Pore Pressure Changes during Shear in Two Undisturbed Clays

Variations de la pression de l'eau interstitielle mesurées pendant le cisaillement dans deux argiles non remaniées

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Summary

The results are reported of two investigations in which the measurement of pore water pressure changes during shear has aided the solution of practical problems.

In the first case measurements were made of the residual pore pressure remaining in a saturated soil after the transient application of a shear stress. In the case of a heavily overconsolidated clay the residual pore pressure was negative and in the presence of free water led to softening of the clay. In a normally consolidated clay the residual pore pressure was positive and led to further consolidation. The results are related to the problem of estimating the safe loads to be applied to formations and subgrades of clay.

In the second case the pore pressure changes in sampling and testing of normally consolidated stratum are considered and related to ways of estimating the in situ strength of natural deposits where, in general, the horizontal and vertical stresses are not equal.

Sommaire

Cette communication donne les résultats de deux expériences dans lesquelles le changement de la pression de l'eau interstitielle mesuré pendant le cisaillement a contribué à résoudre des problèmes pratiques.

Dans le premier cas la pression résiduelle dans l'eau interstitielle à été mesurée dans un sol saturé après l'application passagère d'un effort de cisaillement. Dans le cas d'une argile hautement hyperconsolidée la pression résiduelle dans l'eau interstitielle est négative, et, en présence de l'eau, tend à un amollissement de l'argile. Dans une argile normalement consolidée la pression résiduelle dans l'eau interstitielle est positive et tend à une consolidation supplémentaire. Les résultats sont apparentés au problème de la détermination de la charge critique de fondations et de terrassement dans l'argile.

Dans le second cas les changements de pression de l'eau interstitielle durant l'échantillonnage et l'analyse d'une couche anisotrope normalement consolidée sont étudiés et comparés aux procédés de détermination de la résistance in situ des dépôts naturels.

PART I. THE EFFECT OF A TRANSIENT LOAD ON THE EQUILIBRIUM MOISTURE CONTENT OF A SATURATED CLAY

Introduction

It is normally considered that loading, even if transient, will tend to compact a soil. However, in the course of research carried out into the shear strength characteristics of saturated sands it has been observed that the application of a shear stress can set up a negative pressure in the water in the pore space between the particles, which persists even after the removal of the stress (Fig. 1). The negative pore pressure is due to the tendency of the soil structure to expand when deformed in shear (known as dilatancy), and the residual negative pressure is associated with the fact that the removal of the stress is not accompanied by a complete reversal of strain.

It was obvious that if this phenomenon occurred in overconsolidated clays it would provide an explanation for the excessive softening which was found to occur in initially stiff clays subjected to transient loading beneath railway sleepers, and which could not be related to the swelling characteristics measured in an oedometer. Tests were therefore carried out on undisturbed samples of a heavily over-consolidated Weald clay, in which this softening had been encountered, to measure the changes in pore-pressure during transient loading, and the subsequent moisture content changes. (Transient loading is used in the sense of an application of stress during which no dissipation of pore pressure occurs.)

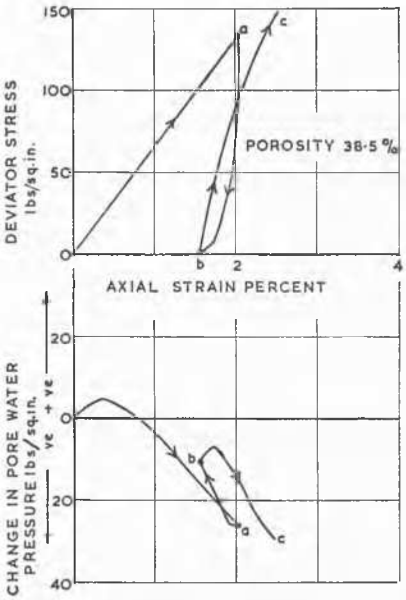

Fig. 1 Relation between Dewiator Stress, Pore Water Pressure and Strain for a Uniform Medium Sand
Rapport entre l'effort, la pression dans l'eau interstitielle et la déformation pour un sable moyen uniforme

