Studies of Some Effects Resulting from the Unloading of Soils

Etudes concernant quelques effets de la suppression des charges sur les sols

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SUMMARY

Two allied investigations have been made into effects resulting from the unloading of soils. Part A considers the effects of stress release, in the sampling operation, on the effective stresses of undisturbed samples. The results of laboratory tests simulating the sampling operation are presented. Part B deals with some aspects of loading on *in-situ* effective stresses. This is considered to be important in understanding the effects of repeated loading on soils in road bases.

SOMMAIRE

Deux études connexes ont été consacrées aux effets de la suppression des charges sur les sols. Dans la partie A de cet exposé, on considère les effets de la suppression des contraintes au cours du prélèvement d'échantillons sur les contraintes effectives des échantillons intacts. On présente les résultats d'essais en laboratoire qui simulent le prélèvement d'échantillons. La partie B traite de certains aspects de l'effet de la suppression des charges sur les contraintes effectives *in situ*. On le considère essentiel à la compréhension de l'effet de chargements réitérés sur les sols dans les couches de fond des routes. L'étude de ces deux phénomènes intervient dans la solution de problèmes pratiques dans les travaux de génie concernant les sols.

A. EFFECTS OF STRESS RELEASE IN THE UNDISTURBED SAMPLING OPERATION

MOST PREDICTIONS in soil mechanics, whether of settlement, slope stability, or bearing capacity, make use of soil properties measured on reputedly "undisturbed samples." Release of stress in the sampling operation is unavoidable. Stresses of a purely compressive nature can be retained in most fine-grained saturated soils by capillary pore water pressures. Shear stresses existing in the field will inevitably be released on sampling. The release of shear stress generally alters the effective stress in the sampled soil even if no change in water content or over-all void ratio has taken place (Bishop and Henkel, 1953).

The Relation between In-Situ Vertical Effective Stress and the Effective Stress after Sampling

In general, the horizontal stress in a body of soil is not equal to the vertical stress. When shear stresses are applied by slope loading or other agencies, the difference between horizontal and vertical stresses becomes critical.

Fig. 1A represents the stresses acting on an element of soil at a depth, H, below ground level and at equilibrium with a water table, h feet above the element. The vertical effective stress is:

$$\sigma_V' = \sigma_V - u_0 = \gamma H - \gamma_w h \tag{1}$$

where γ and γ_w are respectively the bulk density of the soil and the density of water. If K is the ratio of the horizontal effective stress σ_H' to the vertical effective stress σ_V' then,

$$\sigma_H' = K\sigma_V' = K(\gamma H - \gamma_w h). \tag{2}$$

If it is assumed that both σ_V' and σ_H' are principal stresses, the maximum shear stress on the element under consideration will be

$$\sigma_V' - \sigma_H' = (1 - K)(\gamma H - \gamma_w h). \tag{3}$$

If the element is sampled without change in either water content or volume, the shear stress will be released and, on removal from the sampling device, the soil will be acted on by an isotropic effective stress:

$$\sigma' = -u_0 - \Delta u \tag{4}$$

where u_0 is the original (*in-situ*) pore pressure and Δu is the change in pore pressure due to the sampling operation. Δu has two components: (*a*) one due to the release of the horizontal compressive stress, σ_H, and (*b*) one due to the release of the shear stress ($\sigma_V - \sigma_H$). For a saturated soil (Skempton, 1954), Δu can be written:

$$\Delta u = \Delta\sigma_H + A_s(\Delta\sigma_V - \Delta\sigma_H) \tag{5}$$

that is,

$$\Delta u = -\sigma_H + A_s(-\sigma_V + \sigma_H)$$

where A_s is a pore-pressure parameter for the release of stress during sampling. Alternatively,

