In-Situ Measurement of Soil Properties with the Pressuremeter

By R. E. Gibson, B.Sc.(Eng.), Ph.D., A.M.I.C.E. and W. F. Anderson, M.A., A.M.I.C.E.

The salient characteristic of drift geology is that soil, besides having wide variations in basic structure, is extremely heterogeneous. It is this attribute which has given rise to the Ménard Pressuremeter, which is an instrument designed to carry out a large number of in-situ load tests on the soil at varying depths in a borehole, and in this way to arrive at an average of the soil parameters that will enable the engineer to forecast its behaviour under load.

The aim of this article is to describe the instrument and some of the field tests that have been used in its calibration, and to outline a theory for the evaluation of test results.

THE standard pressuremeter (Fig. 1) consists of two main components—the probe which goes down the borehole, connected by plastic tubes to the pressure-volumeter which is the surface instrument.

The probe is a cylindrical metal assembly which grips rubber membranes to form three independent cells capable of being inflated to approximately twice their original diameter by CO_2 gas pressure applied from a bottle at the surface.

The central cell is designed so as to apply a radial pressure in the borehole and simultaneously to measure the increase in diameter of the hole; the outer guard cells expand under equal pressure and reduce end effects in the central measuring cell. Water is present in the measuring cell so that the volumetric strain of the probe is recorded by the drop in water level in the volumeter at the surface.

A programme of timed readings over two-minute periods at constant pressure it made, the pressure steps being chosen so as to give at least eight points on the pressure-volume graph.

Corrections

At depths beyond 30ft the hydrostatic effect of the column of water in the measuring cell system becomes important, and it is necessary to introduce a similar effect into the guard cells; this is done by using a second pressure-volumeter in the surface instrument.

The resistance of the rubber membrane must be allowed for, as must that of the elastic protective sheath which is usually placed over the membrane; this is calibrated by a surface test to full volumetric expansion, which gives a graph for the appropriate deduction in pressure at each position of volumetric strain.

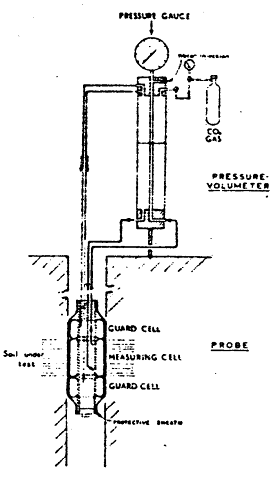


Fig. 1.
The standard pressuremeter showing its main components.

The nylon tubes dilate a small amount under pressure, and thereby produce a reading for volumetric strain which must be corrected; this effect can be calibrated by carrying out a surface test with the probe confined in a pipe, so that all expansion takes place in the leads. This compensation will be a function of pressure, length of nylon leads and ambient temperature.

In gravels and coarse sands, where the borehole is liable to collapse, the test can be carried out through a specially prepared section of drilling casing, with six 2mm wide slots 42in long cut at 60deg on the axis of the tube. These segments have a very small bending moment and the difference in pressure is about 1·4 lb/sq.in in the probe at full volumetric expansion.

Pressuremeter tests can be carried out above or below the water table without difficulty. Where necessary, the borehole can be kept open by using bentonite injection, the specific gravity of the mud being about 1·05. Since the pressure due to this head of mud slightly exceeds the pore water pressure in the adjacent soil, it will induce a small inflow into the borehole walls that will cease as soon as the mud in suspension has sealed the pores.

It is obvious that some disturbance effect must take place on the borehole wall by the auger in the course of making the hole, but the use of bentonite injection avoids the necessity for pulling the rods and the effect is believed to be small. This is borne out by results obtained in sensitive clays, when Pressuremeter values corresponded closely to Vane Test values and were, in general, higher than laboratory tests on samples (1)*.

Theoretical Considerations

In order to interpret the test data and obtain the Young's modulus and strength characteristics of the soil, it is necessary to derive a theoretical relationship between the volume of the measuring cell and the applied radial pressure. The pressuremeter test is well adapted to measure the Young's Modulus and apparent cohesion of clays under undrained conditions, and the Young's Modulus and angle of shearing resistance of sands under drained conditions, and it is these cases we shall consider in the following theoretical development.

