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

[a]August 1998, Vol. 124, No. 8, by Jorge G. Zornberg, Nicholas Sitar, and James K. Mitchell (Paper 14817).

[4]Assoc. Prof., COPPE, Fed. Univ. of Rio de Janeiro, Cx. P.: 68506, RJ 21945-970, Brazil.
[5]Grad. Student, COPPE, Fed. Univ. of Rio de Janeiro, Brazil.

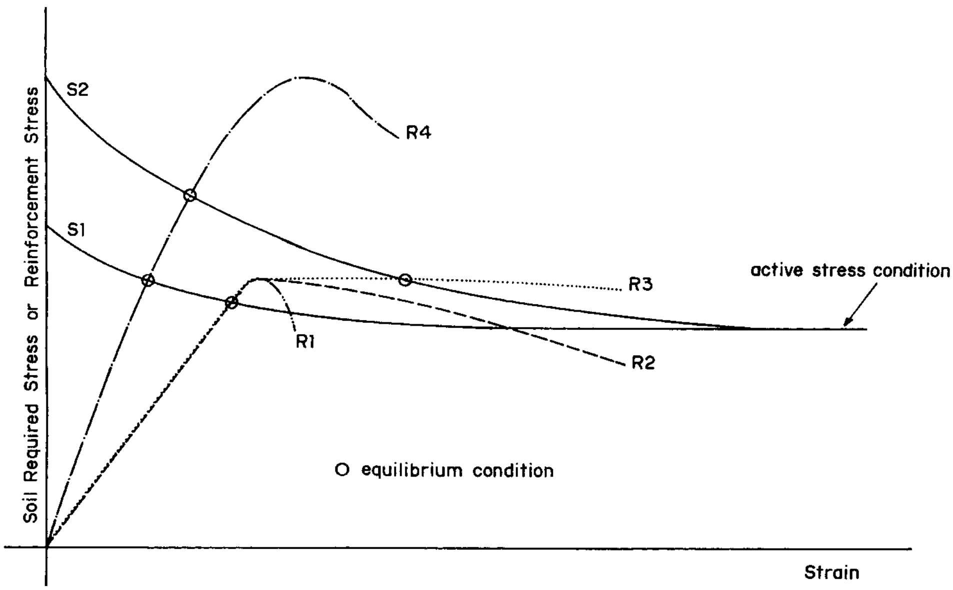

FIG. 19. Soil and Reinforcement Compatibility Curves, Including Compaction-Induced Stresses

