Technical Paper by J.G. Zornberg

PEAK VERSUS RESIDUAL SHEAR STRENGTH IN GEOSYNTHETIC-REINFORCED SOIL DESIGN

ABSTRACT: Current design guidelines for geosynthetic-reinforced soil structures disagree over the shear strength parameters that should be selected to characterize the backfill material. Most geosynthetic reinforcing materials are classified as extensible inclusions for almost all practical applications. The extensible nature of geosynthetic reinforcements has led to the recommendation by several agencies and reinforced soil designers toward the use of the residual shear strength instead of the peak shear strength for design. However, common practice in the US has been the use of the peak shear strength. The main purpose of this paper is to provide experimental evidence regarding selection of either peak or residual shear strength to characterize the backfill material for the design of geosynthetic-reinforced soil structures. Specifically, experimental results from reduced-scale models tested in a geotechnical centrifuge indicate that the stability of geosynthetic-reinforced slopes is governed by the peak soil shear strength.

KEYWORDS: Soil reinforcement, Shear strength, Centrifuge, Design.

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1 INTRODUCTION

The selection of the backfill shear strength properties in the design of geosynthetic-reinforced soil structures is an issue of major disagreement among design guidelines. Unlike steel reinforcements, which are considered inextensible inclusions for design purposes, most currently available geosynthetic reinforcing materials are classified as extensible inclusions for almost all practical applications. The extensible nature of geosynthetic reinforcements led to recommendations toward adopting the residual shear strength (or the friction angle at constant volume) for the design of reinforced soil structures (e.g., McGown et al. 1989, Jewell 1991). The rationale for this recommendation has been that the soil strength is expected to reach its peak before the reinforcements achieve their ultimate strength. However, common practice in the US has been the use of the peak shear strength for the design of geosynthetic-reinforced structures. This is reflected in the recent US Federal Highway Administration (FHWA) design guidelines (Elias et al. 2001).

The main purpose of the current paper is to provide additional basis for the selection of the backfill shear strength in the design of geosynthetic-reinforced soil structures. Even though use of the residual shear strength in design represents a conservative approach, this conservatism is not supported by the observed good performance of monitored reinforced soil structures. Full-scale monitored structures constructed with a factor of safety of unity have reached stress levels significantly below those considered in design (Christopher et al. 1992). In addition, experimental data presented herein support the recommendation of using peak shear strength in the design of reinforced soil slopes.

The current paper initially evaluates the current state-of-the practice regarding selection of backfill shear strength for reinforced soil design, as compiled by several proposed methods and design manuals. Next, experimental evidence is presented to assess the shear strength properties governing failure in a series of centrifuge tests on reduced-scale reinforced soil models. Finally, guidance is provided regarding selection of the shear strength properties for the design of geosynthetic-reinforced soil structures. The focus of the current paper is on the evaluation of internal stability of geosynthetic-reinforced soil structures, and the findings presented herein should not be extended to other failure mechanisms (e.g., direct sliding along soil-reinforcement interfaces) without careful consideration.

2 CURRENT GUIDELINES REGARDING SELECTION OF BACKFILL SHEAR STRENGTH

The use of inclusions to improve the mechanical properties of soils dates to ancient times. However, it is only within approximately the last three decades (Vidal 1969) that analytical and experimental studies have led to contemporary soil reinforcement techniques. Soil reinforcement is now a highly attractive alternative for embankment and retaining wall projects because of the economic benefits it offers in relation to conventional retaining structures. Moreover, its acceptance has also been triggered by a number of technical factors, which include aesthetics, reliability, simple construction techniques, good seismic performance, and the ability to tolerate large deformations without structural distress. The design of reinforced soil slopes is based on the use of limit equilibrium methods to evaluate both external (global) and internal stability. After adopting the shear strength properties of the backfill material, the required tensile strength of the reinforcements can be defined in the design so that the margin of safety is adequate.

Geosynthetics are classified as extensible reinforcements. Consequently, the soil strength may be expected to mobilize rapidly, reaching its peak strength before the reinforcements achieve their ultimate strength. This rationale has led to recommendations toward the adoption of the residual shear strength for the design of geosynthetic-reinforced slopes. This is the case of commonly used design methods such as those proposed by Jewell (1991) and Leshchinsky and Boedeker (1989). Several agencies have endorsed the use of residual shear strength parameters in the design of reinforced soil structures, as summarized in Table 1. Zornberg and Leshchinsky (2001) present a review of current design criteria used by different agencies for geosynthetic-reinforced walls, geosynthetic-reinforced slopes, and embankments over soft soils.

Table 1. Summary of guidelines on selection of soil shear strength parameters for geosynthetic-reinforced soil design.