Suppose a borehole of radius a_o is sunk in clay in the dry (if the test is carried out below the water table then appropriate modifications can be made at a late stage in the analysis). As a consequence the total radial pressure in the soil is reduced from p_o to zero, and if the cell pressure is then increased again to this value then, ideally, the original conditions will be restored. If the radial total stress is increased by a further amount Δp, and if the clay is assumed to respond linearly up to a deviator stress of $2c$ (where c is the undrained shear strength of the clay), then the relationship between Δp and the radius of the borehole can be calculated on the basis of an elastic analysis. The mode of deformation of the soil will be approximately two-dimensional in a radial plane, but it will correspond exactly neither to a condition of plane stress nor of plane strain. We shall assume as a first approximation that a condition of radial plane strain holds, and then the radial and circumferential components of strain will be related to the changes in the principal total stress components by the equations:

$$E\Delta e_r = (1-\nu^2)\Delta\sigma_r - \nu(1+\nu)\Delta\sigma_\theta = -E(du/dr) \quad . (1)$$

$$E\Delta e_\theta = (1-\nu^2)\Delta\sigma_\theta - \nu(1+\nu)\Delta\sigma_r = -Eu/r \quad . (2)$$

* *Figures in parentheses refer to the bibliography following the article.*

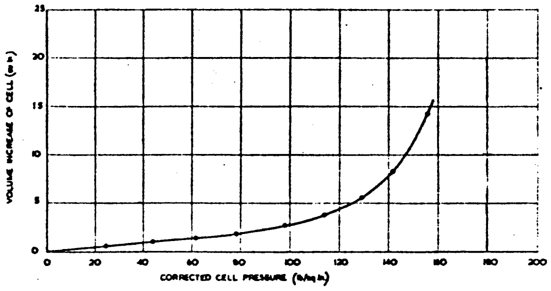


Fig. 2. Curve relating increase in cell volume to internal pressure.

where $u(r)$ is the corresponding radial displacement, and where compressive stress is taken as positive.

Since

$$\sigma_r = p_o + \Delta\sigma_r \quad \ldots . (3)$$

and

$$\sigma_\theta = p_o + \Delta\sigma_\theta \quad \ldots . (4)$$

it follows that

$$\sigma_\theta = p_o - [(1-\nu)E/(1+\nu)(1-2\nu)] \{u/r + [\nu/(1-\nu)](du/dr)\} \quad \ldots . (5)$$

$$\sigma_r = p_o - [(1-\nu)E/(1+\nu)(1-2\nu)] \{du/dr + [\nu/(1-\nu)](u/r)\} \quad \ldots . (6)$$

Now, at every point in the clay the equation of equilibrium

$$d\sigma_r/dr + (1/r)(\sigma_r - \sigma_\theta) = 0 \quad \ldots . (7)$$

must be satisfied. If the stresses are eliminated between equations 5 and 6 and 7 we find that the elastic displacement satisfies the equation

$$r^2(d^2u/dr^2) + r(du/dr) - u = 0$$

of which the general solution is

$$u = A/r + Br \quad \ldots . (8)$$

where A and B are arbitrary constants. Since

$$u \to 0 \text{ as } r \to \infty$$

it is necessary that $B = 0$. To determine A we note from equations 6 and 8 that

$$\sigma_r = p_o + [E/(1+\nu)](A/r^2)$$

and since

$$\sigma_r = p_o + \Delta p \quad \text{when } r = a_o \quad \text{then}$$

$$A = \Delta p[(1+\nu)/E]a_o^2$$

where Δp is the increase in the radial pressure in the borehole above the original in-situ horizontal stress p_o. Therefore, so long as the soil remains in the elastic range, the total radial stress at a distance r from the centre line of the borehole will be

$$\sigma_r = p_o + \Delta p(a_o^2/r^2) \quad \ldots . (9)$$

and the radial displacement of the surface of the borehole will be

$$u(a_o) = \Delta p[(1+\nu)/E]a_o \quad \ldots . (10)$$

This last equation enables the parameter $E/(1+\nu)$ to be found from the results of the pressuremeter test during which the relationship between $u(a_o)$ and Δp is determined.