$$\Delta u = [A_s(K - 1) - K][\gamma H - \gamma_w h] - \gamma_w h$$

and since $u_0 = \gamma_w h$,

$$\sigma' = [K - A_s(K - 1)][\gamma H - \gamma_w h]$$

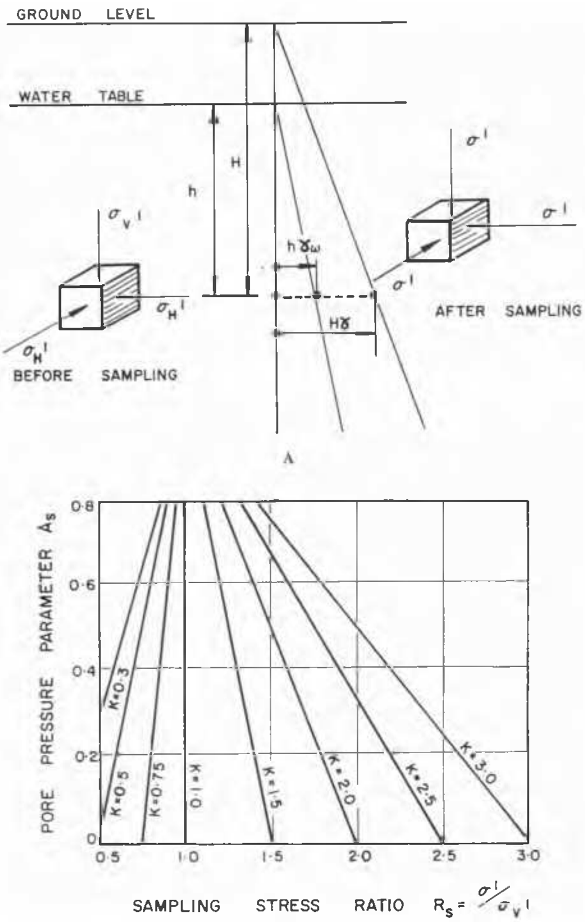
or

$$R_s = \sigma'/\sigma_V' = K - A_s(K - 1). \tag{6}$$

R_s is the sampling stress ratio. It should be noted that if $A_s = 0$

$$R_s = K \text{ or } \sigma' = \sigma_H'. \tag{7}$$

Fig. 1B shows the relation between the sampling stress ratio, R_s, the *in-situ* stress ratio, K, and the parameter A_s. From this diagram it appears that for values of K between



B

FIG. 1. A, stresses on an element of soil before and after sampling; B, relation between pore-pressure parameter, A_s, *in-situ* stress ratio, K, and sampling stress ratio, R_s.

0.5 and 1.5, K is relatively insensitive to variations in A_s and the approximation $K = R_s$ can be expected to give fair accuracy.

The Effect of Consolidation History on the Sampling Parameters R_s and A_s

Little information has been published on the effects of consolidation history and soil type on the parameters R_s and A_s. Only K_0 conditions have been investigated (Skempton, 1961; Skempton and Sowa, 1963), but it is of practical importance to consider stress ratios varying from the active condition (K_A) to the passive condition (K_p), including K_0 conditions as a special case.

Fig. 2 shows the results of two series of tests on a saturated clay soil in which the sampling stress ratio, R_s, was measured at several overconsolidation ratios for K_0 consolidation. The overconsolidation ratio has been taken as the ratio $\sigma_V'(\text{maximum})/\sigma_V'(\text{before sampling})$.

In the first series, the soil was consolidated in the oedometer to various overconsolidation ratios. The sampling operation was simulated by removing the soil from the oedometer ring and the suction in each sample was then measured using a pressure plate (Bishop, *et al.*, 1960) to obtain the values of R_s.

In the second test series, the soil was consolidated in the triaxial cell to various overconsolidation ratios; K_0 conditions were maintained throughout. The method used was that of correlating axial strain with volumetric strain (Bishop, 1958). K_0 for normally consolidated soil was found to be 0.48. Jaky's approximate expression of $K_0 = (1 - \sin \phi')$ gives a value for K_0 of 0.52. The sampling operation was simulated by releasing the deviator stress under undrained conditions.

If the values of K_0 for triaxial consolidation are compared with corresponding values of R_s, it can be seen that R_s is a fairly good approximation of K_0, provided K_0 does not exceed about 1.2. Values of A_s increase with K_0 in the region $K_0 < 1$ and also increase with K_0 for $K_0 > 1$. In the region $K_0 = 1$ there seems to be a discontinuity in the variation of A_s. At $K_0 = 1$, A_s is indeterminate and the presence of the discontinuity is difficult to test directly.

Fig. 3 shows the results of sampling tests following anisotropic consolidation of undisturbed samples of saturated marsh clay. This type of test gives a direct relationship between R_s and K and eliminates the need to assess A_s. *In-situ* values of K can be inferred directly from the observed values of R_s. The pattern followed by A_s in these tests is similar to that shown in Fig. 2 and this appears to confirm the presence of a discontinuity in the variation of A_s when $K = 1$.