Method/Agency	Shear strength parameters	Reference
Jewell's method	Residual	Jewell (1991)
Leshchinsky and Boedeker's method	Residual	Leshchinsky and Boedeker (1989)
Queensland DOT, Australia	Residual	RTA (1997)
New South Wells, Australia	Residual	QMRD (1997)
Bureau National Sols-Routes (draft French Standard)	Residual	Gourc et al. (2001)
Federal Highway Administration (FHWA), AASHTO	Peak	Elias et al. (2001) AASHTO (2002)
National Concrete Masonry Association	Peak	NCMA (1997, 1998)
GeoRio, Brazil	Peak	GeoRio (1989)
Canadian Geotechnical Society	Peak	Canadian Geotechnical Society (1992)
German Society of Soil Mechanics and Geotechnical Engineering	Peak	EBGEO (1997)
Geotechnical Engineering Office, Hong Kong	Peak	GCO (1989), GEO (1993)
Public Works Research Center, Japan	Peak	Public Works Research Center (2000)
British Standards, United Kingdom	Peak	British Standard Institution (1995)
Leshchinsky's hybrid method	Hybrid	Leshchinsky (2001)

The use of the peak friction angle has been common practice in the US for the design of geosynthetic-reinforced slopes. Guidance in soil reinforcement design procedures has been compiled by several federal agencies in the US, including the American Association of State Highway and Transportation Officials (AASHTO 2002) and the Federal Highway Administration (Elias et al. 2001). Design guidance is also provided by the National Concrete Masonry Association (NCMA 1997), possibly the only industry manual of soil reinforcement practice. The above mentioned design guidance manuals recommend the use of the peak friction angle in the limit equilibrium analyses. Other agencies that have also endorsed the use of peak shear strength parameters in the design of reinforced soil structures are summarized in Table 1.

A hybrid approach was recently proposed by Leshchinsky (2000, 2001). Central to his approach is the use of a 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.

To address the controversial issue regarding selection of shear strength properties in reinforced soil design, the current paper presents experimental evidence on failed reinforced slopes. Specifically, the experimental information obtained from centrifuge modeling supports the use of peak shear strength parameters in the design of geosynthetic-reinforced soil structures. The perceived conservatism in design is also not supported by the generally observed good performance of monitored reinforced soil structures.

3 OVERVIEW OF CENTRIFUGE TESTING PROGRAM

Limit equilibrium analysis methods have been traditionally used to analyze the stability of slopes with and without reinforcements. However, to date, limit equilibrium predictions of the performance of geosynthetic-reinforced slopes have not been fully validated against monitored failures. This has led to a perceived overconservatism in their design. Consequently, an investigation was undertaken to evaluate design assumptions for geosynthetic-reinforced slopes (Zornberg et al. 1998a, 2000). The results of centrifuge tests provide an excellent opportunity to examine the validity of various assumptions typically made in the analysis and design of reinforced soil slopes. The current paper presents the results of these centrifuge tests, which evaluate the shear strength properties governing failure of reinforced soil slopes.

All reinforced slope models in the experimental testing program had the same geometry and were built within the same strong box. A transparent Plexiglas plate was used on one side of the box to enable a side view of the models during testing. The other walls of the box were aluminum plates lined with Teflon to minimize side friction. The overall dimensions of the geotextile-reinforced slope models are as shown in Figure 1 for a model with nine reinforcement layers. Displacement transducers are also indicated in Figure 1.

The number of reinforcement layers in the models ranged from six to eighteen, resulting in a reinforcement spacing ranging from 37.5 to 12.5 mm. All models used

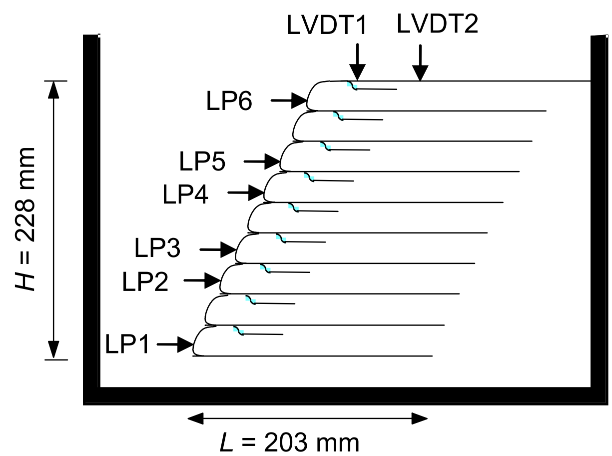

Figure 1. Typical centrifuge model.

the same reinforcement length of 203 mm. The use of reasonably long reinforcement lengths was deliberate, since the current study focused on the evaluation of internal stability against breakage of the geotextile reinforcements. In this way, external or compound failure surfaces were not expected to develop during testing. As shown in Figure 1, the geotextile layers were wrapped at the slope facing in all models. Green colored sand was placed along the Plexiglas wall at the level of each reinforcement layer to identify the failure surface. In addition, black colored sand markers were placed at a regular horizontal spacing (25 mm) to monitor lateral displacements within the backfill material.

