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Failure mechanisms in geosynthetically reinforced soil slopes

Mécanismes de rupture des pentes renforcées avec géosynthétiques

Jorge G. Zornberg – *GeoSyntec Consultants, Huntington Beach, Calif., USA*
Nicholas Sitar – *University of California, Berkeley, Calif., USA*
James K. Mitchell – *Virginia Tech, Blacksburg, Va., USA*

SYNOPSIS: Centrifuge tests were used to investigate the failure mechanisms of geosynthetically reinforced steep soil slopes and to evaluate the assumptions in their design. The moment of failure was defined by a sudden change in the rate of settlements at the crest of the slope, as monitored from transducers placed on top of the centrifuge models. Failure in the models was characterized by well-defined curved shear surfaces through the toe of the slope, which is in good agreement with current design methods for reinforced slopes based on limit equilibrium. However, in contrast to the current design assumptions that failure should develop from the toe of the reinforced slopes, failure was observed to initiate at midheight of the slopes. Consistent with the experimental observations, a new distribution of reinforcement forces with depth is proposed for the design of geosynthetically reinforced soil slopes.

1 INTRODUCTION

A wide range of geotechnical problems can be investigated using centrifuge physical modeling techniques. Advances in this field have been documented in the proceedings of conferences organized under the auspices of the ISSMFE (Corte, 1988; Ko and McLean, 1991; Leung et al., 1994) and elsewhere. Evaluation of the behavior of reinforced soil structures is no exception. Of particular interest is the investigation of the performance of geosynthetically reinforced soil slopes at failure, as failure is the condition for which reinforced slopes are conventionally designed using a limit equilibrium approach.

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 geosynthetically reinforced slopes have not been fully validated against monitored failures. This has led to a perceived overconservatism in their design. Consequently, an investigation was initiated to evaluate design assumptions for geosynthetically reinforced slopes (Zornberg, 1994; Zornberg et al., 1995). The results of the centrifuge tests provided an excellent opportunity to examine the validity of various assumptions typically made in the analysis and design of reinforced soil slopes. This paper presents the aspects of that study aimed at the evaluation of the failure mechanisms.

2 CENTRIFUGE TESTING OF REINFORCED SOIL STRUCTURES

Centrifuge modeling is based upon the requirement of similarity between the model and the prototype. If a model of the prototype structure is built with dimensions reduced by a factor $1/N$, then an acceleration field of N times the acceleration of gravity, g, will generate stresses by self-weight in the model that are the same as those in the prototype structure. Additional scaling relationships can be determined either by analysis of governing differential equations or by dimensional analysis and the theory of models.

The scaling laws governing the behavior of cohesionless reinforced soil slopes at failure were developed by assuming the validity of limit equilibrium. Specifically, similitude requirements which guarantee identical factors of safety in the model and the prototype structures lead to the conclusion that same soil density and soil friction angle should be used in the model and in the prototype. These conditions can be satisfied by building the model using the same backfill soil used in the prototype structure. However, the scaling factor for the reinforcement tensile strength should be equal to $1/N$, where N is model scale. That is, an Nth-scale reinforced slope model should be built using planar reinforcements N times weaker than the prototype reinforcement elements (Zornberg et al., 1995).

3 CHARACTERISTICS OF THE CENTRIFUGE MODELS

All reinforced slope models in this experimental testing program had the same geometry and were built within the same strong box. A transparent plexiglass plate was used on one side of the box to enable side viewing of the model 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 the case of a model with nine reinforcement layers. The displacement transducers are also indicated in the figure.

The number of reinforcement layers in the models varied from six to eighteen, giving reinforcement spacings from 37.5 mm to 12.5 mm. All models used the same reinforcement length of 203 mm. The use of a reasonably long reinforcement length was deliberate, since this study focused on the evaluation of internal stability against breakage of the geotextile reinforcements. In this way, external or compound failure surfaces were expected not to develop during testing. As shown in the figure, the geotextile layers were wrapped at the slope face in all models. Green colored sand was placed along the plexiglass wall at the level of each reinforcement in order to identify the failure surface. Moreover, black colored sand markers were placed at a regular horizontal spacing (25 mm) in order to monitor lateral displacements within the backfill material.

