Therefore, a higher value of $R_e$ will be expected except $R_f$ is the unity.

In practice, the cut-off line is included in the hyperbolic equation, as shown by Deschamps and Leonards. Fig. 17 shows the result of the stability analysis including the cut-off line effect, corresponding to three cases of $R_f = 0.7, 0.8, 0.9$ in Fig. 16. The failure shear strains for those cases are estimated from the strains at the intersections of the cut-off line and the hyperbolic curve as shown in Fig. 16. As shown in Fig. 17, it can be seen that the failure patterns inferred from the failure shear strain contours for all cases approximately agree with the critical slip surface from the limit equilibrium method. Also, the values of $R_e$ for all cases (such as that $R_e = 0.94$ for the case of $R_f = 0.7, R_e = 0.95$ for the case of $R_f = 0.8$ and $R_e = 0.96$ for the case of $R_f = 0.9$) approximately agree with $F_e (= 0.92)$ from the limit equilibrium method.

Finally, the writers would like to take this opportunity to clarify some features of the Shear Strength Reduction Technique that Yeo and Chan seem not to have understood very well. Design philosophy of the technique consists of such three steps as selection of failure shear strain, determination of failure pattern and measure of safety factor. The failure shear strain can be obtained from the standard triaxial test. The writers will demonstrate that consistent stability analysis results for both loose and dense sands with their failure shear strains obtained from triaxial tests (for example, Matsui and San, 1992b, 1993). Design methodology was not discussed in the present paper, but will be found in our other papers (for example, Matsui and San, 1993). In our finite element analysis, both geometry non-linearity and material non-linearity have been taken into account, and for the large-deformation formulation, updated Lagrangian description (for example, Davidson and Chan, 1978) has been used. Also, the writers summarize several common definitions of failure used in the finite element analysis, such as non-convergence definition of failure, continuous yield-elements zone definition of failure, limit average stress level definition of failure and critical displacement definition of failure. The aim of the present paper is to propose an alternative definition of failure, being based on strain-based failure judgment.

References

PREDICTION OF THE PARTIALLY DRAINED BEHAVIOR OF SOFT CLAYS UNDER EMBANKMENT LOADING)

Discussion by Jin-chun Chai\(^\text{(b)}\) and D. T. Bergado\(^\text{(a)}\)

The Authors are to be congratulated on making a most useful contribution to the knowledge of the partially drained settlement of the soft clays under embankment loading by systematic finite element analyses. Special-
ly, the method of inversely evaluating the mass permeability of the clay foundation based on the limited observed consolidation settlement and normalized loading intensity. This has practical meaning because there is no satisfactory method for determining the mass permeability of the soft clay foundation (Tavenas and Leroueil, 1980). In interpreting the results of the settlement ratio versus permeability plots (Fig. 4), except the partial drainage effect, two other factors need to be considered, namely: (1) the difference between the excess pore pressure dissipation ratio and the settlement ratio; and (2) the influence of the loading intensity on foundation deformation pattern. Only in the case where the settlement is linearly proportional to the increment of the effective stress that the settlement ratio is the same as consolidation ratio defined by the excess pore pressure dissipation ratio. However, in the Sekiguchi-Ohta’s model used by the Authors or the Cam clay model, the compression coefficient, \( k \), is constant in void ratio, \( e \), versus logarithm of the mean effective stress, \( \Delta \sigma \). In this case, the settlement is not linearly related to the increase in effective stress. For example, if the mean effective stress increased from \( p_0 \) to \( p_1 \) with an incremental value of \( \Delta \sigma \) yield a reduction in void ratio of \( \Delta e \). And, the mean effective stress increased from \( p_1 \) to \( p_2 \) also with the same incremental value of \( \Delta \sigma \) yield a reduction of void ratio of \( \Delta e \). And \( \Delta e \) will be larger than \( \Delta e \). Then, the settlement ratio may not represent the consolidation ratio defined by the excess pore pressure dissipation ratio.

In the Authors analyses, the embankment loading was applied by surface loading with the top and the bottom widths of the embankments assumed fixed, while the height was increased (increased the load intensity). Thus, increasing the normalized load intensity implies an increase of the embankment slope or loading gradient at the embankment toe. In other words, the geometry of the embankments is changed. For embankment A, with the top width of 26.4 m and bottom width of 55.2 m, when loading intensity increased from 18.3 kPa to 170.0 kPa, the corresponding embankment height increases from 0.92 m to 8.5 m, if the unit weight of the fill material is 20 kN/m\(^2\). Then, the embankment slope varies from 1:15.6 (V:H) to 1:1.7 (V:H). Increasing the slope of the embankment will increase the shear stress level in the foundation soil near the embankment toe and increase the lateral displacement which is mainly caused by undrained disturbance in the foundation soil. Subsequently, the settlement during the loading period can be increased.

The lateral displacement is also sensitive to the foundation permeability. The lower the foundation permeability, the larger the lateral displacement (Hird and Kwok, 1986), and subsequently, the larger the total settlement which was also indicated in Fig. 8 of the paper. In this case, the lateral displacement induced settlements increase with the decrease of the foundation permeability.

Whether the assumed foundation soil is normally consolidated or overconsolidated is not clear to the discussers. It seems a normally consolidated ground is assumed, but the initial vertical effective stress is larger than the stress calculated by the buoyant unit weight multiplied by the corresponding depth. However, if the ground is overconsolidated, another factor that influences the settlement ratio is the change of the compression coefficient during the loading process. When the loading intensity is low during the loading process, the larger portion of the loading path might stay within the yield locus of the foundation soil and yield smaller portion of the total settlement at the end of loading. Then, the settlement ratio at the end of loading condition is lower.