The variables investigated in the current study were selected so that they could be taken into account in a limit equilibrium framework. Accordingly, the selected variables were as follows:

1. vertical spacing of the geotextile reinforcements (four different reinforcement spacings were adopted);
2. soil shear strength parameters (the same sand at two different relative densities was used); and
3. ultimate tensile strength of the reinforcements (two geotextiles with different ultimate tensile strength were selected).

Of particular relevance, for the purpose of the issues addressed in the current paper, is the fact that the same sand placed at two different relative densities was used as backfill material for the centrifuge models. The backfill material at these two relative densities has different peak shear strength values, but the same residual shear strength.

The model slopes were built using Monterey No. 30 sand, which is a clean, uniformly graded sand classified as SP in the Unified Soil Classification System (Zornberg et al. 1998b). The particles are rounded to subrounded, consisting predominantly of quartz with a smaller amount of feldspars and other minerals. The average particle size for the material is 0.4 mm, the coefficient of uniformity is 1.3, and the coefficient of curvature is approximately 1.1. The maximum and minimum void ratios of the sand are 0.83 and 0.53, respectively. To obtain the target dry densities in the model slopes, the sand was pluviated through air at controlled combinations of sand discharge rate and discharge height. The unit weights for the Monterey No. 30 sand at the target relative densities of 55 and 75% are 15.64 and 16.21 kN/m^3, respectively.

Two series of triaxial tests were performed to evaluate the friction angle for the Monterey No. 30 sand as a function of relative density and confining pressure. The tests were performed using a modified form of the automated triaxial testing system developed by Li et al. (1988). The specimens had nominal dimensions of 70 mm in diameter and 150 mm in height and were prepared by dry tamping. Figure 2 shows the

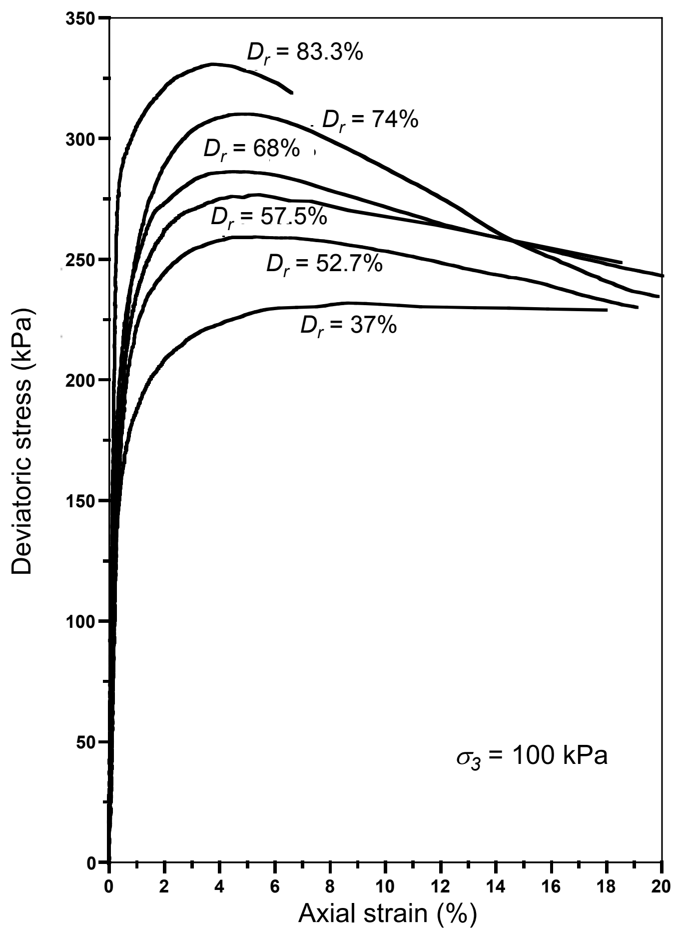

Figure 2. Stress strain behavior of Monterey No. 30 sand pluviated at different relative densities (D_r) and tested in triaxial compression under the same confinement.

