

COMPARISONS OF THE RESULTS FROM PRESSUREMETER TESTS
AND LARGE IN-SITU PLATE TESTS IN LONDON CLAY

by A Marsland and M F Randolph

SUMMARY

Comparisons have been made of the engineering properties of stiff fissured London Clay determined from pressuremeter tests, using a 60 mm diameter probe in a pre-drilled borehole with the operational values determined from deep in-situ loading tests on 865 mm diameter plates in 900 mm diameter boreholes. The shear strengths determined from the pressuremeter tests are all higher than the operational strength. The ratio between the two strengths depends upon the method of interpretation, the in-situ lateral stresses, and the nature and size of the clay fabric in relation to size of the probe. Strengths determined from the pressuremeter tests range from 1.1 to greater than 3 times the operational values determined from the large in-situ plate tests. The most reasonable and consistent values determined from the pressuremeter tests are those obtained using elasto-plastic analyses incorporating good estimates of the limit pressures and the horizontal in-situ pressures. In order to obtain operational strengths it is necessary

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to reduce these values by factors which depend mainly on the nature and spacing of the discontinuities in the clay and, to a lesser extent, on the values of the effective normal stresses in the ground. For fissured London Clay this reduction factor increases from 1.1 near the surface to about 2.2 at a depth of 24 m. Shear strengths determined using analyses giving the stress-strain curves for small elements adjacent to the wall of the pressuremeter probe are appreciably greater than those determined from the limit pressures.

Moduli determined from the pressuremeter tests are appreciably lower than the operational values determined from large in-situ plate tests. It is, however, considered that the values determined from the re-load cycles can be used as a reasonable basis for the design of foundations. The dangers of using correlations between different types of small field or laboratory tests is pointed out.

RÉSUMÉ

Les propriétés mécaniques de l'argile de Londres, raide et fissurée sont déterminées par des essais pressiométriques avec sonde de 60 mm de diamètre en trou préforé, et en chargement à la plaque, de 865 mm de diamètre, en trous de sondage de 900 mm de diamètre. Toutes les résistances de cisaillement déterminées par les essais pressiométriques sont supérieures aux valeurs opérationelles déterminées par les essais de chargement à la plaque. Le rapport entre les deux résistances dépend de la méthode d'interprétation, des pressions horizontales naturelles du sol et de la nature et de la dimension de la structure de l'argile par rapport à la dimension de la sonde. Les résistances déterminées par les essais pressiométriques varient de 1.1 jusqu'à plus de 3 fois les valeur opérationelles déterminées par les essais de chargement à la plaque. Les valeurs les plus acceptables déterminées par essais pressiométriques sont obtenues par analyses élasto-plastiques à évaluation juste des pressions limites et horizontales naturelles. Pour obtenir les résistances operationelles, les valeurs doivent être réduites par un facteur dependant du type et de l'espacement des fissures de l'argile, et aussi, jusqu'à un certain point, sur les valeurs des pressions naturelles généralement présentes. Pour l'argile fissurée de Londres ce facteur de réduction s'accroît de 1.1 pres de la surface jusqu'à environ 2.2 à une profondeur de 24 m. Les résistances de cisaillement déterminées par analyses donnant les courbes contrainte-déformation pour petites tranches adjacentes à la sonde pressiométrique sont considérablement plus importantes que les valeurs ^{déterminées} opérationelles des pressions limites.

Les moduli de cisaillement determines par essais pressiométrique sont

considérablement inférieurs aux valeurs opérationnelles déterminées par essais de chargement à la plaque à l'emplacement. Cependant, on considère que les valeurs déterminées par les essais de rechargement peuvent être utilisées pour l'établissement des plans de fondations. Les dangers associés avec l'utilisation de comparaisons entre différents types d'essais: sur petite échelle à l'emplacement et en laboratoire, sont soulignés.

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INTRODUCTION

Measurements of the pressures required to inflate cylindrical probes installed in boreholes and the corresponding volume changes have been used for more than a decade to provide data for foundation design. Devices used for making such measurements are known as pressuremeters. In France they have been used in a wide range of soils and comprehensive semi-empirical rules have been developed for estimating both the end-bearing pressures and the allowable side shear mobilised by piles and other types of foundations (Menard, L F, 1957, 1963 and 1965).