Fig. 4 shows the variation of A_s with overconsolidation ratio for remoulded soil consolidated before sampling to two constant values of K. For a value of $K = 0.5$, the range of variation of A_s with overconsolidation ratio is large, but not as large as that accompanying K_0 consolidation or consolidation to various stress ratios. With a value of K before sampling of 2.0, the variation of A_s is somewhat less.

From the tests described in Figs. 2, 3, and 4, the following conclusions are drawn: (1) The sampling stress ratio R_s gives a good approximation to the *in-situ* stress ratio K for values of K up to about 1.2 (2) Values of the pore pressure parameter A_s are influenced mainly by the stress ratio before sampling and to a lesser extent by the consolidation history of the soil. (3) The range of variation of A_s is relatively large and it is probably best to determine *in-situ* stress ratios by means of special tests such as that depicted in Fig. 3.

B. EFFECTS OF REPETITIVE LOADING ON *in-situ* EFFECTIVE STRESSES

An element of soil within the structure of a road is subjected to a large number of very rapid applications of load interspersed with long intervals under overburden pressure where σ_V and σ_H do not vary. The ratio between the time under load and the time under overburden pressure is small, even on very heavily trafficked roads. Since the capacity of a three-lane carriageway may be about 25,000 vehicles per day (Wardrop and Duff, 1956), the number of applications of load on any one element of road base may be 8,000. The total time under load may be in the region of 600 seconds, i.e., for 10 minutes in every 24 hours the soil is stressed by the application of load. It is reasonable to assume that during the application and release of shear stress, no drainage from the stressed zone can occur, and hence the pore pressures will not be dissipated while the load is acting. Tests by Bishop and Henkel (1953) have shown that immediately after applying and releasing a shear stress, there is usually a residual pore pressure which may be greater or less than the equilibrium pore pressure in the soil.

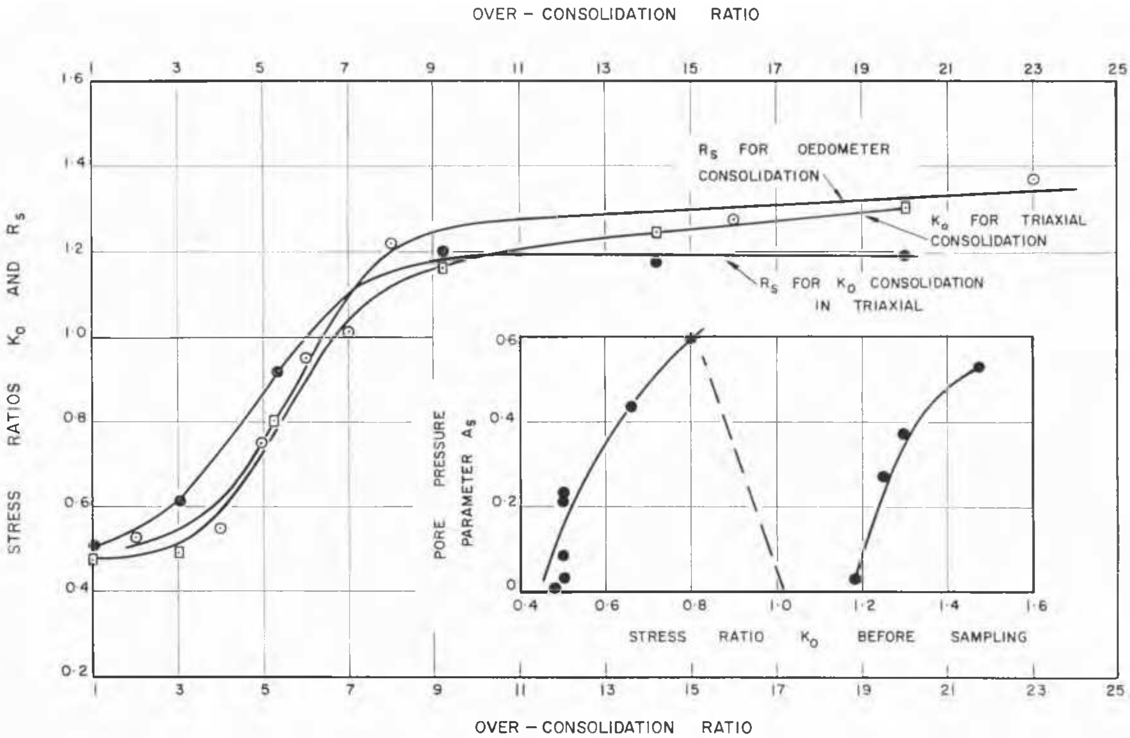


FIG. 2. The effect of stress history on the sampling stress ratio, R_s, and the pore-pressure parameter, A_s, for K_0 consolidation.