stress strain response obtained from the series of tests conducted to evaluate the behavior of Monterey No. 30 sand as a function of relative density. All tests shown in Figure 2 were conducted using a confining pressure of 100 kPa. As can be observed in Figure 2, while the sand shows a different peak shear strength for different relative densities, the shear stress tends to a single residual shear strength for large strain conditions. Figure 3 shows the increase in peak friction angle with increasing relative density at a confining pressure of 100 kPa. Of particular interest are the friction angles obtained at relative densities of 55 and 75%, which correspond to the relative density of the backfill material in the models. The estimated triaxial compression friction angles (ϕ_{tc}) at these relative densities are 35° and 37.5°, respectively. Although the tests did not achieve strain values large enough to guarantee a critical state condition, the friction angles at large strains appear to converge to a residual value (ϕ_r) of approximately 32.5°. This value agrees with the critical state friction angle for Monterey No. 0 sand obtained by Riemer (1992). As the residual friction angle is mainly a function of mineralogy (Bolton 1986), Monterey No. 0 and Monterey No. 30 sands should show similar ϕ_r values. The effect of confining pressure on the frictional strength of the sand was also evaluated. The results showed that the friction angle of Monterey No. 30 decreases only slightly with increasing confinement. The fact that the friction angle of this sand does not exhibit normal stress dependency avoids additional complications in the interpretation of the centrifuge model tests.

Scale requirements for the reinforcing material establish that the reinforcement tensile strength in the models should be reduced by N. That is, an Nth-scale reinforced slope model should be built using planar reinforcement having $1/N$ the strength of the prototype reinforcement elements (Zornberg et al. 1998a). Two types of nonwoven

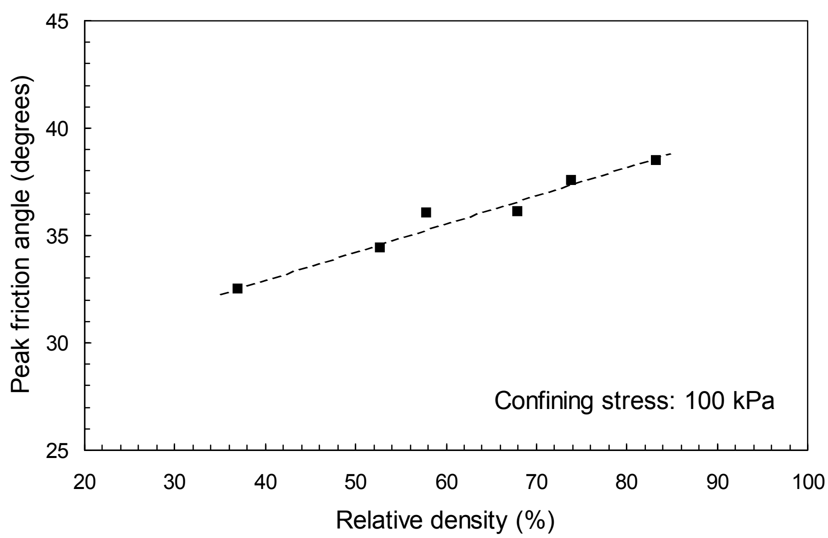

Figure 3. Friction angle for Monterey No. 30 sand obtained from triaxial testing at different relative densities.

interfacing fabrics, having mass per unit area values of 24.5 and 28 g/m^2, were selected as reinforcement. Unconfined ultimate tensile strength values, measured from wide-width strip tensile tests (ASTM D 4595), were 0.063 and 0.119 kN/m for the weaker and stronger geotextiles, respectively. Confined tensile strength values, obtained from back-calculation of failure in the centrifuge slope models, were 0.123 and 0.183 kN/m for the weaker and stronger geotextiles, respectively (Zornberg et al. 1998b). Confined tensile strength values were used for estimating the factor of safety of the models analyzed in the current study under increasing g-levels.

4 TYPICAL CENTRIFUGE TEST RESULTS

The models were subjected to a progressively increasing centrifugal acceleration until failure occurred. A detailed description of the characteristics of the centrifuge testing program is presented by Zornberg et al. (1998a). The centrifuge tests can be grouped into three test series (B, D, or S). Accordingly, each reinforced slope model in the current study was named using a letter that identifies the test series, followed by the number of reinforcement layers in the model. Each test series investigated the effect of one variable, as follows:

1. Baseline, B-series: performed to investigate the effect of the reinforcement vertical spacing.
2. Denser soil, D-series: performed to investigate the effect of the soil shear strength on the stability of geosynthetic-reinforced slopes. The models in this series were built with a denser backfill sand but with the same reinforcement type as in the B-series.
3. Stronger geotextile, S-series: performed to investigate the effect of the reinforcement tensile strength on the performance of reinforced slopes. The models in this series were built using reinforcements with a higher tensile strength than in the B-series, but with the same backfill density as in that series.