The assumed embankment A has been analyzed by a computer program called CRISP-AIT which was developed based on original CRISP computer program (Ditto and Gunn, 1987) using the same finite element mesh and the same boundary conditions. The soft clay is represented by modified Cam clay model and the soil parameters are the same as reported in the paper. The initial void ratio which is needed in Cam clay model is calculated from the unit weight of the soil and assumed specific gravity (2.65) and 100% saturation condition. It is assumed that the
ground is normally consolidated, and the same initial effective stress given in Fig. 2 of the paper is used. Two runs were conducted with load intensity of 18.3 kPa and 109.0 kPa and with the foundation permeability of $10^{-6}$ cm/sec. The loading rate of 1.2 kPa/day was closely followed. The results show that although the settlement ratios are 0.27 and 0.36 for loading intensity of 18.3 kPa and 109.0 kPa, respectively for both cases, the excess pore pressure dissipation ratio is nearly the same. At the end of loading condition, the existing excess pore pressure is about 85% of the applied vertical surficial loading. Regarding the lateral displacement, if the load intensity only influences the partial drainage of the foundation, the ratio, $R_L$, of maximum lateral displacement over the corresponding maximum settlement at end of the loading would be smaller for the higher load intensity because the strength gain in the foundation will reduce the lateral displacement for the later part of loading period. However, the results show that for both cases, the ratio of $R_L$ is the same and about 0.44. This is due to the mutual effect of strength gain in the foundation soil during the loading period and the increase of the loading gradient near the toe of the embankment for higher loading intensity case. It is understood that different soil model and different computer program may yield different results, but it is also believed that the tendency should be alright.

For the case study reported in the paper, at early stages of the loading, the settlement was under predicted. One possible reason cited by the Authors is the difference between the actual and simplified loading procedure. The discussers analyzed the behavior of an embankment on Muar clay, Malaysia by finite element method. Similar phenomenon has been observed. However, for the Malaysian embankment, the actual embankment loading rate and the loading procedure were simulated during the finite element analysis. Muar clay deposit has about 2.0 m topmost weathered clay layer overlying about 16.0 m soft clay. The weathered clay layer has an average over-consolidation ratio (OCR) of about 4 and soft clay layer has an average OCR of 1.1. Tavenas and Leroueil (1980) investigated the performance of several embankments on soft ground and concluded that since all the natural clays are more or less overconsolidated caused by weathering, water level fluctuation, etc., under the embankment loading, the soft ground behaves close to drained condition initially and close to undrained condition after the soil yield. When considering the drastic change of the foundation permeability before and after soil yield which was controlled by the modified Cam clay model, the finite element prediction improved a lot (Bergado et al., 1992). However, the consolidation state of the ground for the case study is not clear to the discussers also.

The embankment loading was applied by surficial loading. In this case, the embankment stiffness and the shear stress between the embankment and the foundation (outward or inward) are not simulated by the finite element method. These factors will influence the stress/strain state of the foundation soil near the embankment base and subsequently influence the deformation pattern. It is understood that for such kind of complicated finite element analyses, in order to obtain a clear trend, a good choice of important factors must be considered. However, when applying the result to the actual case, the stiffness of the embankment, and the possible embankment/foundation soil interaction might need to be taken into account.

It must be mentioned that the Authors made their assumptions and the limitations for application of the results very clearly in the paper. The discussers only raised the aforementioned points which might be useful for further research work in this area.

References

PREDICTION OF THE PARTIALLY DRAINED BEHAVIOR OF SOFT CLAYS UNDER EMBANKMENT LOADING

Closure by Akira Asagai*, Masaki Nakano* and Minoru Matsuo**

We authors appreciate Drs. Jin-Chun Chai and D. T. Bergado for their kind interest in authors' work. In their discussion they have pointed out a few critical questions and also mentioned important suggestions for the practical applications of the settlement ratio in evaluating the mass permeability. This closure addresses will clarify the discussers' comments.

The theoretical background of the method of evaluating the mass permeability using observation of consolidation settlement proposed in the paper can be summarized step-by-step as follows:

1. Even when a loading path on a clay foundation such as embankment procedure is the same, different soil permeability gives different effective stress path to a soil element within the clay.

2. Since the clay is an elasto-plastic material, the different stress path yields different strain path and consequently different permeability gives different "deformation path" to the clay foundation. This is particularly true when two or three dimensional loading problems are considered.

(3) The settlement ratio $p_e/p_i$, in which $p_e$ is the settlement at the end of embanking and $p_i$ is the settlement at the end of consolidation, is probably the simplest parameter of expressing "deformation path" and is also the most easiest parameter to be observed in-situ. Therefore, if the settlement ratio is shown to be the real representative of the "deformation path", then the observation of the settlement ratio $p_e/p_i$ can give significant information on the mass permeability.

Discussers have also checked using a different computer program (CRISP) and a different constitutive model (modified Cam clay). Based on a series of finite element simulation on the elasto-plastic consolidation deformation behaviour of a normally consolidated clay foundation. They suggest that the method of using the settlement ratio observation is applicable for determining the approximate value of mass permeability.

These discussers have also suggested several points that should be carefully taken into account when the method is applied to a real soft soil problem, among them the soil profile construction for the case where the upper part of the clay is in overconsolidated state may be the most critical. The writers are very grateful to have this suggestion. When the method is applied in a particular embankment practice, the chart given in Fig. 4 of the original paper should be redrawn so that the chart may meet both particular boundary conditions and particular soil profile.