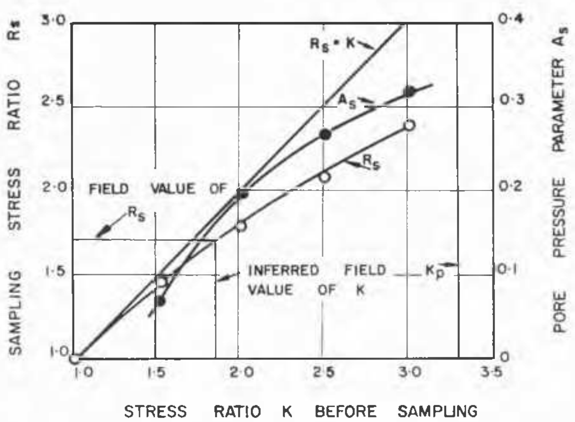


FIG. 3. Relation between *in-situ* stress ratio, K, sampling stress ratio, R_s, and pore-pressure parameter, A_s, for an anisotropically consolidated soil.

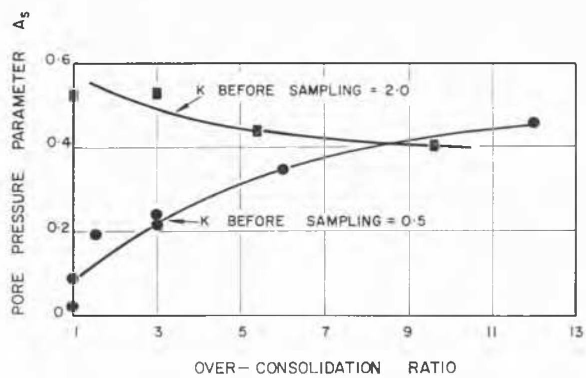


FIG. 4. The effect of stress history and *in-situ* stress ratio, K, on the pore-pressure parameter, A_s, for anisotropic consolidation.

Although it is possible, during periods of high traffic intensity, that complete dissipation of residual pore pressures may not occur in the road base, it appears that with most soils used for the construction of the compacted layers in a road, there is ample total time during off-peak periods for this residual pore pressure to dissipate. However, it is probable that in clayey materials below the road formation some of the residual pore pressure is not fully dissipated.

Behaviour of Normally Consolidated Soil under Repeated Loading

Loading of normally consolidated undrained triaxial specimens produces a positive pore-pressure increment (Bishop and Henkel, 1953). Unloading a specimen, which has not reached failure, initially causes a small increase of pore pressure followed by a decrease. At the end of unloading there is a residual positive pore pressure.

The dissipation of this positive pore pressure in the field after the passage of a vehicle causes a settlement in accordance with the effective stress principle; the soil becomes denser and the strength increases. Fig. 5 shows that successive cycles of loading, unloading, and dissipation reduce the magnitude of the residual pore pressure as the density and rigidity of the soil increase. Eventually, the soil is so rigid that further loading cycles produce virtually no residual pore pressure. At this stage, the soil acts completely elastically and the curve of residual deflection against number of loading cycles becomes asymptotic to a maximum value (Fig. 5B). Fig. 6 indicates that the maximum change of residual pore-pressure occurs during the first loading cycle and that repetition of load on the completely undrained soil