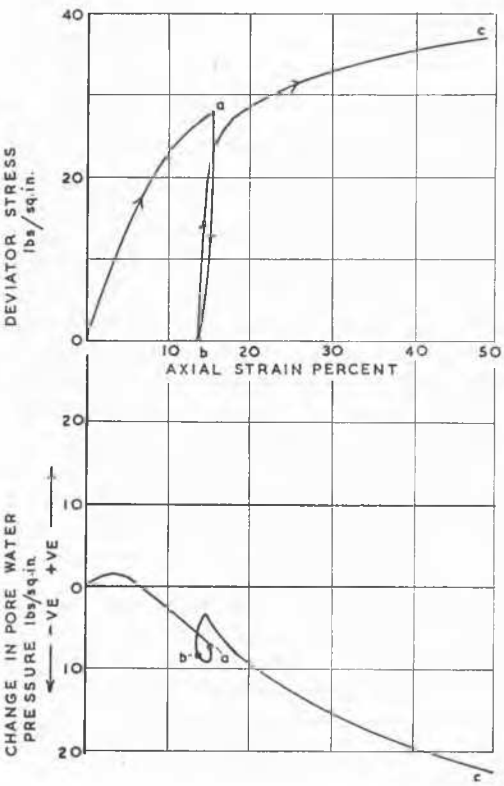

Fig. 2 Relation between Deviator Stress, Pore Water Pressure and Strain for Undisturbed Weald Clay
Rapport entre l'effort, la pression dans l'eau interstitielle et la déformation pour l'argile intacte de Weald

Test results

The Liquid Limit of the undisturbed samples described first was 40 per cent and the Plastic Limit 17 per cent. For the batch of remoulded material used for some subsequent tests the corresponding limits were 43 and 18 per cent.

In the first test an undisturbed sample, having a natural moisture content of 16.1 per cent was placed in a large oedometer under a pressure corresponding to its depth below the formation. No volume change took place and the sample was then tested in shear under undrained conditions, the pore pressure being measured. A deviator stress corresponding to about two-thirds of the ultimate strength was applied and removed, before finally testing the sample to failure. The resulting stress strain curve and pore pressure changes are illustrated in Fig. 2. It will be seen that under small deviator stresses the pore pressure assumed a small positive value, and then decreased again and became negative. On removal of the deviator stress the pore pressure underwent only a small change from the point (*a*) in Fig. 2 to the point (*b*), which

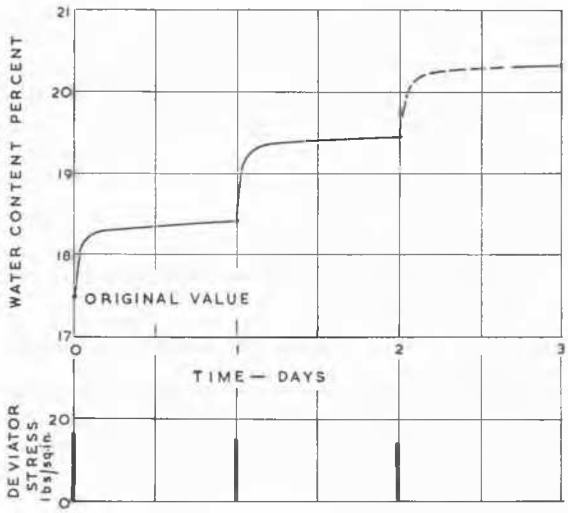

Fig. 3 Water Absorption by Undisturbed Weald Clay after Transient Loading
Absorption d'eau par l'argile intacte de Weald après un chargement transitoire

represents the residual negative pore pressure after a single application of the stress. On reloading the pore pressure again increased at first and then continued to decrease until failure occurred.

It follows that the initially high shear strength of such a sample is due to the large effective stresses resulting from the decrease in pore-pressure during shear. The presence of the residual negative pore pressure after the load has been removed means, however, that the clay will subsequently absorb free water in contact with it and become softer.

