Discussion and Response

PEAK VERSUS RESIDUAL SHEAR STRENGTH IN GEOSYNTHETIC-REINFORCED SOIL DESIGN

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Discussion by J.P. Giroud

I would like to thank all panelists of the first session of the Fifteenth GRI Conference (13 December 2001) for the intellectual pleasure they gave me with brilliant presentations and excellent discussion. Professor Koerner should be commended for inviting you and for placing this session at the beginning of the conference, which put all of us on a high level of excitement for the next two days. I am sure one of Professor Koerner's Austrian ancestors was no less than Mozart, who knew how to hook listeners for an entire symphony by placing the most brilliant notes at the beginning. Take some time off to listen to the Jupiter symphony, if you want to know what I mean.

After the Jupiter, let's come back to Earth (indeed), because I think we might have missed a point during the panel discussion. If the point I discuss below was not missed, I apologize for my lack of attention or my lack of memory.

If a phenomenon Y potentially depends on two independent parameters, A and B, there are several possibilities: (i) it depends on both (Y depends on A and B); (ii) it depends on one of them (Y depends on either A or B); and (iii) it depends on none (Y depends on neither A nor B). If a series of tests is performed with parameter B constant, no conclusion can be drawn on the influence of B and only the influence of A can be evaluated.

Based on the above rationale, the tests performed by J. Zornberg demonstrate that the phenomenon (i.e., the ratio Y = γ/g, which can be used to characterize slope failure) depends on the peak shear strength of the soil. However, the series of tests does not provide any information on the influence of the residual shear strength of the soil. Therefore, it is possible that the slope failure depends on both peak shear strength and residual shear strength. When Jorge writes: "the soil shear strength governing the stability ... is the peak shear strength" (p. 13), he implies that the residual shear strength has no influence. Based on the rationale presented above, it would be more appropriate to state that "the stability depends on the peak shear strength, but the test conducted does not provide any information on the influence of the residual shear strength". (It should be noted that I implicitly assume that peak shear strength and residual shear strength are two independent parameters. Of course, these two parameters are not totally independent; however, it may be legitimate to consider that they are independent from the viewpoint of this demonstration. This could be the subject of another discussion.)

Considering the importance of the subject, and the remarkable demonstration by Jorge of the influence of the peak shear strength of the soil, I encourage Jorge to address the second half of the problem. This could be achieved by performing a series of tests with different soils having the same peak shear strength but different residual shear strengths. This requires a significant amount of work because it is necessary to find an adequate set of different soils.

The series of tests described above may require a lot of time. In the meantime, Jorge could try a theoretical approach. If calculations show that the measured values of γ/g are consistent with the difference in peak shear strength, then it is possible to conclude that stability depends only on peak shear strength, assuming that the calculations are based on a correct theory. In fact, Jorge started using the theoretical approach when he indicated that K = 0.084 for ϕ = 35° and 0.062 for 37.5°, but he did not go further. As an attempt to complete what Jorge started, I used the charts by Schmertmann et al. (1987). Unfortunately, these charts are not very precise. It seems from Figure 9a and Table 3 of Schmertmann et al. that the ratio between the values of K for 35° and 37.5° is approximately 1.2, whereas 0.084/0.062 = 1.35. However, the charts are not sufficiently precise to draw a firm conclusion. Furthermore, as stated above, the demonstration is valid only if the theory is correct, which is hard to know. Therefore, the tests described above seem to be necessary. However, Jorge has calculation means far more powerful than the charts published by Schmertmann et al. (1987) and could pursue the theoretical approach.

Jorge, thank you for this excellent paper on a very important subject. I hope you do not mind that I am sending this discussion to all panelists, which I do because I consider the above comments as an extension of the panel discussion.

Response by J.G. Zornberg

I fully agree with J.P. Giroud that Professor Koerner (with help of Mozart and other Austrian ancestors?) should be commended for keeping track of each note of the GRI-

15 symphony and achieving a remarkable balance between incitement and harmony. I should also thank J.P. Giroud for his insightful comments, and I take this opportunity to commend his inspiring pursuit of truth and attention to detail. I am sure that one of J.P.'s French ancestors includes René Descartes, who knew how to break the whole into parts to seek understanding of complex phenomena. Who knows, perhaps the whole is indeed greater than the sum of its parts.

Genealogy aside, let us go to the variables governing the failure of geosynthetic-reinforced soil structures. In a very Cartesian approach, J.P.'s discussion rigorously outlines how to evaluate the dependence of a phenomenon Y (failure) on two independent variables, A (peak shear strength) and B (residual shear strength). In a more holistic approach, I view the same phenomenon Y (failure) as being governed by variables A (soil shear strength), B (orientation of reinforcement forces), C (distribution of tensile forces with depth), D (shape of failure surface), among others. Within the framework of limit equilibrium, the selection of each one of these variables tends to be dichotomized (e.g., peak or residual shear strength? horizontal or tangential-to-failure-surface orientation of reinforcement forces? uniform or triangular distribution of tensile forces with depth? bilinear or circular failure surface?). I provide some additional information below to justify such dichotomization for the specific case of selection of shear strength parameters (the focus of my Fifteenth GRI publication).