The above analysis is valid only if the stresses in the clay are everywhere below yield, that is if

$$|\sigma_r - \sigma_\theta| < 2c \text{ for } r \geqslant a_o$$

Now, from equations 5 and 6

$$|\sigma_r - \sigma_\theta| = 2\Delta p(a_o^2/r^2) \quad \ldots . (11)$$

and it follows that plastic yielding will first occur on the surface of the borehole when the internal applied pressure attains a magnitude

$$p = p_o + c \quad \ldots . (12)$$

and at this stage the radius of the borehole will be

$$a_o + u(a_o) = [(1+\nu)/E]a_oc \quad \ldots . (13)$$

The above analysis may be carried out under the alternative assumption that the deformation is of the radial plane stress type, but it emerges that the relevant relationships remain unaltered.

If the internal pressure in the borehole is increased above $p_o + c$ then a plastic annulus of clay will extend from the surface of the borehole $r = a$ to a radius $r = R$, where both a and R are functions of p which are unknown at this stage.

We first determine the stresses in the plastic annulus $a \leqslant r \leqslant R$ where the equation of equilibrium 7 must hold, and throughout which a state of failure exists:

$$|\sigma_r - \sigma_\theta| = 2c, a \leqslant r \leqslant R \quad \ldots . (14)$$

From equations 7 and 14, and using the condition that

$$\sigma_r = p \text{ when } r = a$$

we find that

$$\sigma_r = p - 2c \log_e (r/a) \quad \ldots . (15)$$

At the interface $r = R$ between the elastic and plastic regions

$$\sigma_{r=R} = p - 2c \log_e (R/a)$$

and the radial elastic movement from the time when the internal pressure was p_o, of points located currently at $r = R$ will, from equations 10 and 15, be

$$u(R) = [(1+\nu)/E]R[p - 2c \log_e (R/a) - p_o] \quad . (16)$$

It is also known that at $r = R$, just inside the elastic region, the clay is on the point of becoming plastic, and it follows from equations 12 and 15 that

$$p = p_o - 2c \log_e (R/a) = c \quad \ldots . (17)$$

and, therefore, from equations 16 and 17

$$u(R) = [(1+\nu)/E]Rc \quad \ldots . (18)$$

In order to resolve the problem it is necessary to postulate that plastic deformation is equivoluminal and this will, of course, be strictly true in an undrained test. The clay lying currently in the annulus $a \leqslant r \leqslant R$ has a volume per unit height $\pi(R^2 - a^2)$. This same clay existed at the stage of the test when the ground pressure has just been restored ($p = p_o$) in the interval

$$a_o + p_o[(1+\nu)/E]a_o \leqslant r \leqslant R - u(R)$$

with a volume per unit height

$$\pi[\{R - u(R)\}^2 - \{a_o + p_o[(1+\nu)/E]a_o\}^2]$$

These volumes will be equal in the undrained test, and using equation 18 this condition requires that

$$(R/a)^2 + (a_o/a)^2p_o/c = [E/2(1+\nu)c][1-(a_o/a)^2] \quad . (19)$$

where small quantities have been ignored. Therefore, from equations 17 and 19 the current radius a of the borehole is related to the internal pressure p by the expression

$$p = p_o + c + c \log_e\{[E/2(1+\nu)c][1-(a_o/a)^2] - (a_o/a)^2p_o/c\} \quad . . (20)$$

The theory of the pressuremeter test on clays in the undrained condition may be summarised as follows:

Stage 1. The pressure increases from zero to the original in-situ horizontal stress p_o, and during this stage

$$\Delta V/V_o = 2p(1+\nu)/E, \; 0 \leqslant p \leqslant p_o \quad \ldots . (21)$$

where V_o is the original volume of the pressuremeter cell and ΔV is the increase in volume.

