present findings to heterogeneous soil conditions, especially in the case of decreasing permeability above the zone of saturation.

Yours faithfully,

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13 October, 1960.

REFERENCES

CHAPMAN, T. G., 1957*a*. "Two-dimensional ground-water flow through a bank with vertical faces." *Géotechnique*, 7 : 1 : 35.

CHAPMAN, T. G. 1957*b*. *Correspondence on* "Two-dimensional ground-water flow through a bank with vertical faces." *Géotechnique*, 7 : 3 : 141.

The Secretary,
The Institution of Civil Engineers.

DEAR SIR,

THE PORE-PRESSURE COEFFICIENT IN SATURATED SOILS

In 1948 I suggested that the pore-pressure change in an element of saturated clay could be expressed in terms of the changes in total stress by the equation (for the undrained condition):

$$\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3) \qquad (1)$$

where A is a coefficient depending on the nature of the clay, and not necessarily independent of stress. In particular it was shown that A would be greater in plane strain than in the triaxial compression test, but the case of extension was not considered.

This formula proved to be of some value in practice, and in 1954 it was further developed to cover the case of partially saturated soils, by introducing the coefficient B. In the same Paper I also pointed out that the pore-pressure equation is, from a physical point of view, best written in a form which, for saturated soils, and with $\Delta\sigma_2 = \Delta\sigma_3$ (as in the triaxial test), is as follows:

$$\Delta u = \tfrac{1}{3}(\Delta\sigma_1 + 2\Delta\sigma_3) + (A - \tfrac{1}{3})(\Delta\sigma_1 - \Delta\sigma_3) \qquad (2)$$

The above equation was derived from a study of various modifications of elastic theory to allow for dilatancy, made in 1951–52 by Dr R. E. Gibson and myself, and based in part on proposals by Reiner (1948) and Odenstad (1949). At that time we concluded that the simplest expression for volume change which could account even approximately for the observed behaviour of soils was of the type*:

$$\frac{\Delta V}{V} = -C\left(\frac{\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3'}{3}\right) + D\sqrt{(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2} \quad (3)$$

where C is a coefficient of compressibility and D is a coefficient of dilatancy; and both these parameters, like the coefficient A, are not necessarily independent of stress. In saturated soils in the undrained condition $\Delta V = 0$. Hence:

$$\Delta u = \tfrac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3) + \alpha\sqrt{(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2} \quad (4)$$

where

$$\alpha = -D/C$$

and when $\Delta\sigma_2 = \Delta\sigma_3$ we obtain an expression of the same form as equation (2).

Since that part of the volume change or pore-pressure change associated only with a change in shear stress cannot depend on the sign of the shear stress, the positive root must be used in equation (4) or the module of the deviator in equation (2). Also, in these equations

$\Delta\sigma_1$ may be greater or less than $\Delta\sigma_3$. To make these points explicit equation (2) may be written:

$$\Delta u = \tfrac{1}{3}(\Delta\sigma_a + 2\Delta\sigma_r) + \alpha\sqrt{2}.|\Delta\sigma_a - \Delta\sigma_r| \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

where $\Delta\sigma_a$ is the increment in axial stress and $\Delta\sigma_r$ the increment in the two equal radial stresses.

It will be noted that $\alpha\sqrt{2}$ is numerically equal to $(A - \frac{1}{3})$ and it is a matter of choice as to which coefficient is used. In problems lacking axial symmetry, however, when equation (4) becomes operative, α is more convenient.

In triaxial compression tests equations (1), (2), and (5) are identical, but in triaxial extension tests equation (1) can lead to results appreciably different from those given by equations (2) or (5). And the comprehensive series of experiments carried out during 1952–58 under the supervision of Dr D. J. Henkel† has shown that equation (2) or (5) is preferable to equation (1). Even so, the values of A at failure are not identical both in compression and extension.

But for practical calculations it seems reasonable, at present, to use equations (2) or (5), while realizing that the pore-pressure coefficient defined in this way is not strictly a constant. And, by inference, equation (4) can similarly be employed as an approximation in the more general case with three unequal stress increments (Henkel, 1960). This expression has, for example, been found useful in analysing a short-term failure in a deep open excavation in London Clay.

Yours faithfully,

Imperial College, London. A. W. SKEMPTON, D.Sc.(Eng.), M.I.C.E.
4 November, 1960.

* This form of expression was adopted in connexion with the volume changes in sand beneath pile foundations (Skempton, Yassin, and Gibson, 1952).

† Much of the data from these tests is conveniently summarized by Parry (1960).

REFERENCES

HENKEL, D. J., 1958. "The correlation between deformation, pore-water pressure, and strength characteristics of saturated clays." *Ph.D. Thesis. Univ. of London.*

HENKEL, D. J., 1960. "The shear strength of saturated remoulded clays." *Proc. Conf. Shear Strength (Am. Soc. C.E.).* In the press.

ODENSTAD, S., 1949. "Stresses and strains in the undrained compression test." *Géotechnique*, 1 : 4 : 242–249.

PARRY, R. H. G., 1960. "Triaxial compression and extension tests on remoulded saturated clay." *Géotechnique*, 10 : 4 : 166.

REINER, M., 1948. "Relation between stress and strain in complicated systems." *Proc. Int. Cong. Rheology, Amsterdam*, 4 : 44–63.

SKEMPTON, A. W., 1948. "The effective stresses in saturated clays strained at constant volume." *Proc. 7th Int. Cong. App. Mech.*, 1 : 378–392.

SKEMPTON, A. W., 1954. "The pore-pressure coefficients **A** and **B**." *Géotechnique*, 4 : 4 : 143–147.

SKEMPTON, A. W., A. A. YASSIN, and R. E. GIBSON, 1953. "Théorie de la force portante des pieux dans le sable" ("Theory of the bearing capacity of piles in sand"). *Ann. Bâtiment Trav. Publics*, 16 : 285–290.

The Secretary,
The Institution of Civil Engineers.

DEAR SIR,

GAMMA-RAY AND NEUTRON METHODS OF MEASURING SOIL DENSITY AND MOISTURE
by A. C. Meigh, and B. O. Skipp

May I draw your attention to what appears to be an incorrect statement in a footnote on p. 111 of the September issue of *Geotechnique.*

Referring to gamma radiation it is stated that "high energy indicates high velocity radiation". It should be noted that gamma rays are emitted as photons of discrete quanta of electromagnetic energy and travel with the velocity of light—which is constant in a given medium—the rest mass of gamma photons being zero.