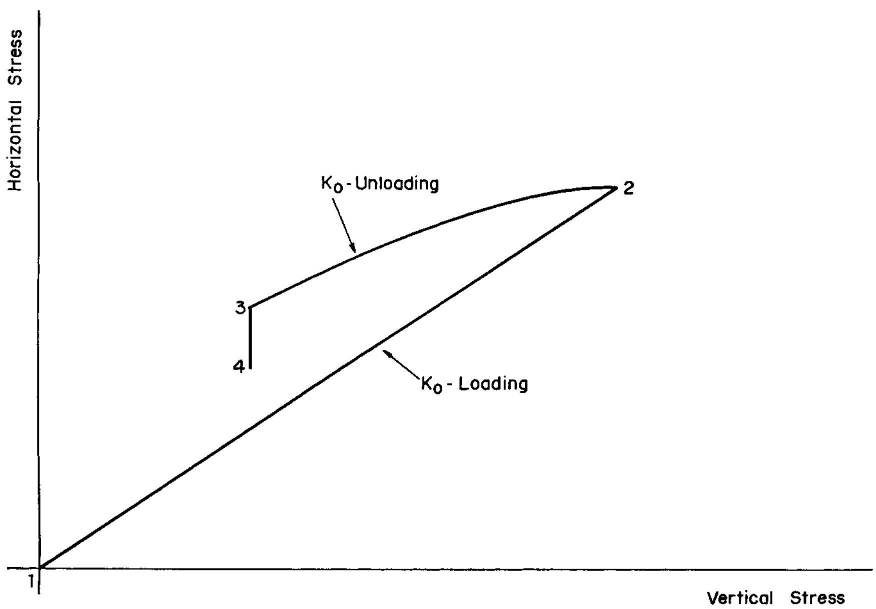

FIG. 20. Assumed Stress Path

increase in the stress state induced by compaction operations is shown by paths 1–2 in Fig. 20. This is followed by unloading under K_0 condition along paths 2–3 to the corresponding vertical stress at the end of construction. The residual stress-state condition is then achieved, letting the soil and reinforcement deform to reach equilibrium (path 3–4). The curve S2 in Fig. 19 represents the soil stress-strain curve for the residual stress-state condition (point 4). For larger soil horizontal deformations, curves S2 and S1 tend to the same Rankine active stress state condition. Ehrlich and Mitchell (1994) assume a more realistic stress path, but the one described above makes the phenomena addressed here easier to comprehend.

For extensile reinforcement, strain compatibility is usually assumed between soil and reinforcement, and the soil is assumed to be fully plastified in the failure surface. Limit equilibrium methods may be used under these conditions. Factors of safety are adopted for soil shear resistance and reinforcement strength in design to guarantee equilibrium.

The curves R1, R2, R3, and R4 represent the stress-strain behavior of four different reinforcements. The curves R1, R2, and R3 are related to reinforcements with the same peak strength, but with different postpeak behavior. The R4 curve represents another reinforcement with a higher tension resistance. All these R-curves may represent actual behavior of ''extensible'' reinforcements in the field, considering the factors of safety usually used in design and the increase of the reinforcement stiffness under confinement.

In Fig. 19 the equilibrium condition is represented by the intersection of S and R curves. Assuming no compaction, i.e., assuming the soil under geostatic condition (curve S1), for the hypothetical case under consideration all reinforcements would match equilibrium at a certain strain. The stiffer the reinforcement, the higher would be the tensile stress and the lower would be the strain at equilibrium.

Compaction increases the required horizontal soil stress at equilibrium. Considering the stresses induced during compaction (curve S2), only the structures built with reinforcements

R3 and R4 would be stable. In spite of the fact that peak strength of R1 and R2 reinforcement curves is greater than the strength corresponding to the active stress state, equilibrium is not possible, even at large strain. Therefore, it can be seen that compaction may have an important role in the stability and working stress condition of reinforced soil systems. Anyhow, large safety factors on soil shear resistance and reinforcement strength are usually adopted in conventional design, and equilibrium may be possible even considering the cases of R1 and R2 reinforcement curves case.

In summary, limit equilibrium methods consider only geostatic stress and may not adequately represent reinforced systems at working stress and at failure under certain conditions. This discussion exemplifies that for a more realistic analysis to better represent actual field conditions, compaction and soil reinforcement stress-strain compatibility should be taken into consideration.

APPENDIX. REFERENCES

Adib, M. E. (1988). ''Internal lateral earth pressure in earth walls,'' PhD dissertation, University of California, Berkeley, Calif.

Dantas, B. T. (1998). ''Working stress analysis method for reinforced soil slopes,'' MSc thesis, COPPE, Federal University of Rio de Janeiro, Rio de Janeiro (in Portuguese).

Duncan, J. M., and Seed, R. B. (1986). ''Compaction-induced earth pressures under K_0-conditions.'' *J. Geotech. Engrg.*, ASCE, 112(1), 1–22.

Ehrlich, M., and Mitchell, J. K. (1994). ''Working stress design method for reinforced soil walls.'' *J. Geotech. Engrg.*, ASCE, 120(4), 625–645.

Jewell, R. A. (1985). ''Limit equilibrium analysis of reinforced soil walls.'' *Proc., 11th Int. Conf. on Soil Mech. and Found.*, 1705–1708.