Stage 2. The pressure increases from p_o to $(p_o + c)$ at which pressure plasticity is initiated, and

$$\Delta V/V_o = 2p(1+\nu)/E, \; p_o \leqslant p \leqslant p_o + c \quad . . . (22)$$

Stage 3. The pressure increases above $(p_o + c)$ and during this stage

$$p = p_o + c + c \log_e\{[E/2(1+\nu)c]\Delta V/V - (1 - \Delta V/V)p_o/c\} \quad . . (23)$$

where $V = V_o + \Delta V$. The limiting pressure at which indefinite expansion of the hole occurs is

$$p_L = p_o + c\{1 + \log_e [E/2c(1+\nu)]\} \quad \ldots . (24)$$

This formula for the limit pressure p_L was obtained earlier by Bishop, Hill and Mott (2), and a later more exact analysis (3) agreed with this expression only for an incompressible medium ($\nu = 0{\cdot}5$) which is the case with which we are concerned.

Theoretical Considerations for Sands

The preceding analysis for clays may be modified to the case of a drained test on sands. It will be assumed that the relationship between effective stress and strain for sand is linear up to yield, and in these circumstances the elastic analysis given previously applies without alteration to the present problem. The parameters E and ν are, however, to be interpreted as the elastic constants of the grain skeleton.

The analysis is as before, except that the criterion of failure of equation 14 under undrained conditions is replaced by:

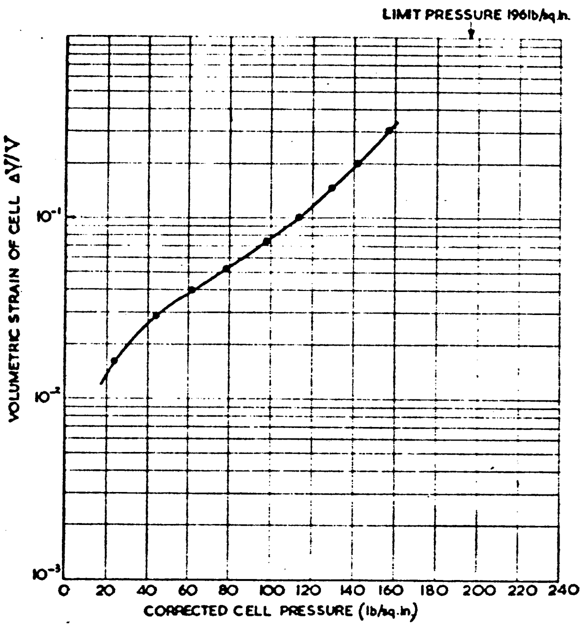
$$\sigma'_\theta/\sigma'_r = (1 - \sin\phi)/(1 + \sin\phi) = N$$

and primed effective stresses (total stress minus water pressure) are used throughout. It can be shown that prior to the formation of a plastic annulus

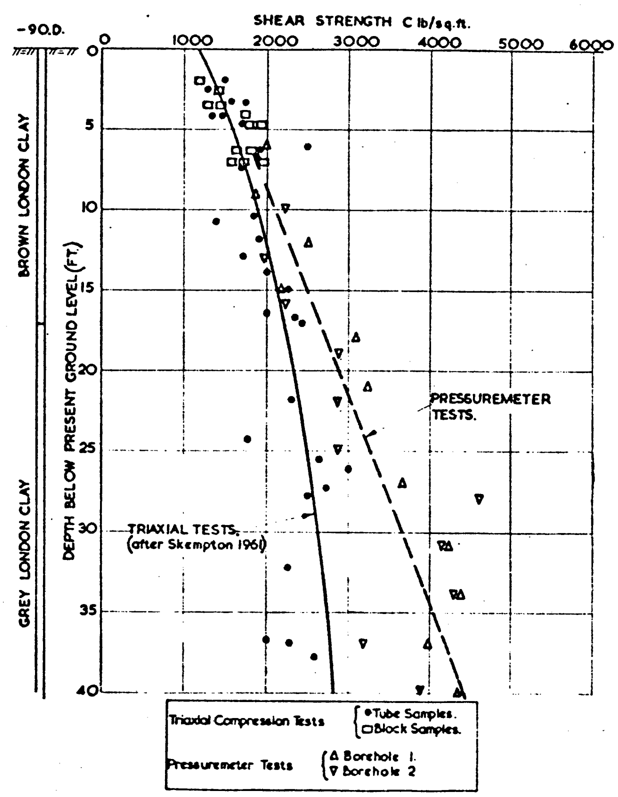
$$\Delta V/V_o = 2(p' - p_o')(1+\nu)/E$$

Plasticity will be initiated at a radial effective pressure

$$p' = 2p_o'/(1 + N)$$



Above: Fig. 3. Volumetric strain of cell plotted against corrected cell pressure.