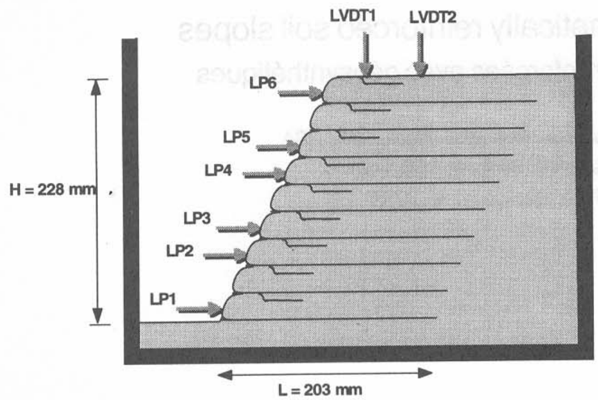

Figure 1. Typical centrifuge model

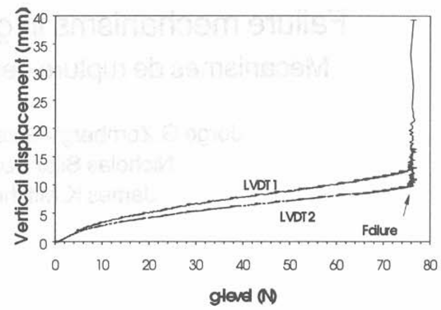

Figure 3. Settlements at the crest of a reinforced slope model

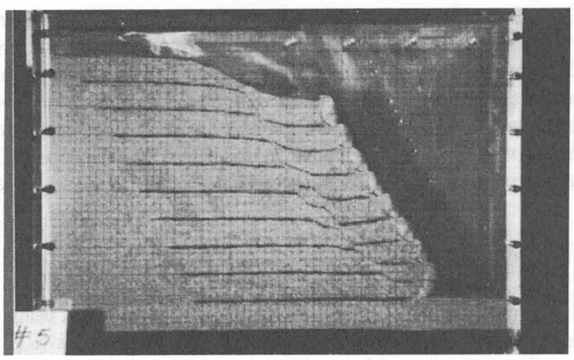

Figure 4. Failed geotextile-reinforced slope model

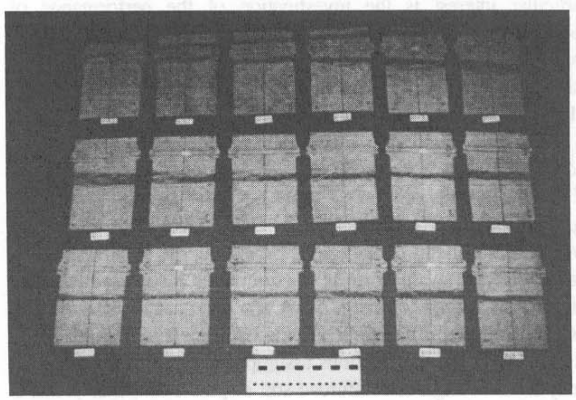

Figure 5. Geotextile reinforcements retrieved after testing

The models were subjected to a progressively increasing centrifugal acceleration until failure occurred. After failure, the backfill was carefully vacuumed out and the geotextile reinforcements were retrieved. The locations of the tears in the retrieved geotextiles was used to identify the failure surface.

The variables investigated in this study were selected so that they can be taken into account in a limit equilibrium framework. Accordingly, the selected variables were:

- Vertical spacing of the geotextile reinforcements: four different reinforcement spacings were adopted;
- Soil shear strength parameters: the same sand at two different relative densities was used; and
- Ultimate tensile strength of the reinforcements: two geotextiles with different ultimate tensile strength were selected.

4 TYPICAL CENTRIFUGE TEST RESULTS

The history of centrifugal acceleration during centrifuge testing of one of the models is indicated in Figure 2. In this particular test, the acceleration was increased until sudden failure occurred after approximately 50 min 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, proved to be invaluable to accurately identify the moment of failure. Figure 3 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 4 shows the typical failure surface that developed in the centrifuge models. As can be seen, the failure surface is well-defined and goes through the toe of the reinforced slope.