In the United Kingdom the application of pressuremeter tests has been more limited and they have been used mainly to determine the properties of soft rocks such as Marls, Mudstones, Sandstones and Chalk (Meigh, A C and Greenland, S W, 1965; Hobbs, N B and Dickson, J C, 1970). Some early tests in the London Clay at a site near Bradwell, Essex, were analysed by Gibson, R E and Anderson W F (1961). These authors outlined a theory for the evaluation of soil parameters from the results of pressuremeter tests following similar lines to those adopted by Bishop, R F, Hill, R, and Mott N F (1945), and Hill, R (1950). The values of shear strengths they determined from the pressuremeter tests near the surface were in reasonable agreement with those obtained

from triaxial tests on 76 mm high x 36 mm diameter specimens. At lower levels the strengths they obtained from the pressuremeter tests were higher than those obtained from the triaxial tests. The ratio between the values determined from the two types of test gradually increased with depth until a value of 1.5 was reached at the maximum depth of 12 m. Gibson and Anderson suggested that these differences may have been due to the in-situ lateral effective stress being larger than the effective overburden pressures, or to the triaxial tests giving lower values as a consequence of sample disturbance. More recent studies on London Clay (Agarwall, K B, 1967; Bishop, A W and Little, A L, 1967; Marsland, A, 1967, 1971(a) and 1971(b)) have shown that triaxial tests on 76 mm high x 38 mm diameter specimens often give strengths which are appreciably higher than those obtained from tests on larger samples and from large in-situ tests. It has been shown that the ratio of the dimensions of the volume of the soil tested to the dimensions of the intact lumps of clay between the fissures can seriously affect the measured shear strengths. As a consequence empirical correlations based on comparisons between small in-situ field tests such as the pressuremeter, vane and the cone penetrometer test (Marsland, A, 1974), and laboratory tests may be misleading unless the possible scale effects are taken into account. The present paper compares the results of pressuremeter tests with those obtained from loading tests on 865 mm diameter plates installed in 900 mm diameter boreholes to depths of 25 m.

LOCATION OF SITE AND PROPERTIES OF THE CLAY

The tests described in this paper were made at a site located near the edge of the flood plain of the River Brent at Hendon approximately 10 km to the north-west of Central London. Here the London Clay extends to the surface

and the top 25 m contains 55 to 60 per cent of particles less than 2 μ m. The liquid limit of the clay varies between 60 and 80 per cent and moisture contents are approximately equal to the plastic limit which ranges from 30 per cent near the surface to 26 per cent at a depth of 25 m.

Down to a depth of 9 m the clay is weathered to a brown colour and consists of hard lumps set in a soft clay matrix. Near the surface the lumps have dimensions of only a few millimetres and are well separated by softened clay. The dimensions of the hard lumps gradually increase with depth until they have dimensions of 6 to 50 mm at a depth of 9 m. At this level there is only sufficient softened clay to provide a small adhesion between the lumps. Below 9 m it is a stiff, highly fissured grey-blue clay which separates easily along the fissures. The spacing of the fissures increases from 6 to 50 mm at a depth of 9 m to 75 to 325 mm at a depth of 25 m. The fissures are inclined at all angles, but there is a tendency towards a predominance of near-vertical and near-horizontal fissures, particularly at the deepest levels. Below a depth of about 15 m small quantities of water could be seen seeping from the sides of the borehole a few hours after drilling. Inspection of the clay below this level showed that there were traces of silt and fine sand on some of the horizontal bedding planes, particularly at depths of between 17 and 22 m. Water levels measured in Casagrande type piezometers installed at depths of 12 and 24 m gave equilibrium piezometric levels about 1 m below ground level.

LOADING TESTS ON 865 mm DIAMETER PLATES IN 900 mm DIAMETER BOREHOLES

In-situ loading tests on 865 mm diameter plates were made in 900 mm diameter boreholes at depths of 6.1, 12.2, 18.3 and 24.4 m below ground level. These tests form part of an extensive investigation of in-situ and laboratory tests on London Clay.