In the second type of test the absorption of water and consequent softening were directly measured. A sample was set up in the triaxial apparatus and allowed to come to equilibrium in contact with water under a small cell pressure corresponding to the present overburden pressure. A transient deviator stress was then applied, and observations made to detect any changes in water content that might take place in the clay after the stress had been removed (based on the measurements of the volume of water entering the sample). The stress was about two-thirds of the ultimate strength of the clay at its initial water content, and was applied and removed in rather less than 1 minute.

It will be seen from Fig. 3 that after the stress had been removed the sample gradually took up an appreciable amount of water. The sample was then subjected to further transient loadings of decreasing magnitude, and absorbed additional water in each case, the water content having increased from 17.5 per cent to 20.3 per cent after three cycles.

The sample was tested to failure under undrained conditions, and was found to have a strength of only one-third of the value corresponding to its initial water content as shown in Fig. 4. These tests clearly indicate that the transient shear stress set up by the passage of a wheel load over an over-consolidated formation or subgrade, though insufficient to cause failure at its intial water content, may lead to softening

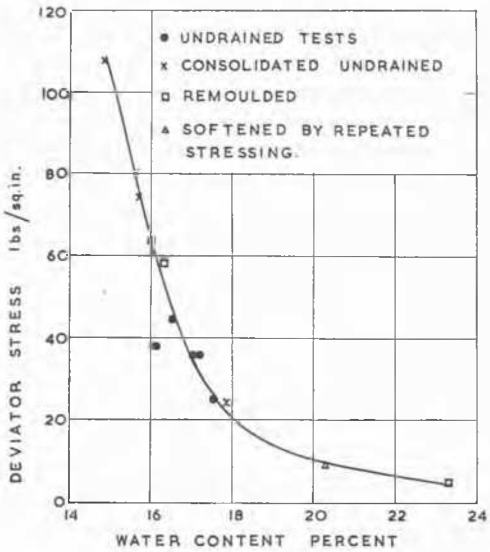

Fig. 4 Relation between Water Content and Compression Strength for Weald Clay
Rapport entre la teneur en eau et la résistance à la compression pour l'argile de Weald

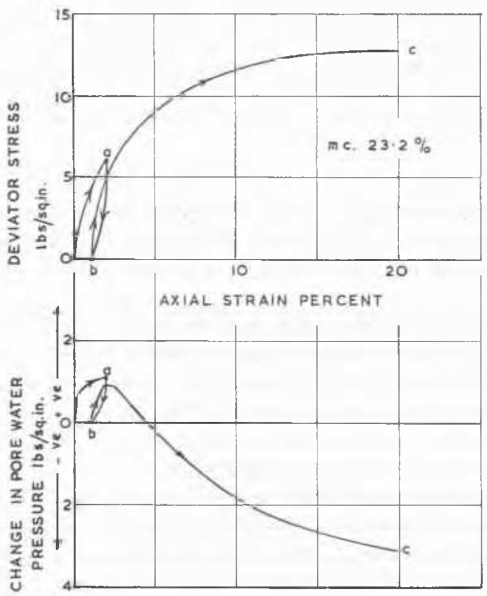

Fig. 5 Relation between Deviator Stress, Pore Water Pressure and Strain for an Overconsolidated Sample of Weald Clay
Rapport entre l'effort, la pression dans l'eau interstitielle et la déformation pour un échantillon hyperconsolidé d'argile de Weald

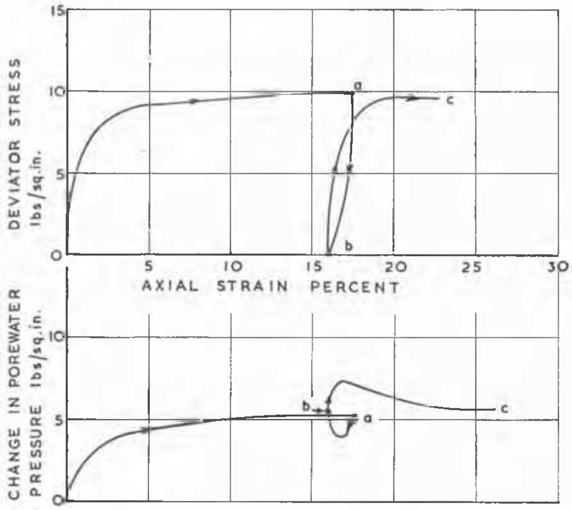

Fig. 6 Relation between Deviator Stress, Pore Water Pressure and Strain for a Normally Consolidated Sample of Weald Clay
Rapport entre l'effort, la pression dans l'eau interstitielle et la déformation pour un échantillon normalement consolidé d'argile de Weald