I agree with J.P. that performing a series of tests with different soils having the same peak shear strength but different residual shear strength would be insightful. I am afraid, though, that this may not be experimentally feasible. This is because, while there is an elegant approach to "manufacture" soils with the same residual but different peak shear strength (i.e., by using the same soil with different densities), the use of different soil types could lead to major inconsistencies in shear strength characterization (nonlinearity of shear strength envelopes, characterization of "residual" versus "large-displacement" conditions, etc.). However, I am glad that J.P. outlined an alternative, theoretical approach. That is, to perform calculations to evaluate if the results are consistent with differences in peak shear strength for models constructed with different backfill densities. Centrifuge results verified that the reinforcement effect could be normalized, which facilitates evaluation of reinforced structures with simple configuration. Note that normalization is an implicit assumption made in published design charts such as the design charts published by Schmertmann et al. (1987), Jewell (1991), and Leshchinsky and Boedeker (1989). Analyses using a rigorous limit equilibrium approach have indeed been performed using Spencer's method and circular failure surfaces (see Zornberg et al. 1998b). Although I will not get into the rigorous limit equilibrium analyses in this response (see Zornberg et al. (1998b) instead), simplified chart analyses are provided below. These simplified charts actually follow an approach similar to that outlined in J.P.'s discussion using charts developed by Schmertmann et al. (1987). Specifically, analyses were performed using design charts developed by Leshchinsky and Boedeker (1989) and Jewell (1991), which use log spiral and bilinear failure surfaces, respectively. Note that plane strain conditions were considered in the analyses. Overall, very good agreement is obtained between experimental results and chart (limit equilibrium-based) predictions when using peak shear strength in the analyses. Also, equally good agreement was obtained between experimental and predicted locations of the failure surfaces when using peak shear strength in the analysis (this cannot be evaluated with charts, but was analyzed using rigorous limit equilibrium analyses).

I should finally add that visual observation of the development of failure surfaces showed no progressive failure. This is important because identification of progressive failure mechanisms would have revealed that failure might depend on both peak and residual shear strength properties. In fact, failure initiation, failure progression, and final collapse developed within a single g-level increment.

CHART ANALYSES

The design charts by Leshchinsky and Boedeker (1989) and Jewell (1991) have similar characteristics: the overall factor of safety is accounted for using a factored soil friction angle which, together with the slope inclination, yields the required normalized summation of reinforcement forces. Validation of the centrifuge model results can be done using the design charts in the reverse order, i.e., for a given normalized summation of reinforcement forces, the mobilized friction angle can be estimated. As the analysis is performed for centrifuge models at failure (i.e., $FS = 1$), the mobilized friction angle obtained from the design charts equals the actual friction angle. Leshchinsky and Boedeker (1989) and Jewell (1991) consider a triangular distribution of the reinforcement forces, with maximum tension at the base of the structure. This distribution does not agree with the experimental centrifuge results. Nevertheless, the design charts can also be used for the case of uniform distribution of reinforcement forces with depth.

Figure 10 shows the analysis using the chart developed by Leshchinsky and Boedeker (1989). The coefficient m in the chart defines the slope inclination ($m = 2$ for a 1H:2V slope). For the case of uniform distribution of reinforcement forces with depth, the dimensionless mobilized equivalent tensile resistance K in the chart corresponds to the normalized coefficients (K_D and $K_B = K_S$) obtained from Figure 9 of the paper. For these normalized coefficients, the mobilized friction angles ϕ_m obtained from the charts are $\phi_m = 39°$ and $\phi_m = 42°$ for centrifuge models built with sand at 55 and 75% relative densities, respectively. These predicted mobilized friction angles are in good agreement with the peak plane strain friction angles obtained for Monterey sand at the relative densities used in the centrifuge models ($\phi_{ps} = 39.5°$ and $\phi_{ps} = 42.5°$). Consequently, the predicted factors of safety are approximately 1.00, which indicates good agreement with the experimental results.

Figure 11 shows the analysis using design the chart developed by Jewell (1991). The coefficient K in the chart corresponds to the normalized coefficients (K_D and $K_B = K_S$) obtained from Figure 9 of the paper, ϕ_d is the design friction angle of the backfill soil, and β is the slope inclination. Using the normalized coefficients obtained from the centrifuge tests, the design friction angles ϕ_d obtained using the charts for a slope angle $\beta = 63.4°$ (1H:2V) are approximately equal to 39° and 42.5° for centrifuge models built with sand at relative densities of 55% and 75%, respectively. The predicted design friction angles are in very good agreement with the peak plane strain friction