A general description of the equipment and test procedures has already been given (Marsland, A, 1971(b)). The equipment was designed with the object of keeping the time required to erect the loading and settlement measuring systems to a minimum. A helical flight auger was used to drill the 900 mm diameter borehole to within about 600 mm of the test level, and a flat-bottomed bucket auger was used to take out the remainder so as to produce a flat-bottomed hole. Loose and protruding clay was removed from the base of the hole by hand in all the tests and for some of the tests additional clay was removed from the base using a sharp spade and a hand scraper. As soon as the removal of spoil from the base of the hole was completed a 15 to 20 mm thick layer of a high-strength Gypsum plaster mix was spread over the base of the borehole and the loading plate carefully bedded into position. The average time between completion of the machine boring and the commencement of a loading test was about 40 minutes for the tests in which only the loose material was removed by hand from the test level. For tests where additional clay was removed from the bottom of the borehole by hand digging the average time interval was about 80 minutes. The settlement of the plate was transferred to ground level by means of an independent axial load-free reference column built up from lengths of 100 mm diameter tube. Dial gauges fixed to a 12 m long reference beam were used to measure the settlement. The end of the reference beam rested on rollers cast into concrete blocks which in turn rested on concrete pad foundations well removed from both the tension piles and the test borehole. During the loading test the jack was extended at a constant rate of 2.5 mm per minute using a multi-speed hydraulic pumping unit which ensured a very smooth penetration, free from pressure pulses.

These tests gave load/settlement curves with reasonably well defined maxima and a typical example is given in Figure 1. They did not show any obvious bedding errors which are common in smaller tests made in boreholes which are too small to permit access for cleaning the test surface by hand. Slight initial concavity of the load/settlement curve occurred in a few of the tests in which only the loose clay was removed from the base of the borehole prior to setting the plate in plaster. In tests where additional clay was dug from the base of the borehole by hand there was no sign of initial concavity in the load/settlement curve.

PRESSUREMETER TESTS

Description of equipment

The complete pressuremeter system is shown diagrammatically in Figure 2. It consisted of two separate units: a radially expandable probe which is installed in the borehole connected to a pressurising and volume measuring instrument (volumeter) resting on the ground surface. Details of the measuring equipment and probe used in the present series of tests are shown in Figure 3. The probe consisted of a hollow aluminium shaft surrounded by two rubber membranes. The inner membrane which formed the measuring cell was 216 mm long and filled with water. An outer membrane, 480 mm long, protected the inner membrane and also formed the guard cells at both ends of the measuring cell. Protection of the outer membrane was provided by overlapping longitudinal strips made of thin spring steel.

The water pressure was applied to the inner measuring cell of the probe via the inner section of a coaxial tube which connected it to the pressurised

water cylinder forming the volume measuring device. Expansion of the ends of the outer membrane was achieved by pressurised gas passing down the outer annular section of the connecting tube. The pressure of this gas was kept lower than that of the water in the measuring cell to ensure that the measuring cell always remained in contact with the outer membrane. The volumeter, which had a capacity of 800 cc, was split into 2 parallel tubes with diameters in the ratio of 10 to 1. In normal operation the water was supplied from both tubes, but for very small volume changes the outlet from the larger tube was closed. The water in the volumeter was pressurised by gas supplied from a high-pressure cylinder via a regulating valve. Gas to pressurise the guard cells at a slightly lower pressure was tapped from the pressure control supply to the volumeter via a second differential pressure valve.

The external diameter of the probe in its deflated condition was 58 mm, providing a measuring cell with a length of approximately 4 times its diameter. The guard cells helped to create a condition approaching plane strain in the region of the measuring cell by increasing the axial extent of the radial stress field.

Test procedures

A NX size casing was drilled in to 1 m above the proposed test level using a tricone bit. Below this, a test pocket 1.30 m deep having a diameter of approximately 60 mm was drilled with a BX sized tricone bit. The pressuremeter was installed as quickly as possible into the test pocket leaving a gap of about 0.15 m between the probe and the bottom of the pocket. The elapsed time between the end of drilling the test pocket and the commencement of

the test was about 15 minutes. After completing the first test the borehole was advanced to within 1 m of the next test level using the NX sized bit and casing and the same procedures were followed as before. Water was used as the flushing fluid during the drilling of borehole 1, while in borehole 2 a drilling additive was added to the water. The object of this additive was to produce a thin impermeable mud cake on the sides of the pocket as drilling progressed and thus reduce the ingress of water into the clay adjacent to the walls of the borehole.