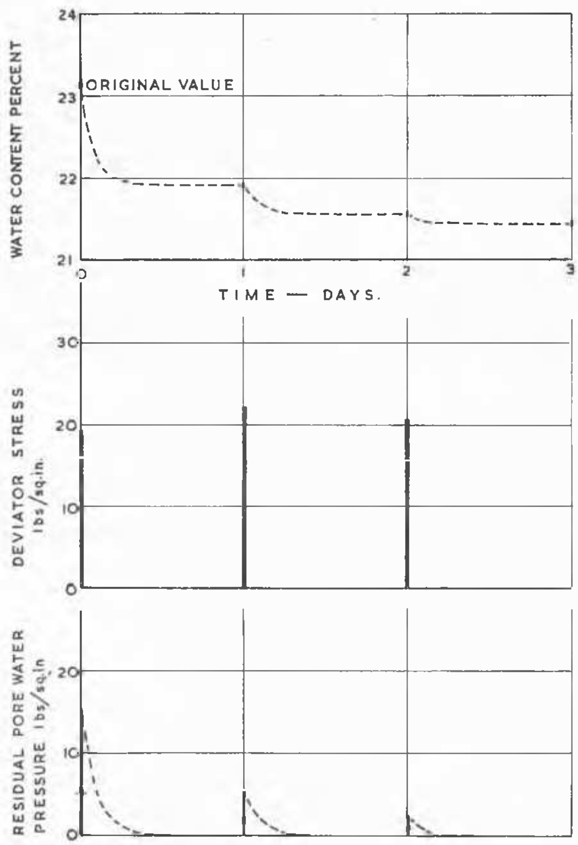

Fig. 7 Consolidation of a Normally Consolidated Sample of Weald Clay after Transient Loading
Consolidation d'un échantillon normalement consolidé d'argile de Weald après un chargement transitoire

and failure under subsequent applications of the same load. (Small residual shear stresses of an opposite sign will probably remain on the removal of the load due to small irreversible plastic deformations in the more highly stressed zones, and allowance must be made for this in applying the test results quantitatively.)

Under a deviator stress which represents a small proportion of the ultimate strength it is seen from Fig. 2 that the change in pore-pressure is positive, and a residual negative pressure is less likely to occur on unloading. In Fig. 5 the results of an undrained test on an overconsolidated sample are given, in which the sample is unloaded when the pore-pressure increase has almost its maximum positive value. (This test and two subsequent tests on normally consolidated samples were carried out on a batch of remoulded material from the same strata.) A drop in pore-pressure occurs on unloading, but does not lead to a negative residual pressure. No softening would therefore occur after the transient application of a stress of this magnitude.

The effect of a transient load on a normally consolidated sample of the same clay is shown in Fig. 6. Removal of the deviator stress leaves a residual positive pressure, of approximately one half of the deviator stress. The transient application of a shear stress will therefore tend to cause additional consolidation. (A similar result is obtained on a normally consolidated clay discussed in Part II, Fig. 9.) This consolidation is illustrated in Fig. 7, in which water content changes due to transient stressing are recorded for a sample initially fully consolidated under the cell pressure. A deviator stress almost equal to the ultimate strength at the initial water content was applied and removed, and the residual positive pore-pressure recorded. The pore-pressure was then allowed to dissipate, and the volume change recorded. It will be seen that after each cycle of stressing a residual positive pore-pressure was set up, leading to further consolidation when the sample was allowed to drain, though the effect decreased in magnitude with the number of repetitions.

