This procedure has its limitations and may be considered as a simple approach. Only geostatic stress could be taken into consideration through these formulations. It has been shown that compaction-induced stresses may be the major contributor to reinforcement tension to depths of more than 6 m (Adib 1988; Ehrlich and Mitchell 1994; Dantas 1998).

It is the discussers' intention to provide some insights on the role of compaction-induced stresses on reinforced soil behavior and its importance for design. Compaction may be considered as a kind of soil preconsolidation and would lend a stiffer behavior to the reinforced soil. This mechanical improvement is due not only to reduction in soil void ratio, but also to the increase in the soil horizontal stress that generates a prestressed material.

Fig. 19 shows a simple qualitative approach based on Jewell's (1985) representation of the strain compatibility between soil and reinforcement in the soil surrounding the reinforcement at maximum tension point. The curve S1 is a generic representation of the soil stress-strain behavior under geostatic conditions.

A more complex situation occurs if compaction is taken into account. The actual multicycles stress path for soil placement and compaction during construction may be simplified by the assumption that each layer is subject to only one cycle of loading (Duncan and Seed 1986; Seed and Duncan 1986; Ehrlich and Mitchell 1994). Loading under K_0 conditions due to the weight of the overlying soil layers plus some equivalent

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