Right: Fig. 4. Relationship between undrained strength and depth of London clay at Bradwell, Essex.

and thereafter the increase in volume of the cell will be connected with the internal pressure p' by the equation

$$p' = [2p_o'/(1 + N)]\{[E/2p_o'(1 + \nu)][(1 + N)/(1 - N)]\Delta V/V\}^{1/2(1-N)} \quad . . (25)$$

if the ratio p_o'/E is small compared with unity which is usually the case. A somewhat similar expression for the limit pressure, when $\Delta V/V = 1$, has been obtained by Kerisel (4) and is also given in reference (5). The orginal horizontal effective stress p_o' in the ground at the depth at which the test is carried out is related to the total overburden pressure p_r and the groundwater pressure p_w at this point by the equation:

$$p_o' = K_o(p_r - p_w)$$

where the coefficient of earth pressure at rest K_o ranges between about 0·35 for dense sand and about 0·5 for loose sands (6).

Analysis of Test Results in London Clay

A series of undrained tests on the London Clay has recently been carried out using the pressuremeter at a site near Bradwell, Essex. These test results enable a comparison to be made with the strengths obtained at this site by standard sampling and testing procedures, and also provide an example of the method of analysing pressuremeter test data to determine the ground strength. A typical curve relating increase in cell volume to internal pressure is given in Fig. 2 which refers to a test carried out at an elevation of —43 O.D. From this curve, and using equation 22, the undrained modulus E in the pressure range 0 — 60 lb/sq.in, taking Poisson's ratio ν as 0·5, is found to be 4,243 lb/sq.in.

The limit pressure p_L cannot, of course, be measured directly since this would imply infinite expansion of the borehole. However, by rewriting equation 23 in the form

$$p = p_L + c \log_e [\Delta V/V - 2(1 - \Delta V/V)(1 + \nu)p_o/E] \quad . (26)$$

the results may be presented (Fig. 3) in the form of a curve of p (arithmetic scale) against $\Delta V/V$ (logarithmic scale). The initial part of the curve up to a pressure of about 80 lb/sq.in corresponds to the " elastic " behaviour of the clay, while the remainder of the curve is approximately linear (strictly so if p_o/E is small compared with $\Delta V/V$) and corresponds to the " elastic-plastic " phase. To determine the undrained shear strength c of the clay from this part of the curve we select two points on it say $\Delta V/V = 0{\cdot}1$, $p = 114$ lb/sq.in and $\Delta V/V = 0{\cdot}3$, $p = 156$ lb/sq.in. Using these values in equation 26 we find that

$$c = \frac{42}{\log_e\left[\dfrac{0{\cdot}3 - 2{\cdot}1p_o/E}{0{\cdot}1 - 2{\cdot}7p_o/E}\right]} \text{ lb/sq.in} \quad . . . . (27)$$

In order to determine c from this expression some estimate of p_o must be made. Recent research carried out by Prof. A. W. Skempton on the London Clay at Bradwell (7) has shown that p_o is about 1·7 of the overburden pressure at this depth, and therefore

$$p_o = (120 \times 34 \times 1{\cdot}7)/144 = 48{\cdot}1 \text{ lb/sq.in}$$

where the average bulk density of the clay has been taken as 120 lb/cu.ft. The ratio p_o/E is therefore 0·0133 and from equation 27 the shear strength of the clay is 30·7 lb/sq.in. Although the above calculation of ground strength has relied upon a prior knowledge of p_o this is not in fact necessary; p_o can be calculated from the curve itself but this involves the solution of a transcendental equation. This may be avoided since a reasonable estimate of p_o can usually be made, and the accuracy of this estimate can then be assessed in the following way.