After three cycles the water content had dropped from 23.1 per cent to 21.5 per cent, and the undrained strength when tested at this water content was 50 per cent greater than at the initial value. It therefore follows that in normally consolidated soils the transient application of a shear stress smaller than the ultimate strength causes additional consolidation and an increase in strength, and thus an increase in load bearing capacity.

The results quoted above mean that in estimating the safe load bearing capacity of a formation or subgrade on the basis of undrained tests alone (*Glossop* and *Golder*, 1944), a very different factor of safety may be required for different soils depending on the degree of overconsolidation. The results also have implications in relation to moisture content changes adjacent to bored and driven piles, in compaction by rolling, and in other civil engineering problems where transient stressing occurs.

Part II. The Influence of Anisotropic Consolidation on the Interpretation of Laboratory Data

Introduction

The use of the undrained test on isotropically consolidated[1]) samples for predicting the in situ strength of a soil (*Taylor*, 1948) and for estimating its gain in strength after consolidation under an increased over-burden pressure has been found to require re-examination in view of discrepancies between strengths predicted on this basis, and those occurring in the field. Attention has been drawn to this problem by *Hansen* and *Gibson*, 1949, on the basis of theoretical work in terms of the "λ" theory put forward by *Skempton*, 1948.

The principle involved became of practical importance in relation to the construction of an earth dam on a foundation containing a thin horizontal stratum of clayey silt (Liquid Limit 28, Plastic Limit 18), in which it was necessary to calculate the pore-pressures set up during construction and the gain in strength due to partial dissipation.

Test results

As the test results refer to samples consolidated in the field and in the laboratory under a range of overburden pressures, it is convenient to express the test results in terms of the parameter (c/p) where c is the apparent cohesion in the undrained tests ($= \frac{1}{2}$ compression strength) and p is the effective overburden pressure under which the sample has been consolidated in the ground or the vertical stress under which it has been consolidated in the triaxial cell.

Undrained tests on carefully prepared undisturbed samples gave a value of c/p of 0.36. An isotropically consolidated undrained test, however, gave a value for c/p of 0.57 (Fig. 8).

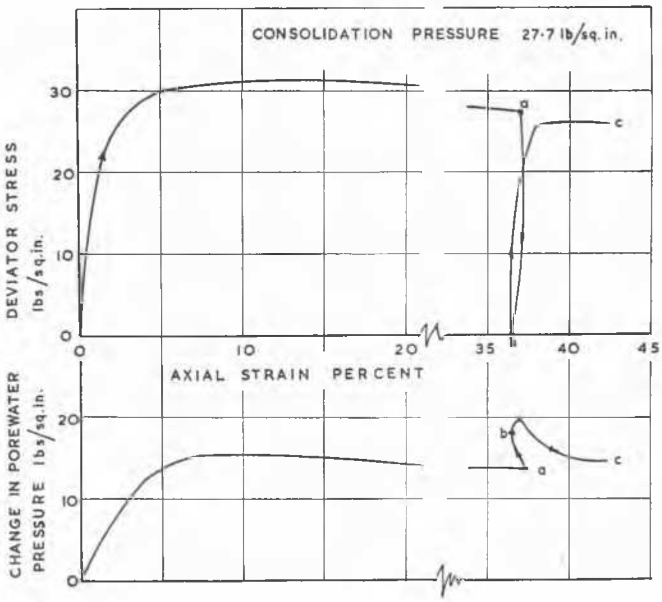

Fig. 8 Relation between Deviator Stress, Pore Water Pressure and Strain for a Clayey Silt
Rapport entre l'effort, la pression dans l'eau interstitielle et la déformation pour un limon argileux

[1]) We shall call this the isotropically consolidated undrained test.

This large discrepancy might be explained in two ways: (*a*) sampling disturbance causing the undrained test on the undisturbed samples to give values lower than the in situ strength or (*b*) the use of isotropic consolidation, which is not representative of the state of stress in the ground. In order to examine these alternatives the cycle of stress changes occurring in ideal sampling from natural strata were reproduced in the laboratory.