The history of centrifugal acceleration during centrifuge testing of one of the models is indicated in Figure 4. In this particular test, the acceleration was increased until sudden failure occurred after approximately 50 minutes of testing when the acceleration imparted to the model was 76.5 times the acceleration of gravity. Settlements at the crest of the slope, monitored by LVDTs, were useful in accurately identifying the moment of failure. Figure 5 shows the increasing settlements at the top of a reinforced slope model during centrifuge testing. The sudden increase in the monitored settlements indicates the moment of failure when the reinforced active wedge slid along the failure surface. Figure 6 shows a typical failure surface as developed in the centrifuge models. As can be seen, the failure surface is well defined and goes through the toe of the reinforced slope.

Following the test, each model was carefully disassembled in order to examine the tears in the geotextile layers. Figure 7 shows the geotextiles retrieved after centrifuge testing of a model reinforced with 18 geotextile layers. The geotextile at the top left corner of Figure 7 is the reinforcement layer retrieved from the base of the model. The

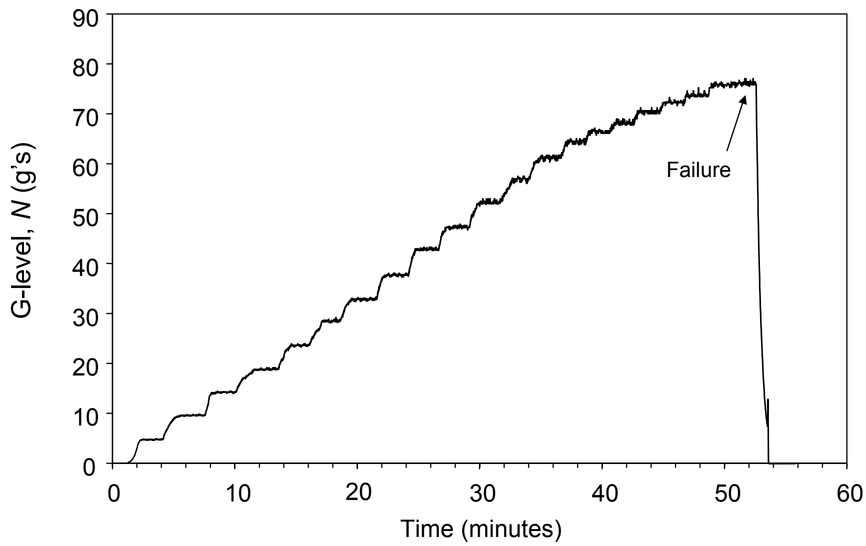

Figure 4. **G-level, *N*, versus time during centrifuge testing.**

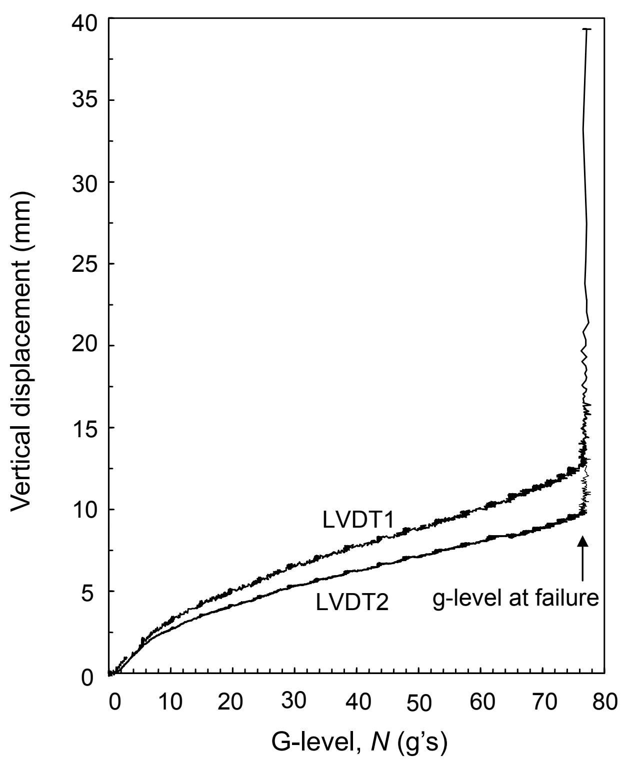

Figure 5. **Settlements at the crest of a reinforced slope model.**

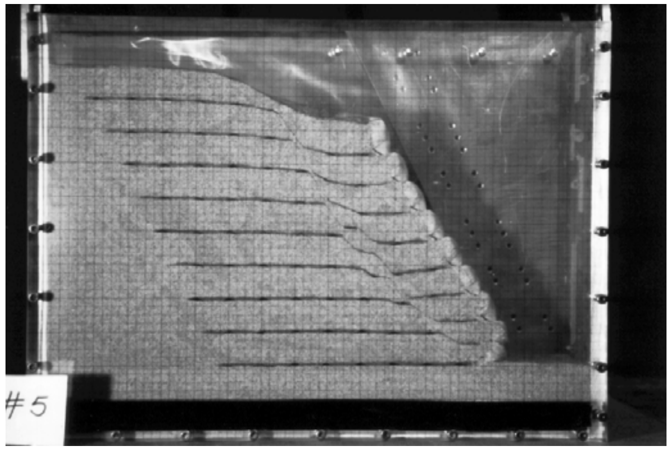

Figure 6. Failed geotextile-reinforced slope model.

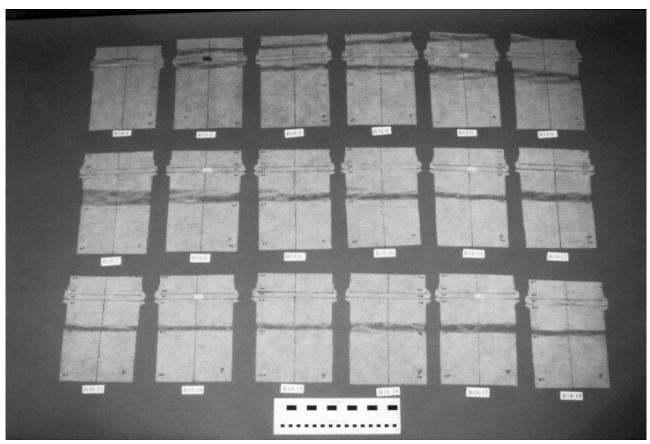

Figure 7. Geotextile reinforcements retrieved after testing.