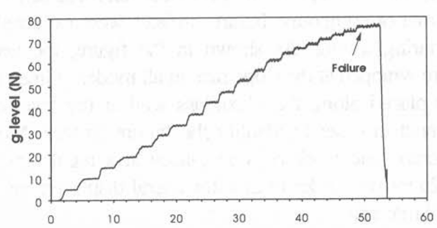

Figure 2. G-level (N) versus time during centrifuge testing

Following the test, each model was carefully disassembled in order to examine the breaks in the geotextile layers. Figure 5 shows the eighteen geotextiles retrieved after centrifuge testing a model reinforced with eighteen geotextile layers. The geotextile at the top left corner is the reinforcement layer retrieved from the base of the model. The geotextile at the bottom right corner is the reinforcement retrieved from the top of the model. All retrieved geotextiles show clear breaks 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 towards 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 DEVELOPMENT OF THE FAILURE SURFACES

Figure 6 shows an in-flight view of one of the models (model B12) at the moment of failure initiation during centrifuge testing. This image corresponds to the moment of failure defined by the transducers that monitored the settlements at the crest of the model. The image was recorded using a TV camera located inside the centrifuge. As can be observed in the figure, the initiation of failure occurs approximately in the middle of the slope. This can be observed by the kinks in the horizontal colored sand layers placed during construction at the levels of the reinforcements. Although kinks initially appeared at approximately the midheight of the slope, failure rapidly extended to the layers at the upper half of the model. The lower geotextile layers showed no evidence of kinking until the moment of ultimate structure collapse. The development of failure observed in the centrifuge slope models indicates that the lower reinforcement layers were not the ones to first reach failure. This contradicts the assumed distribution of reinforcement forces that is generally used in the design of reinforced soil slopes. The same pattern of behavior was observed for all the reinforced soil models.

Figure 7 shows the final collapse of model slope B12 as recorded in-flight during centrifuge testing. There was a very good agreement between the location of the failure surfaces obtained from the images on the plexiglass wall and from the measurements of the geotextile reinforcement tears for all the tested models.

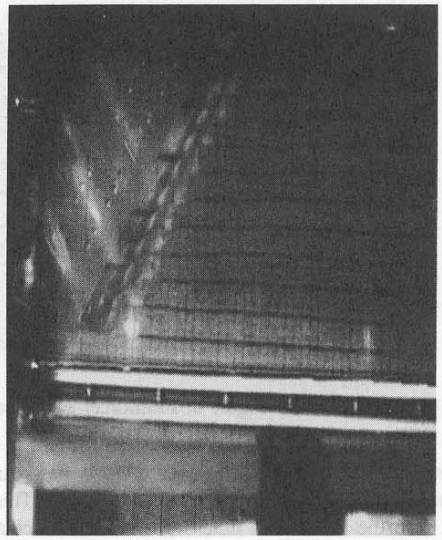

Figure 6. Initiation of failure in Model B12

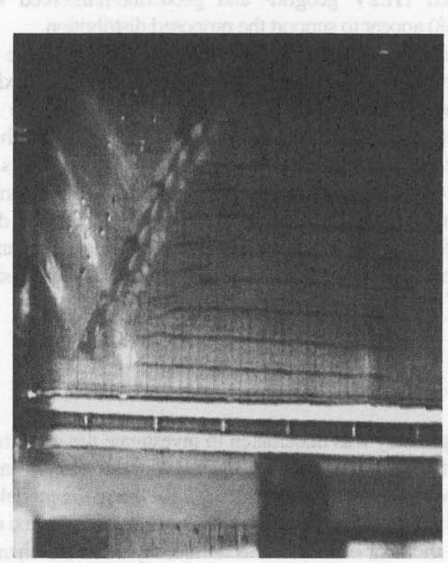

Figure 7. Final collapse of Model B12

6 INTERPRETATION OF THE FAILURE MECHANISMS

Interpretation of the failure mechanisms in reinforced soil slopes depends on a correct evaluation of the distribution of reinforcement forces with depth. From this distribution, the location of the first reinforcement that achieves its ultimate tensile strength can be identified. Current design methods for reinforced soil walls are based on assuming that reinforcement forces are proportional to the overburden pressure from the top of the wall. The rationale behind this assumption is that reinforcements should resist the active earth pressure (Mitchell and Christopher, 1990).

In the case of reinforced soil slopes, which have their design based on limit equilibrium and not on working stress methodologies, the reinforcement force distribution with depth must also be assumed. Extending observations gathered for the case of reinforced soil walls, triangular reinforcement tension distribution increasing proportionally with the depth below the slope crest has been conventionally assumed for reinforced soil slopes (e.g. Leshchinsky and Boedeker, 1989; Christopher et al., 1990; Jewell, 1991).