An undisturbed sample taken from a depth at which the effective overburden pressure was 5.8 lbs. per sq. inch was consolidated in the triaxial apparatus under conditions of no lateral yield up to a vertical stress of 25.4 lbs. per sq. inch. The ratio of the increases in lateral and vertical effective stresses required to maintain the condition of no lateral strain is a measure of the coefficient of earth pressure at rest K_0 and in this case the average value of the coefficient K_0 was found to be 0.43. The test procedure is described more fully by *Bishop* (1950) and *Bishop* and *Eldin* (1953), the only difference in the case of soils of low permeability being the need for side drains made of thin filter paper strips and the use of a slow rate of application of stress.

In a saturated sample the only change in effective stress which occurs during ideal sampling is that due to the removal of the deviator stress under which it was existing in the ground. The removal of the isotropic stress has no effect unless negative pore-pressures are set up large enough to cause cavitation in the pore space or to exceed the capillary forces. The effect on the pore water pressure of removing the deviator stress is shown in Fig. 9 and will be seen to be a small increase in pore-pressure of 1.7 lbs. per sq. inch (from *a* to *b*). (Strains are expressed in terms of the length of sample at zero deviator stress.) This means that a sample, which in the ground was consolidated under an effective overburden pressure of 25.4 lbs. per sq. inch and a lateral stress of about 10.6 lbs. per sq. inch, would on removal from the sampler only retain an isotropic

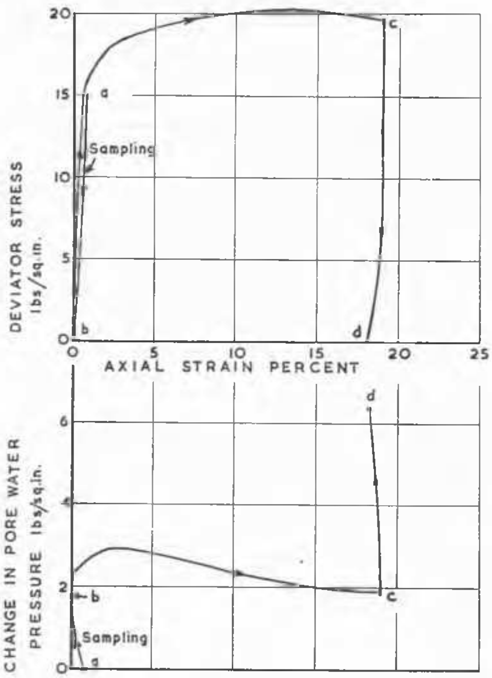

Fig. 9 Relation between Deviator Stress, Pore Water Pressure and Strain for a K_0-Consolidated Sample of Clayey Silt
Rapport entre l'effort, la pression dans l'eau interstitielle et la déformation pour un échantillon K_0 consolidé de limon argileux

effective stress of 8.9 lbs. per sq. inch, even if no overstressing or partial remoulding had occurred.

However, when the sample is tested in undrained compression with the deviator stress in the same vertical direction, the rate of increase in pore-pressure with stress (Fig. 9) is much less than in a sample normally consolidated under isotropic conditions (Fig. 8). The strength would thus tend to be greater, but, owing to the large drop in effective stress during sampling, the resulting value of c/p is only 0.40 which is much lower than the value of 0.57 given by isotropic consolidation. This value of c/p is in good agreement with the relation between undrained strength and the value of the effective overburden pressure existing in the natural stratum.

While this test shows that the strength of samples consolidated in the field and sampled is quite consistent with that of a sample consolidated in the laboratory provided the natural condition of an anisotropic stress system is reproduced, and the operation of sampling simulated, it does not measure the resistance to shear of the ground in situ. It follows from the analysis made by *Hansen* and *Gibson* (1949), that the resistance to shear of the ground, under undrained conditions, varies with the direction of application of the shear stress, and is only approximately indicated by the undrained compression test on vertical samples (though except in silty clays the error is likely to be small).