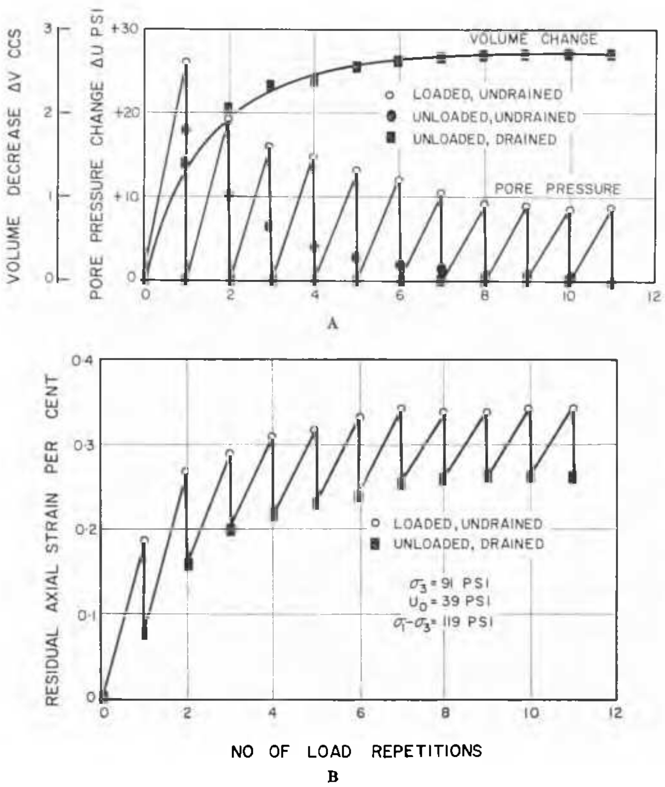


FIG. 5. Repetitive load applications on a normally consolidated clay silt. A, relation between volume change, pore-pressure change, and number of load repetitions; B, relation between residual axial strain and number of load repetitions.

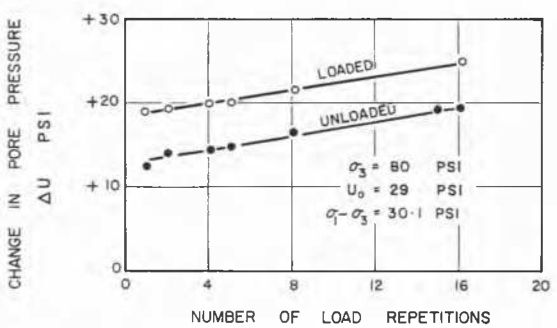


FIG. 6. Variation in change of pore pressure for completely undrained repetitive loading of a normally consolidated clay silt.

causes only a small increase in the residual pore pressure. Under field conditions, sufficient dissipation may occur between successive loads to reduce the effect of this small increase. It is possible, in very poorly drained situations, for the pore pressure to build up until shear failure occurs; this has been shown in laboratory tests by the authors.

Work by Wagener (1960) on repetitively loaded consolidometer specimens indicates that the consolidation settlement expected from a statically loaded specimen is equal to the settlement produced in the repetitively loaded specimen, provided the cumulative time during which the repetitive load acts is equal to the time during which the static load acts. For normally consolidated soils, therefore, the distress to the road surface may be estimated by considering the settlement produced by an equivalent static load.

Behaviour of Heavily Overconsolidated Soil under Repeated Loading

In a heavily overconsolidated soil subjected to increasing shear stress, the pore pressure increases slightly at first and then decreases until the pore-pressure increment becomes negative. If the shear stress applied is greater than a certain minimum, a reduction of shear stress results initially in an additional decrease of the pore pressure followed by an increase. Full release of the load leaves a pore-pressure deficiency. Drainage after unloading enables the soil to absorb water (Bishop and Henkel, 1953) and repeated cycles of this type lead to a substantial increase in the moisture content of the soil and a reduction in soil strength (Fig. 7).