geotextile at the bottom right corner is the reinforcement retrieved from the top of the model. All retrieved geotextiles show clear tears at the location of the failure surface. The pattern observed from the retrieved geotextiles shows that internal failure occurred when the tensile strength on the reinforcements was achieved. The geotextile layers located toward the base of the slope model also showed breakage of the geotextile overlaps, which clearly contributed to the stability of the slope. No evidence of pullout was observed, even on the short overlapping layers.

5 EFFECT OF BACKFILL SHEAR STRENGTH ON THE EXPERIMENTAL RESULTS

The criteria for characterizing reinforcements as extensible or inextensible has been established by comparing the horizontal strain in an element of reinforced backfill soil subjected to a given load, to the strain required to develop an active plastic state in an element of the same soil without reinforcement (Bonaparte and Schmertmann 1987). Accordingly, reinforcements have been typically classified as follows:

1. extensible, if the tensile strain at failure in the reinforcement exceeds the horizontal extension required to develop an active plastic state in the soil; or as
2. inextensible, if the tensile strain at failure in the reinforcement is significantly less than the horizontal extension required to develop an active plastic state in the soil.

The geotextiles used to reinforce the centrifuge model slopes are extensible reinforcements. The effect of reinforcement spacing on the stability of the reinforced slope models, as indicated by the measured g-level at failure N_f, is shown in Figure 8. The num-

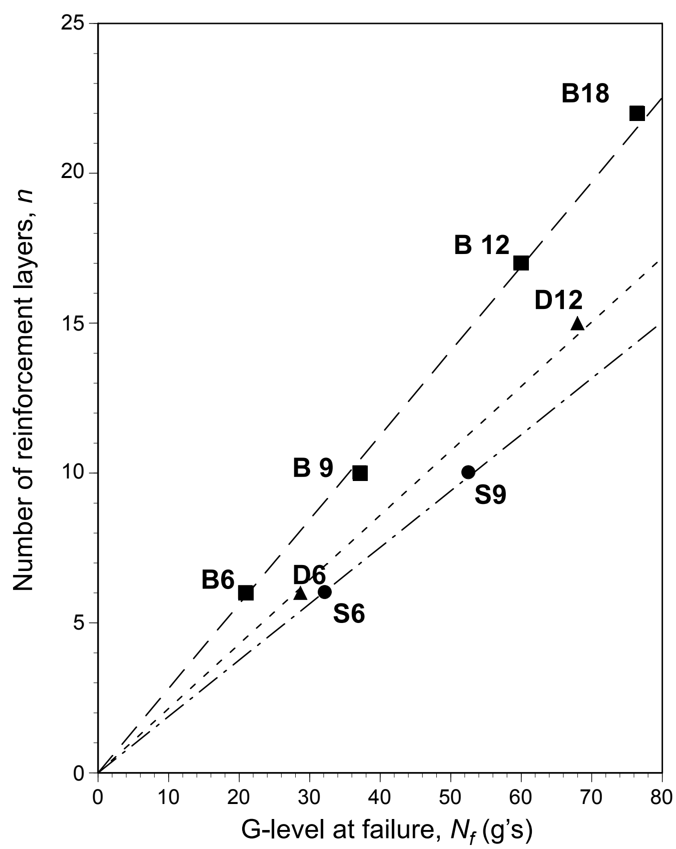

Figure 8. G-level at failure for the centrifuge models.