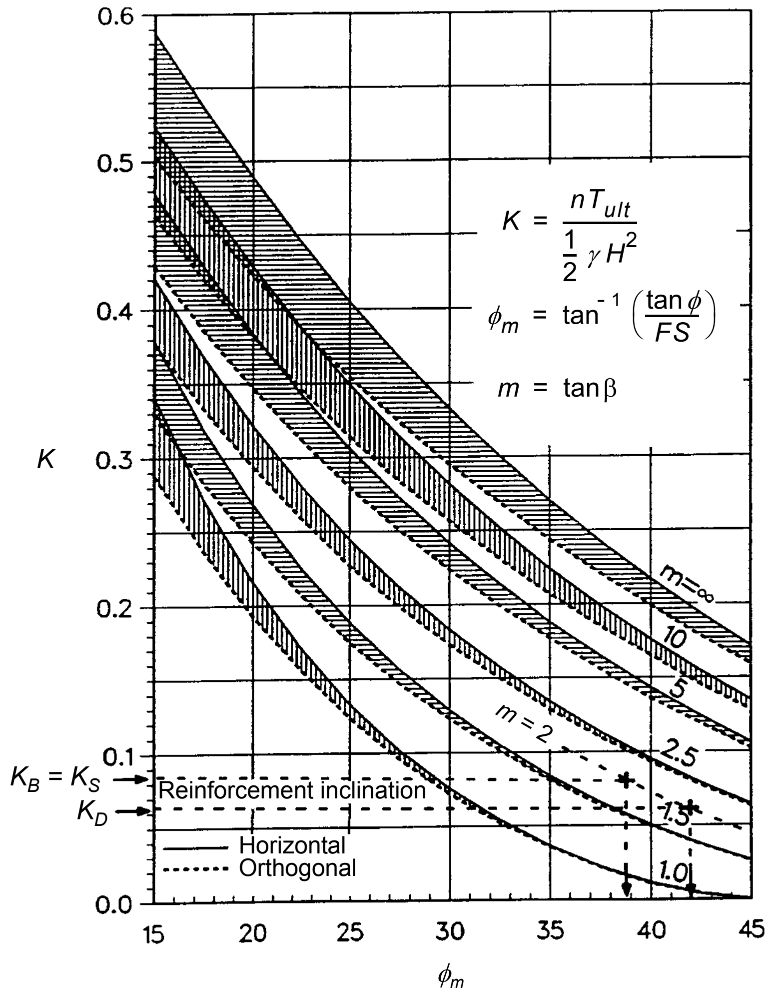

Figure 10. Design chart for reinforced soil slopes (adapted from Leshchinsky and Boedeker (1989)).

angles obtained for Monterey sand at the relative densities used in the models. Consequently, Jewell's limit equilibrium design methodology also shows good agreement with the experimental results.

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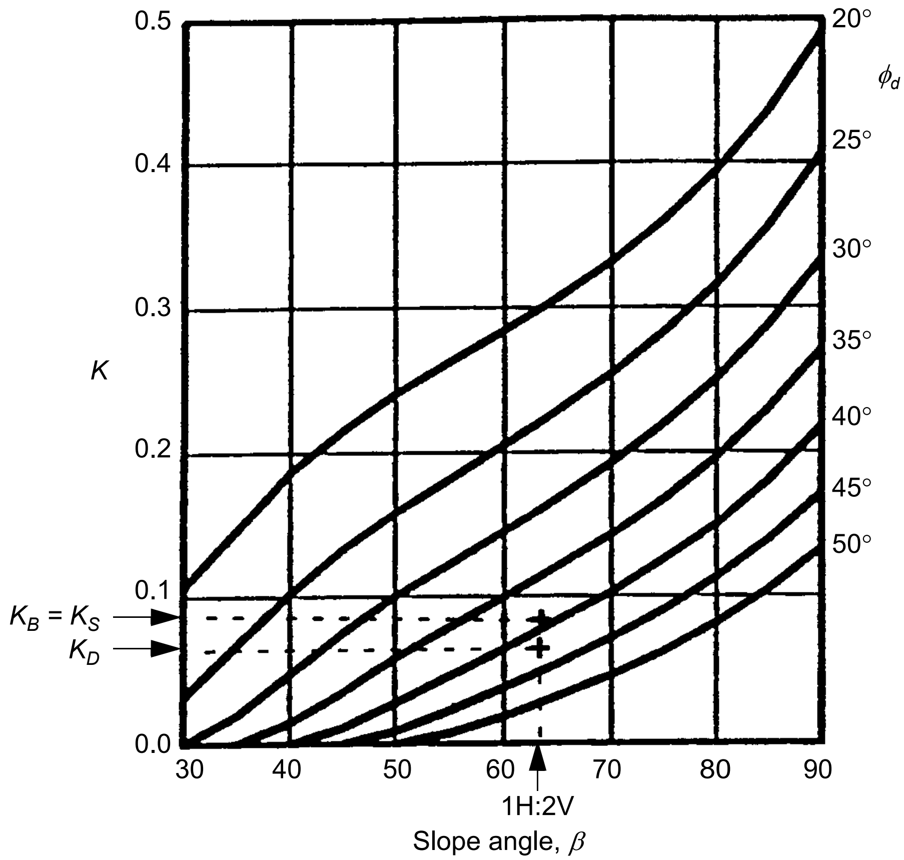

Figure 11. Design chart for reinforced soil slopes (adapted from Jewell (1991)).

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