In the present case we are concerned with the resistance of the ground to a horizontal shear stress, set up by the tendency of the embankment to spread by sliding laterally. Using an analysis similar in principle to that of *Hansen* and *Gibson* (1949), it can be shown (*Bishop*, 1952) that the change in pore-pressure Δu in a thin normally consolidated stratum, subjected to a change in total vertical stress Δp and a horizontal shear stress τ, given by the expression

$$\Delta u = \Delta p + p_0\left(\frac{1-K_0}{2}\right) + (2A-1)\left\{p_0^2\left(\frac{1-K_0}{2}\right)^2 + \tau^2\right\}^{\frac{1}{2}} \quad (1)$$

where p_0 = denotes initial effective overburden pressure
K_0 = denotes coefficient of earth pressure at rest
τ = denotes the horizontal shear stress
A = denotes the rate of increase of pore-pressure with deviator stress, defined by the expression $\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)$, $\Delta\sigma_1$ and $\Delta\sigma_3$ being the major and minor principal stress changes.

Under conditions of zero drainage failure will therefore occur when the value of the apparent cohesion c is given by the expression:

$$\left(\frac{c}{p_0}\right)\operatorname{cosec}\varphi' = \frac{1+K_0}{2} + \frac{c'}{p_0}\cot\varphi' - (2A-1)\left\{\left(\frac{1-K_0}{2}\right)^2 + \left(\frac{c}{p_0}\right)^2\right\}^{\frac{1}{2}} \quad (2)$$

where c' and φ' are the shear strength parameters from the Mohr envelope in terms of effective stress. (This method avoids the necessity of determining the Hvorslev shear parameters and is a good approximation where a common effective stress envelope can be drawn to all normally consolidated failure circles.)

In this case the measured values of c' and φ' were 0 and 33 degrees respectively, and the value of A at failure obtained for the normally consolidated soil in which no stress reversals had occurred (Fig. 8) was 0.49. The value of K_0 is taken as 0.43, and equation (2) then gives the value of c/p for horizontal shear as 0.39.

The conclusion may therefore be reached that the isotropically consolidated undrained test gives a misleading and unsafe result as conventionally used for predicting in situ strength or the gain in strength with one dimensional consolidation. The effect is likely to be less marked in soils having a low angle of true internal friction, but test results on clay (*Victor de Mello*, 1951) and on sand (*Bishop* and *Eldin*, 1953) indicate that it is a phenomenon of general importance. It is of interest to note that the estimated resistance to horizontal shear is given within 8 per cent by the undrained tests, but is overestimated by 46 per cent by the isotropically consolidated undrained test.

Acknowledgments

Acknowledgments are due to Mr. *J. C. L. Train* of the Railway Executive, to the Civil Engineer of the Southern Region, British Railways, and to Mr. *T. E. Hawksley* for permission to publish the data relating to the undisturbed samples. The work was carried out in the Department of Civil Engineering, Imperial College, University of London.

References

Bishop, A. W. (1950): Summarised proceedings of a conference on stress analysis. Brit. J. App. Phy.

Bishop, A. W. (1952): Ph.D. Thesis. London.

Bishop, A. W. and *Eldin, A. K. Gamal.* (1953): The effect of stress history on the relation between φ and porosity in sand. Proc. Third Int. Conf. Soil Mech., vol. I, p. 100.

Glossop, R. and *Golder, H. Q.* (1944): The construction of pavements on a clay foundation soil. Inst. Civ. Eng. Road Paper No. 15.

Hansen, J. B. and *Gibson, R. E.* (1949): Undrained shear strength of anisotropically consolidated clays. Géotechnique I, 3, 189-204.

Mello, V. B. F. de (1951): Ensais de Compressão Triaxial de Argilas con Medida de Pressos Neutras. Anais da Ass. Brasil de Mec. dos Sol I, 31–41.

Skempton, A. W. (1948): A study of the immediate triaxial test on cohesive soils. Proc. Second Int. Conf. Soil Mech. I, 192–196.

Taylor, D. W. (1948): Fundamentals of Soil Mechanics. John Wiley and Sons, New York, p. 397.