ber of reinforcement layers n in Figure 8 includes the total number of model geotextiles intersected by the failure surface (i.e., primary reinforcements and overlaps intersected by the failure surface). The overlaps intersected by the failure surface developed tensile forces and eventually failed by breakage and not by pullout. Figure 8 shows that a well-defined linear relationship can be established between the number of reinforcement layers and the g-level at failure. As the fitted lines for each test series passes through the origin, the results in each test series can be characterized by a single n/N_f ratio.

Models in the B- and D-series were reinforced using the same geotextile reinforcement, but using sand backfill placed at two different relative densities (55% and 75%). As mentioned, the Monterey sand at these two relative densities has the same soil residual friction angle (32.5°) but different peak friction angles (35° and 37.5°). As shown in Figure 8, models in the D-series failed at higher g-levels than models in the B-series built with the same reinforcement spacing and reinforcement type. Since the backfill soil in models from the D- and B-series have the same residual soil shear strength, the higher g-level at failure in the D-series models is due to the higher peak soil shear strength in this test series.

Analysis of the data presented in Figure 8 emphasizes that the use of a single residual shear strength value, common to the two backfill materials used in the test series, cannot explain the experimental results. Instead, the experimental results can be explained by acknowledging that the stability models constructed with the same reinforcement layout and the same sand backfill, but placed at different densities, is governed by different shear strength values. Indeed, limit equilibrium analyses (Zornberg et al. 1998b) indicated that the shear strength value that should be used in the analysis of these slope failures is the plain strain peak shear strength of the backfill.

The experimental results indicate that the stability of structures with extensible reinforcements is governed by the peak shear strength and not by the residual shear strength of the backfill soil. A plausible explanation of these experimental results is that, although the soil shear strength may have been fully mobilized along certain active failure planes within the reinforced soil mass, shear displacements have not taken place along these failure surfaces. That is, although the soil may have reached active state due to large horizontal strains because of the extensible nature of the reinforcements, large shear displacements (and drop from peak to residual soil shear strength) only take place along the failure surface during final sliding of the active reinforced wedge (Zornberg et al. 1998b).

An additional way of evaluating these experimental results is by using dimensionless coefficients, which have been used to develop design charts for geosynthetic-reinforced soil slopes (Schmertmann et al. 1987; Leshchinsky and Boedeker 1989; Jewell 1991). The validity of the proposed normalization can be investigated from the centrifuge results of the current study. For a reinforced slope model that failed at an acceleration equal to N_f times the acceleration of gravity, a dimensionless coefficient K can be estimated as follows:

$$K(\phi, \beta) = n\,T_{ult}\left(\frac{2}{\gamma H^2}\right)\frac{1}{N_f} \tag{1}$$

where: n = number of reinforcements; T_{ult} = reinforcement tensile strength; H = slope height; N_f = g-level at failure from the centrifuge test; ϕ = soil friction; β = slope inclination; and γ = sand unit weight. The value of n used in Equation 1 includes the number of overlaps that were intersected by the failure surface in the centrifuge slope models in addition to the number of primary reinforcement layers. The coefficient K is a function of the soil shear strength and the slope inclination, i.e., $K = K(\phi,\beta)$. All centrifuge slope models were built with the same slope inclination β. Consequently, validation of the suggested normalization requires that a single coefficient $K(\phi,\beta)$ be obtained for all models built with the same backfill. If the soil shear strength governing failure of the models is the residual strength, a single coefficient $K(\phi,\beta)$ should be obtained for all models. On the other hand, if the soil shear strength governing failure is the peak shear strength, a single coefficient should be obtained for those models built with sand placed at the same relative density.

