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Discussion and authors closure on Performance of test embankment constructed to failure on soft clay

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APPENDIX. REFERENCES

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Closure by B. Indraratna[6] and A. S. Balasubramaniam[7]

RESPONSE TO DISCUSSION BY SIEW-ANN TAN

It has been explained in the paper (page 18) that the Cam-clay parameters have been selected so that the undrained experimental stress-strain curve coincides with the undrained predictions from the modified Cam-clay theory. The method of parameter estimation proposed by Britto and Gunn (1987) is most appropriate if the conditions of associated flow rule and the normally consolidated behavior are satisfied. While these conditions were approached at certain depths, Muar clay samples generally exhibited light overconsolidation and nonassociated flow. The deviation from the modified Cam-clay theory was significant in the upper clay layer, where desiccation and weathering close to the ground surface influenced the preconsolidation pressure and shear strength (page 13). In fact, within the first 2 m below ground level (i.e., crust), the overconsolidation ratio (OCR) exceeds 2.0. Tan is quite correct in pointing out that the simple relationship between M and ϕ does not hold for the parameters given by the authors, particularly for the upper clay layer. The authors have not used this algebraic expression to estimate M from ϕ or vice versa. Instead, the parameters given in Tables 2 and 3 are determined from independent laboratory tests as mentioned on page 16.

In addition to the simple relationship between λ and C_c, Schofield and Wroth (1968) have also proposed simple relationships between λ, the plasticity index and the liquid limit:

$$PI = 1.71\ \lambda = 0.615\ (LL - 0.09) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

For the Muar clay, the variations of the plasticity index and liquid limits are significant depending upon the depth. At certain depths, the plasticity index remains approximately constant, but the liquid limit varies substantially. This relationship implies a variation of λ from say 0.1 to 0.3, if the evaluation of λ had been purely based on the Atterberg limits. For similar reasons, the simple relationship of $\lambda = C_c/2.303$ is not applicable for clays that deviate from the conventional Cam-clay assumptions. Therefore, the authors have used independent consolidation tests and CK_oU triaxial tests to determine the most suitable parameters for lightly overconsolidated Maur clay. Considering the Cam-clay parameters suggested by Schofield and Wroth (1968) for an array of normally consolidated to lightly overconsolidated clays, the values used by the authors for Muar clay are certainly not unreasonable.

Tan has suggested a set of Cam-clay parameters for a Singapore marine

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clay based on CIU tests. The authors acknowledge that these are typical of some normally consolidated clays found in Southeast Asia at certain depths (Indraratna et al. 1992). Nevertheless, particularly for the upper clay layers, the value of λ decreases as a result of light overconsolidation or crusting. A very recent analysis is elucidated in the following section to explain the use of pseudo-λ for a Bangkok clay sample which deviates from the normally consolidated line (NCL). The pseudo-λ values for Muar clay are quite similar. The actual experimental shape of the yield locus is shown in Fig. 24. The experimental undrained stress-strain response corresponding to the actual yield locus is matched with the stress-strain behavior associated with the modified Cam-clay yield locus by moving point A_e to B (zone ii) in Fig. 25. Consequently, the modified λ_{oc} of 0.146 is smaller than the conventional value of 0.357 corresponding to the normally consolidated line (NCL). This technique reveals accurate results within the working stress range from P_A to P_B (i.e., between points A and B in Fig 25). For a stress state exceeding P_B, the value of λ should be representative of the NCL. As a result of the thick weathered crust (high overconsolidation ratio), the underlying clay experiences a stress range below P_B. Therefore, the use of pseudo-λ is more appropriate for modeling the soil behavior. Further details of estimating λ for Muar clay and the effect of its sensitivity on lateral yield have been discussed by Balachandran (1990). A subsequent study by Ratnayake (1991) has shown that depending upon the depth of the Muar clay samples, the appropriate value of λ can vary from 0.1 to 0.5 for a relatively constant M value of 1.0–1.1.

If CRISP is employed for clays that deviate from the conventional normally consolidated behavior, the authors also recommended that the parameters λ, κ, and M should be related to correct stress path testing. For example, in the vicinity of the embankment toe where swelling has to be modeled accurately, load-extension testing is preferable to load-compression triaxial testing. It has been shown by Yau (1990), that although Cam-clay theory regards λ, κ, and M to be unique for a specific yield surface, they are influenced by the nature of stress path testing (Fig. 26).

RESPONSE TO DISCUSSION BY E. W. BRAND

Brand has correctly recognized the difference between the findings of the current study and previous predictions given elsewhere by Balasubramaniam et al. (1989). The authors wish to emphasize that the previous predictions made by Balasubramaniam et al. (1989) have no direct relevance to the current analysis. This independent study adopted a detailed finite element analysis using both undrained and fully coupled (Biot) consolidation models. In addition to the modified Cam-clay constitutive model, the hyperbolic stress-strain behavior was also incorporated to extend the scope of predictions. As mentioned by Brand, it may be true that for soft clay embankments, the prediction of slip surface by conventional undrained stability analysis is sufficient. However, a detailed effective stress analysis based on the finite element approach is often preferable to a simple undrained stability analysis for the following reasons.