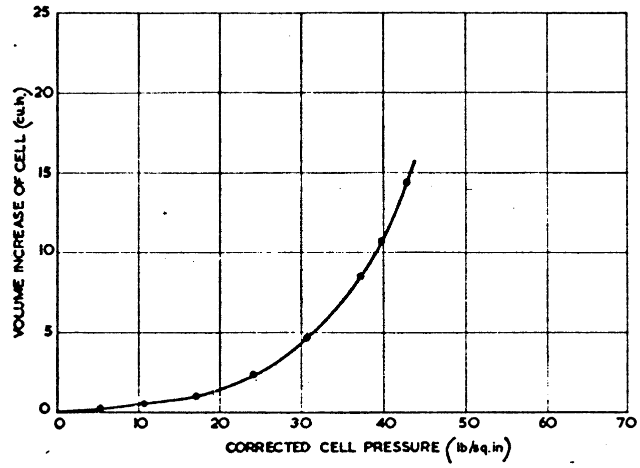
From equation 23 at $\Delta V/V = 0{\cdot}3$ (say) we may write

$$p_o = 156 - 30{\cdot}7 - 30{\cdot}7 \log_e\left[\frac{3 \times 4{,}243 \times 0{\cdot}3}{30{\cdot}7} - \frac{0{\cdot}7 \times 48{\cdot}1}{30{\cdot}7}\right]$$
$$= 47{\cdot}7 \text{ lb/sq.in}$$

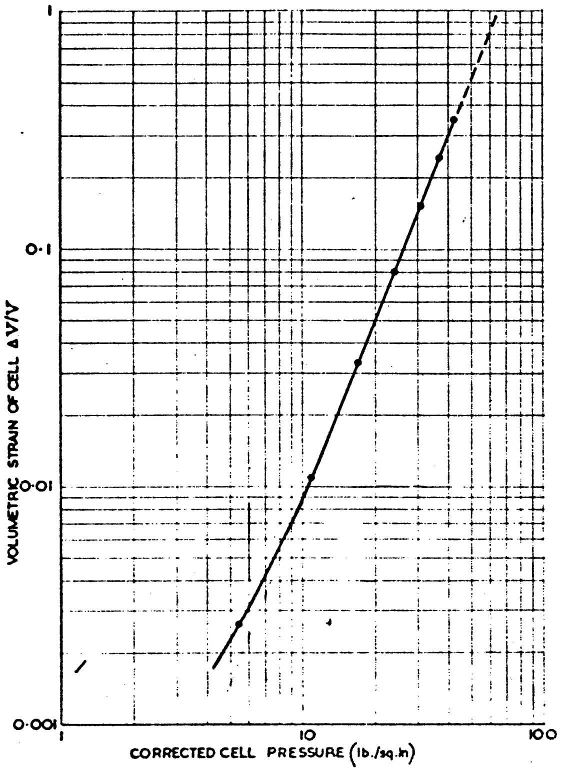
which agrees, for this case, very closely with the value of 48·1 lb/sq.in assumed in determining c.

At depths less than 20ft below ground level the ratio p_o/E was so small that this term could be ignored in equation 27 when computing the clay strength and this circumstance greatly simplified the numerical work.

The undrained strength-depth relationship at Bradwell is shown in Fig. 4, and compared with the strength obtained from undrained triaxial tests (7) on samples taken from nearby boreholes. The pressuremeter strengths are consistently higher than the triaxial test strengths and this difference increases with depth.



Above: Fig. 5. Volume increase of cell plotted against corrected cell pressure.



Right: Fig. 6. Volume strain of cell plotted against corrected cell pressure.

The reasons for this have not yet been fully explained, but it is known that the shear strength mobilised on a vertical plane—as in the pressuremeter test—is greater than on planes inclined at about 45deg to the horizontal—as in the triaxial compression test—due to the higher lateral effective pressure in the ground compared with the effective overburden pressure (8). Moreover, the strengths from hand trimmed block samples, driven tube samples, and from the pressuremeter tests agree well to a depth of about 10ft below ground level, and the lower strengths obtained from driven tube samples at greater depths may be due in part to sample disturbance. An investigation of the effects of this type of sample disturbance on the strength of London clay has recently been carried out by Ward, Samuels and Butler (9). Their studies suggest that a reduction of strength of 20 per cent due to open drive sampling may occur in the deeper and stronger parts of the London clay. Finally, the linear increase of c with depth above 40ft (Fig. 4) is unlikely to be maintained at greater depths.