However, since failure of the models initiated at midheight of the slope, the conventional triangular distribution of reinforcement forces with depth is not supported by the experimental results obtained in this study. This has major implications for design, since vertical spacing and ultimate strength of the reinforcements are currently selected considering that the most critical zone is at the base of the structure.

It is reasonable to assume that the reinforcements resist the horizontal stresses in the soil *at the location of the potential failure surface*. In the case of vertical reinforced soil walls, horizontal soil stresses along the potential failure surface are proportional to the overburden pressure which increases with depth below the top of the wall. In the case of reinforced soil slopes, horizontal soil stresses along the potential failure surface are approximately proportional to the overburden pressure, but they increase with the depth below the *slope face*.

Note that the conventional triangular distribution in reinforced slopes has been obtained by considering the overburden pressure to increase proportionally with depth below the *slope crest*. Figure 8 shows a reinforced soil slope with the two reinforcement tension distributions under discussion. The triangular distribution is obtained assuming that the reinforcement forces are proportional to the overburden pressure calculated using the depth z below the slope crest. The alternative distribution is estimated proportionally to the depth z^* below the slope face. The difference between z and z^* is indicated in the figure for point A along the potential failure surface. The proposed distribution is for a 1H:2V reinforced slope, that corresponds to the geometry of the centrifuge models in this study. As indicated in the figure, the location of the maximum force in the reinforcements is at a height h_P from the base of the slope. This height is determined by the location of the point P, which is the point along the potential failure surface directly below the slope crest. Above h_P, the reinforcement tension distribution increases proportionally with depth below the slope crest ($z = z^*$ in this case), while below h_P the reinforcement tension decreases, being proportional to z^* and becoming zero at the toe of the slope.

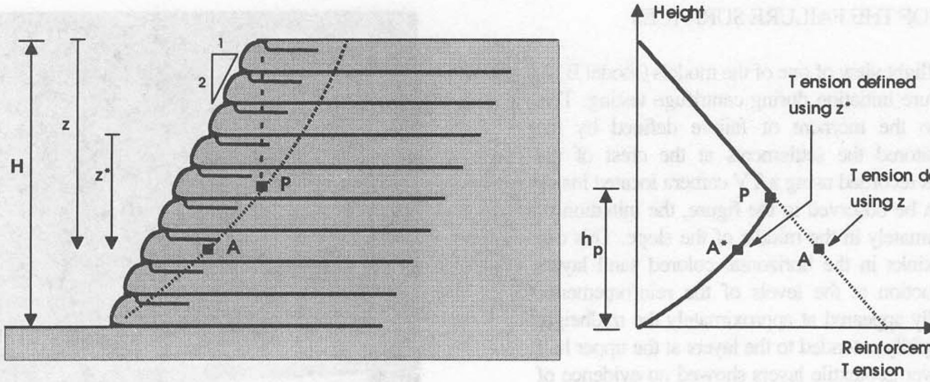

Figure 8. Reinforcement force distribution with depth for reinforced soil slopes

In the case of a slope inclined at 1H:2V, h_P is approximately equal to one half of the total height H of the slope. This is in agreement with the location of failure initiation in all centrifuge models in this study. Moreover, the distribution of maximum reinforcement tension with depth measured from well instrumented 1H:2V geogrid- and geotextile-reinforced slopes (Adib, 1988) appear to support the proposed distribution.

In a general case, the height h_P will mostly depend on the angle of the slope face. However, the location of the maximum reinforcement force will also depend on the soil friction angle since the location of the failure surface also depends on the soil shear strength. For the particular case of vertical slopes (i.e., reinforced walls), the point P will be at the toe of the structure and, consequently, $h_P = 0$. This is in agreement with current design methods for reinforced soil walls that consider a triangular distribution of the reinforcement forces with maximum tension at the base of the structure.

7 CONCLUSIONS

A centrifuge study was undertaken to investigate the performance of geosynthetically reinforced steep soil slopes at failure and to evaluate the assumptions in their design. Failure in the models was characterized by well-defined shear surfaces through the toe of the slope. The moment of failure was defined by a sudden change in the rate of settlements at the crest of the slope, as monitored from transducers placed on top of the centrifuge models.

In contrast to the current design assumptions that failure should develop from the toe of the reinforced slopes, failure of all centrifuge slope models was observed to initiate at midheight of the slopes. Consistent with these experimental observations, a new distribution of reinforcement forces with depth is proposed for the design of geosynthetically reinforced soil slopes.

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