The complete behavior of a soil structure cannot be interpreted, unless the development of any excess pore water pressures and deformation mechanisms are properly investigated. Using a finite element approach, the authors have shown that the failure surface or shear band propagation can be interpreted on the basis of the displacement vectors and shear stress contours. Unlike in a conventional factor of safety analysis, the incremental

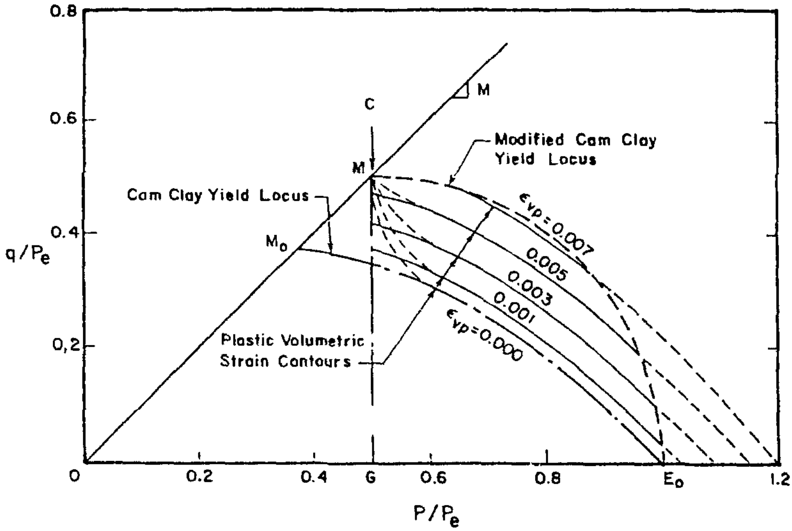

FIG. 24. Simplified Plastic Volumetric Strain Contours within State Boundary Surface from CID Tests (Wet Zone)

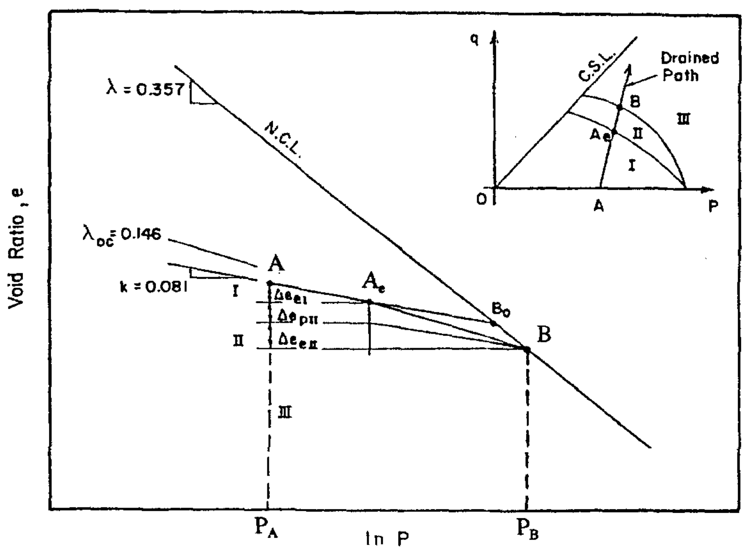

FIG. 25. Schematic Diagram for Plastic Volume Change during Conventional Drained Compression and Definition of λ_{oc}

displacements provide a more comprehensive picture of how the failure initiates and propagates. There is no doubt that finite element applications in geotechnical engineering are not yet perfect, as further refinements in constitutive modeling and stress path characteristics are still needed. While basic empirical rules and simple conventional methods may still be attractive and sufficient for a limited array of geotechnical problems, the advancement

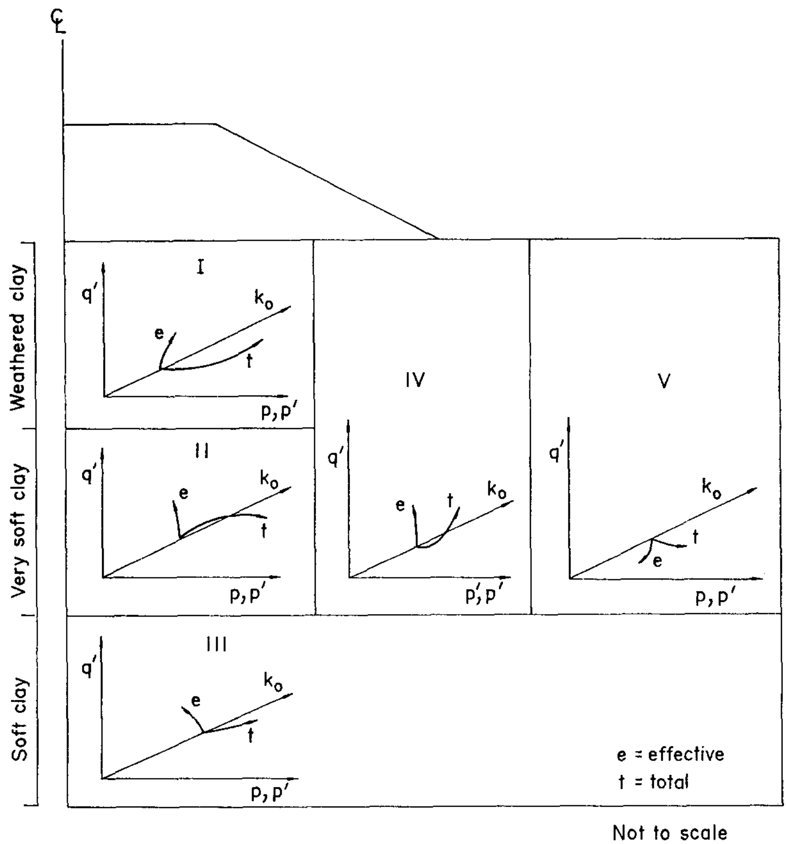

FIG. 26. Categorization of Stress Paths in Soft Clay Subjected to Embankment Loading (Yau 1990)

in computer technology and numerical know-how will continue to expand the scope of better predictions in stability analysis for both simple and complex situations.

APPENDIX. REFERENCES

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