Seed, R. B., and Duncan, J. M. (1986). ''FE analyses: Compaction-induced stresses and deformations.'' *J. Geotech. Engrg.*, ASCE, 112(1), 23–43.

Closure by Jorge G. Zornberg,[6] Nicholas Sitar,[7] Members, ASCE, and James K. Mitchell,[8] Honorary Member, ASCE

The writers thank the discussers for their interest and their discussion. The limitations of limit equilibrium analysis regarding its inability to deal with displacements and its limited representation of the interaction between dissimilar or incompatible materials comprising the slope have been well acknowledged in the technical literature [e.g., Leshchinsky (1999)]. Typically, adequate selection of materials' properties and safety factors has ensured acceptable displacements without consideration of the possibly significant effect of compaction-induced stresses not only in reinforced, but also in unreinforced structures analyzed using limit equilibrium. However, the purpose of the investigation presented in the paper was not to assess aspects of limit equilibrium that this method admittedly cannot handle, but to validate the aspects that this method has been credited of handling.

The horizontal strain versus horizontal stress diagrams illustrated in Fig. 19 presented by the discussers are useful for schematic representation of the deformability and strain compatibility between soil and reinforcement in reinforced soil structures. However, the writers believe that these diagrams should not be used for schematic representations of mechanisms leading to instability of reinforced soil structures. This is because stability of reinforced slopes is not governed by the horizontal strain compatibility between soil and reinforcement, but instead is governed by the development of a (nonhorizontal) shear failure surface.

APPENDIX. REFERENCE

Leshchinsky, D. (1999). ''Stability of geosynthetic reinforced steep slopes.'' *Proc., Int. Symp. on Slope Stability Engrg., IS-Shikoku*, Balkema, Rotterdam, The Netherlands (in press).

[6]Asst. Prof., Dept. of Civ., Envir. and Arch. Engrg., Univ. of Colorado at Boulder, Boulder, CO 90309.

[7]Chancellor's Prof., Dept. of Civ. and Envir. Engrg., Univ. of California, Berkeley, CA 94720.

[8]Univ. Distinguished Prof. and Via Prof. of Civ. Engrg., Virginia Tech, Blacksburg, VA 24061.

INTERPRETATION OF PILE ACCEPTANCE CRITERIA FROM DEFICIENT DATA[a]

Discussion by Adrian F. K. Yong,[2] Member, ASCE

The author should be commended on the contribution to forecasting the ultimate bearing capacity of a pile not tested to failure. However, the discusser does not share the author's opinion that ''These criteria (the pile acceptance criteria) can be applied only if the settlement of the pile is large (10% of the pile diameter in the case of the Indian Standard and 50 mm in the case of the Australian Standard).''

Indeed, a pile load test is conducted for a variety of reasons, including the following as given by Fleming et al. (1986) and Poulos and Davis (1980):

- To serve as a proof test to ensure that failure does not occur before a selected proof load is reached, this proof load being the minimum required factor times the working load.
- To determine the ultimate bearing capacity as a check on the value calculated from dynamic or static approaches, or to obtain backfigured soil data that will enable other piles to be designed.
- To determine the load-settlement behavior of a pile, especially in the region of the anticipated working load. This information can be used to predict group settlements and settlements of other piles.
- To indicate the structural soundness of the pile.

Although not explicit, British Standard 8004 (''Foundations'' 1986) alluded to the primary purposes of (1) load settlement behavior and (2) ultimate bearing capacity. So, the objective of a load test is not always to determine the ultimate bearing capacity. Nonetheless, the topic attracted numerous researchers including Chin (1970) and Brinch Hansen (1963).

In normal practice, the design working load for a pile is determined by static design methods. Under the Australian Standard AS2159, the pile may be tested in a sustained load test with two load cycles. The first load cycle shall apply a maximum load equal to the design working load and the sec-

[a]October 1998, Vol. 124, No. 10, by Shenbaga R. Kaniraj (Technical Note 16340).