Pressuremeter Tests on Sands

The pressuremeter is particularly well adapted for measuring the strength and compressibility characteristics of sands and gravels in-situ since no sampling is involved and disturbance can be kept to a minimum. The test in these soils is conducted under fully drained conditions and it is essential therefore that adequate opportunity be given for the excess pore water pressures to dissipate before volume readings are taken, for example, from saturated sands.

No detailed comparative study has yet been undertaken between soil properties measured by the pressuremeter and by other methods. Indeed, apart from laboratory tests which rely upon driven (10) or recompacted samples, no field tests for measuring E and ϕ for undisturbed non-cohesive soils exist, and in practice reliance has usually been placed upon the results of Standard Penetration tests or Dutch deep sounding tests which do not measure these parameters directly, although some attempts have been made to establish some correlations.

However, an extensive series of tests were carried out on a fine-medium closely graded sand above the water table near Aldershot and the data from a typical test is given in Fig. 5. From the initial slope of this curve the soil parameter $E/(1 + \nu)$ was found to be 2,080 lb/sq.in, and the results of the test were plotted in the form of a curve of p' (logarithmic scale) against $\Delta V/V$ (logarithmic scale) in Fig. 6. The test points lie on an approximately straight line, which is in accordance with the predictions of theory (equation 15), and by extrapolation the limit pressure is found to be about 65 lb/sq.in. The slope of this line gives

$$\tfrac{1}{2}(1 - N) = 0{\cdot}386$$

and hence $\phi = 39$deg.

The angle of shearing resistance is rather greater than had been anticipated for this sand deposit, and this is supported by the results of the other pressuremeter tests which gave values of ϕ ranging from 37 to 49deg. On the basis of this limited evidence, it would seem, therefore, that the existing theory for sands is not entirely satisfactory for determining the angle of shearing resistance, and this is not altogether surprising in view of the assumption made in the theory that sand behaves as a perfect elastic material below yield. The possibility of deriving a more satisfactory theory is being explored at present.

Acknowledgments

The authors are particularly indebted to Professor A. W. Skempton for data on the clay at Bradwell and for allowing them to be used in the Paper.

BIBLIOGRAPHY

(1) Ménard, L. "Mésures in-situ des propriétés physique des sols." *Ann. des Ponts et Chaussées,* May 1957.
(2) Bishop, R. F., Hill, R. and Mott, N. F. "Theory of indentation and hardness tests." *Proc. Phys. Soc.,* Vol. 57, 1945, p. 47.
(3) Hill, R. *The Mathematical Theory of Plasticity.* Oxford, 1950, p. 125.
(4) Kerisel, J. "La méchanique des sols: recherches et investigations récentes." *Travaux,* Sept. 1958.
(5) Skempton, A. W., Yassin, A. A. and Gibson, R. E. "Théorie de la force portante des pieux." *Ann. Bâtiment et Trav. Publics,* March 1953.
(6) Bishop, A. W. "Test requirements for measuring earth pressure at rest." *Conference on Earth Pressure.* Brussels, 1958, Vol. 1, p. 2.
(7) Skempton, A. W. "Horizontal stresses in an overconsolidated Eocene clay." To be published in *Proc. 5th Int. Conf. Soil Mech.,* Paris, 1961.
(8) Brinch Hansen, J. and Gibson, R. E. "Undrained shear strength of anisotropically consolidated clays." *Geotechnique,* 1949, No. 1, p. 189.
(9) Ward, W. H., Samuels, S. G. and Butler, M. E. "Further studies of the properties of London clay." *Geotechnique,* 1959, No. 9, p. 33.
(10) Bishop, A. W. "A new sampling tool for use in cohesionless sands below ground water level." *Geotechnique,* 1948, No. 1, p. 125.