Figure 9 shows the centrifuge results in terms of $(n\ T_{ult})\ (2/\gamma H^2)$ versus the g-level

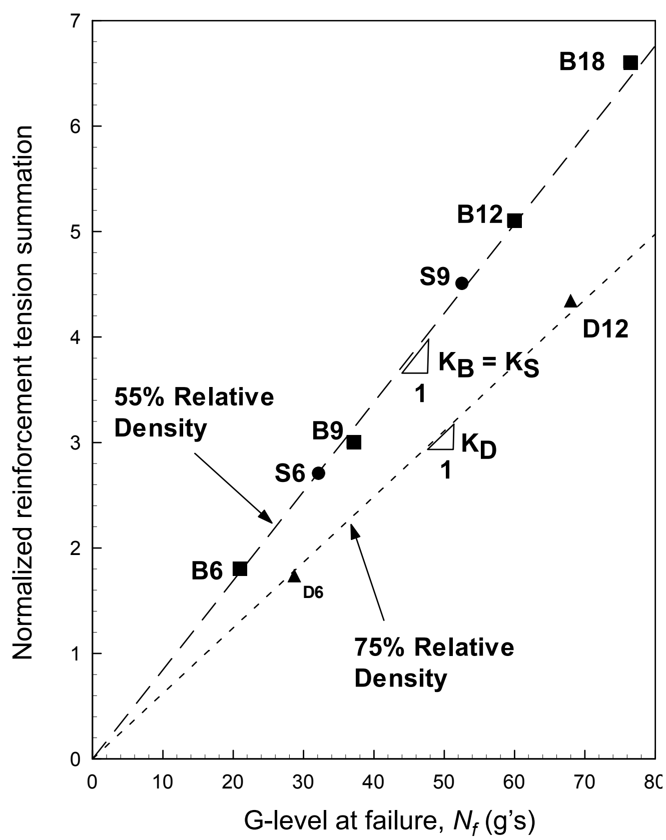

Figure 9. Normalized reinforcement tension summation values from centrifuge test results.

at failure N_f. The results in Figure 9 show that a linear relationship can be established for those models built with sand placed at the same relative density. As inferred from Equation 1, the slope of the fitted line corresponds to the dimensionless reinforcement tension summation coefficient $K = K(\phi,\beta)$. The results obtained using the centrifuge models from the B- and S-series, built using Monterey sand placed at 55% relative density, define a normalized coefficient $K(\phi,\beta) = K_B = K_S = 0.084$. Similarly, centrifuge results from the D-series models, built using Monterey sand at 75% relative density, define a normalized coefficient $K(\phi,\beta) = K_D = 0.062$. These results provide sound experimental evidence supporting the use of charts based on normalized coefficients for preliminary design of geosynthetic-reinforced slopes. If failure of reinforced soil slopes were governed by the residual soil shear strength, the results of all centrifuge tests should have defined a single line. However, as can be observed in Figure 9, different normalized coefficients are obtained for different soil densities. This confirms that the normalization should be based on the peak shear strength and not on the residual shear strength of the backfill material.

6 CONCLUSIONS AND RECOMMENDATIONS

The selection of the backfill shear strength properties in the design of geosynthetic-reinforced soil structures is an issue over which design guidelines disagree. The main debate has been over whether the peak or the residual shear strength of the backfill material should be adopted for design. 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 main purpose of the current investigation was to provide experimental evidence addressing this presently unsettled issue.

The experimental results presented herein indicate that the soil shear strength governing the stability of geosynthetic-reinforced soil slopes is the peak shear strength. A centrifuge experimental testing program was undertaken which indicated that reinforced slopes constructed with the same reinforcement layout and the same backfill sand, but using different sand densities, failed at different centrifuge accelerations. That is, nominally identical models built with backfill material having the same residual shear strength, but different peak shear strength, did not have the same factor of safety. Since the residual shear strength of the sand backfill is independent of the relative density, these results indicate that the soil shear strength governing stability is the peak shear strength of the backfill material.

Several design guidance manuals have implicitly recommended the selection of the peak shear strength for the design of reinforced soil slopes. Considering the current debate over the selection of the soil shear strength in design and the experimental results presented herein, design manuals should explicitly endorse selection of peak shear strength values for the design of reinforced soil structures. This approach would not only be consistent with the observed experimental centrifuge results, but also with the US practice of using peak shear strength in the design of unreinforced slopes.

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NOTATIONS

Basic SI units are in parentheses.

H = slope height (m)

K = normalized coefficient (dimensionless)

K_B = normalized coefficient for test series B (dimensionless)

K_D = normalized coefficient for test series D (dimensionless)

K_S = normalized coefficient for test series S (dimensionless)

N = scale factor (dimensionless)

N_f = g-level at failure from the centrifuge test (dimensionless)

n = number of reinforcements (dimensionless)

T_{ult} = reinforcement tensile strength (N/m)

β	=	slope inclination (degrees)
ϕ	=	soil friction angle (degrees)
ϕ_r	=	residual soil friction angle (degrees)
ϕ_{tc}	=	soil friction angle from triaxial compression test (degrees)
γ	=	soil unit weight (N/m^3)

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