[2]Dir., IHN Geoconsultancy Sdn Bhd, 24-2 Jalan USJ 10/1, Subang Jaya, 47620 Petaling Jaya, Selangor, Malaysia.

ond shall apply a maximum load of 1.5 times the design working load.

The test pile would be accepted if the pilehead settlement is less than those quoted in the technical note for the above test loads per the pile acceptance criteria presented in the Australian Standard. The acceptance criteria should therefore be read as the upper bound limit of pile performance, and there is never a need to mobilize the settlements given in the acceptance criteria.

The discusser is not familiar with the Indian Standards. However, a pile tested to 1.5 times the working load with a deflection less than 12 mm or two times the working load with a deflection less than 10% of the pile diameter of 7.5% of the bulb diameter would surely have met the acceptance criteria.

The British Standard 8004 considers that constant rate of penetration is more appropriate for determination of ultimate bearing capacity and provided guidelines for ascertaining failure. The guidelines state that the load corresponding to a pilehead settlement equal to 10% of the diameter of the base of the pile is normally taken as the ultimate bearing capacity. However, the guidelines noted the practical difficulty of loading a large pile to a settlement equal to 10% of the base diameter.

In conclusion, it is not necessary to mobilize the settlement given in the acceptance criteria or to ascertain the failure load to accept the performance of a pile in a load test.

APPENDIX. REFERENCES

Brinch Hansen, J. (1963). ''Discussion of 'Hyperbolic stress-strain response cohesive soils.''' *J. Soil Mech. and Found. Div.*, ASCE, 89(4).

Chin, F. K. (1970). ''Estimation of the ultimate load of piles not carried to failure.'' *Proc., 2nd Southeast Asian Conf. on Soil Engrg.*

Fleming, W. G. K., Weltman, A. J., Randolph, M. F., and Elson, W. K. (1986). *Piling engineering*. Wiley, New York.

''Foundations.'' (1986). *British Standard 8004*, British Standards Institute, London.

Poulos, H. G., and Davis, E. H. (1980). *Pile foundation analysis and design*. Wiley, New York.

Closure by Shenbaga R. Kaniraj[3]

The writer thanks the discusser for his interest in the paper. The Indian Standard (''IS: 2911 (part 4)'' 1985) gives the specifications for two types of pile load tests, namely, initial and routine tests. According to this standard, the initial test is required for one or more of the following purposes: (1) to determine ultimate load capacities and arrive at safe load by application of factor of safety; (2) to provide guidelines for setting up the limits of acceptance for routine tests; (3) to study the effect of piling on adjacent existing structures and decide on the suitability of the types of piles to be used; (4) to get an idea of the suitability of the piling system; and (5) to have a check on calculated load by dynamic or static approaches. The routine test is required for one or more of the following purposes: (1) one of the criteria to determine the safe load of the pile; (2) to check the safe load and extent of safety for the specific functional requirement of the pile at working load; and (3) to determine any unusual performance contrary to the findings of the initial test, if carried out. The criteria given in the paper are for determining the safe load from initial load tests. In the case of routine tests, the standard specifies that the test shall be carried out for a test load of at least one and one-half times the working load; the maximum settlement of test loading in position is not to exceed 12 mm. The discusser is therefore correct in his conclusions that pile load tests are performed for different reasons. It is also not always necessary to mobilize the settlement given in the acceptance criteria or to ascertain the failure load to accept the performance of pile in a load test. That would be necessary only in initial load tests where the purpose is to determine the safe load or service load. If this were the purpose of the test and still the settlement specified in the criteria had not been mobilized, the paper describes procedures by which the deficient data can be interpreted. It may be also noted that even in initial tests the test load need not exceed one and one-third times the load corresponding to 12 mm settlement. If under this load the settlement is less than 10% of pile diameter then the first criterion would govern the safe load.

[3]Asst. Prof., Dept. of Civ. Engrg., Indian Inst. of Technol., New Delhi 110 016, India.