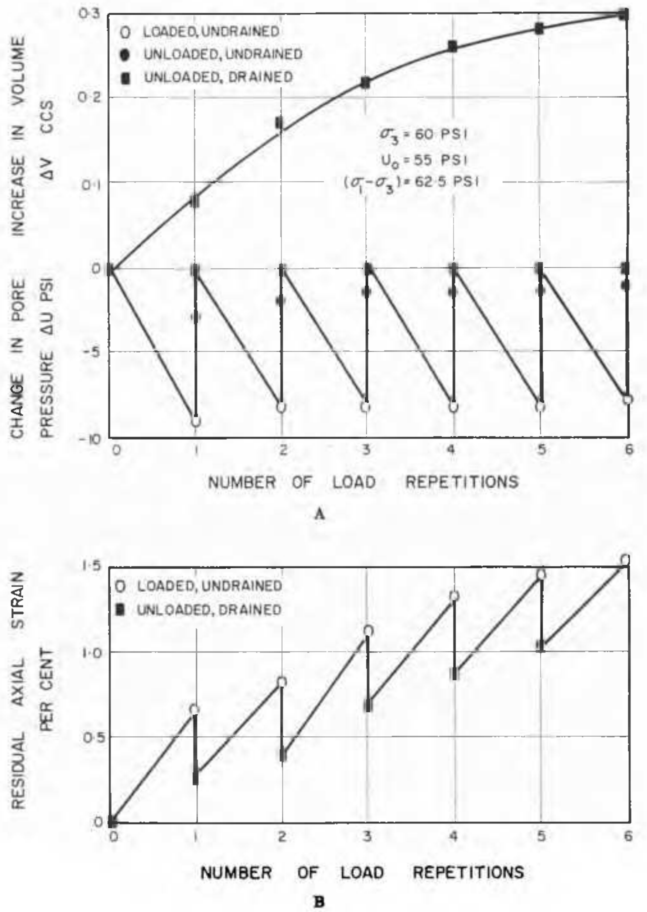
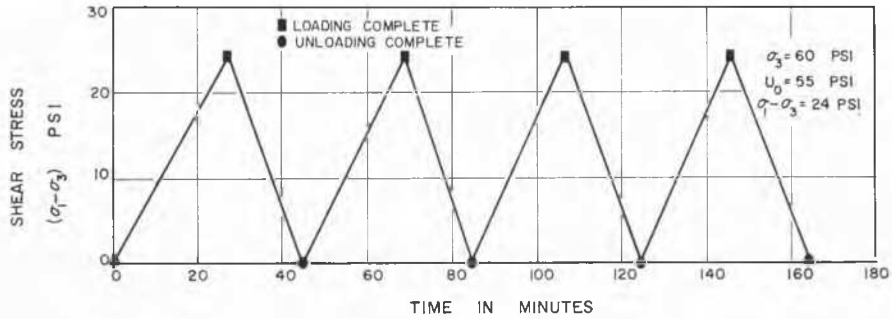


FIG. 7. Repetitive load applications on a heavily overconsolidated clay sand. A, relation between volume change, pore-pressure change, and number of load repetitions; B, relation between residual axial strain and number of load repetitions.

As with normally consolidated soils, the first loading cycle on the completely undrained soil produces the biggest change in residual pore-pressure deficiency. Subsequent cycles tend to increase the deficiency as shown in Fig. 8. The residual pore-pressure deficiency rapidly becomes asymptotic to a value more negative than the deficiency when the soil is loaded. In the field, it may be inferred that it is the drainage time and the water available rather than the density of traffic that control the quantity of water absorbed and the reduction in strength. In heavily overconsolidated soils the following patterns of pore pressure change were observed: (1) At the end of the unloading cycle, the pore-pressure



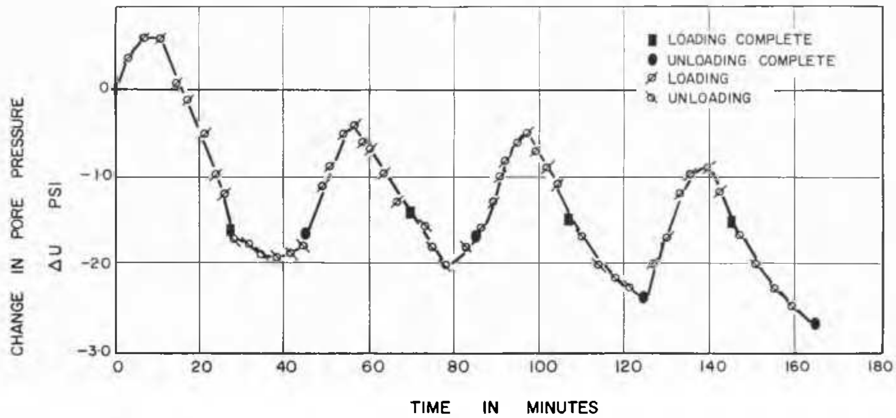


FIG. 8. Variation in change of pore pressure for completely undrained repetitive loading of a heavily overconsolidated clay sand.

deficiency can be greater than that at the end of loading; this occurs when the applied shear stress is sufficient to produce a substantial pore-pressure deficiency. (2) In a completely undrained soil, the pattern of pore-pressure change alters as the number of load repetitions increases. After a few cycles, unloading results only in a decrease of pore pressure, that is, an increase in pore-pressure deficiency.

CONCLUSIONS

There is a critical range of stress in which the effects of repeated loading are significant. The pore pressures measured in the triaxial test are not necessarily equal to the pore pressures generated by equivalent traffic loading, but give an indication of the types of pore-pressure change that can occur in the field.

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