Having lowered the probe to the correct level within the test pocket water was allowed to flow from the volumeter to inflate the measuring cell in the probe until the central section of the probe came into contact with the sides of the hole. Since the borehole was filled with water or drilling fluid the tests started at a total pressure equal to $\gamma_f z$, where γ_f is the density of the drilling liquid. For tests at depths less than 10 m the differential pressure valve was set so that the gas pressure applied to the guard cells was approximately 2 kg/cm^2 below that applied to the top surface of the water in the volumeter. For tests carried out at depths greater than 10 m the gas pressure in the guard cells was increased until the pressure was sufficient to start pushing the water back into the volumeter when the water in the volumeter was kept at atmospheric pressure. The gas pressure in the guard cells was then reduced by $1\text{--}2 \text{ kg/cm}^2$ using the differential pressure valve. This ensured that the gas pressure supplied to the guard cell throughout the test was kept this much lower than the water pressure applied to the measuring cell in the probe.

The size of the pressure increments was chosen to give 15 or 16 points on the pressure volume curve from the start of the test until the probe reached its maximum permissible extension when the radial strain was about 25 per cent. In practice this meant that pressure increments of 0.5 kg/cm^2 were used in the upper levels of the clay, while increments of 1.0 kg/cm^2 and 1.5 kg/cm^2 were applied for tests at depths of 10 m and 20 m respectively.

It took about 30 seconds to increase the pressure at each increment. As soon as the increment had been applied a zero reading was taken for the new pressure and further readings taken at 15 seconds, 30 seconds, 1 minute and 2 minutes before applying the next pressure increment. In this way the short-term creep occurring at each pressure was measured. The change in volume which occurred between 30 seconds and 2 minutes was plotted for each pressure to provide an indication of the amount of creep taking place. The total volume change of the measuring cell from the beginning of the test to the end of each pressure increment was plotted on the same graph. Curves for a typical test are shown in Figure 4, from which it can be seen that the creep readings are relatively small for pressure corresponding to the linear region of the $p - V$ curve, and only increase substantially when the clay adjacent to the test pocket starts to become plastic. This pressure is denoted by p_f . When this stage was reached the pressure was reduced incrementally down to a value approaching $\gamma_f z$ using the same pressure intervals as when the pressure was being increased. This unloading was followed by a re-loading cycle up to a pressure corresponding to p_f . Creep readings were not taken during the unloading and re-loading processes which were therefore substantially quicker than the initial loading

process. Further pressure increments were then applied, during which creep measurements were taken, until the probe was expanded to its maximum permissible size which was approximately twice its volume at the beginning of the test.

The field measurements were corrected as follows:

- (i) to obtain the total pressure acting on the inside wall of the measuring cell, the pressure due to the head of water between the water surface in the volumeter and the test level ($\gamma_w H$ in Figure 2) was added to the gas pressure applied to the top of the volumeter.
- (ii) the total pressure acting on the clay at the side of the test pocket is the pressure given by (i) above less the pressure required to stretch the rubber membranes and the metal protective sheath where this was used. Calibration tests were made by expanding the probe in the unconfined state and plotting the changes in volume against pressure. One result of this correction is that even though equal increments of gas pressure were applied to the volumeter, the pressure increments acting on the ground decreased as the applied pressure was increased. Towards the end of a test only about 10 to 20 per cent of the applied gas pressure was transferred to the clay under test.
- (iii) the increase of the volume of the measuring cell was smaller than the measured value due to the expansion of the volumeter and the tube connecting it to the pressure cell. Use of the coaxial connecting tube significantly reduced this effect and the corrections in the present tests were only about 1 cc over the full range of pressure.

The errors due to the expansion of the connecting tube and the influence of small enclosed air bubbles can be measured by pressurising the probe while it is inserted into a steel tube having an internal diameter slightly larger than the outside diameter of the deflated probe. This procedure also enables water levels in the volumeter to be related to the corresponding volumes of the measuring section of the probe.

It is recommended that the calibration outlined in (ii) and (iii) above should be made prior to every test and after the last test on a particular day.

It must be stressed that the procedures adopted in these tests were those currently used in commercial practice in the UK. While the procedures were reasonable in London Clay more attention could be given to the methods of calibration and pressure control. There is also a need for more research on the influence of the rates of testing and the method of formation of the test pockets.

EVALUATION OF SOIL PARAMETERS FROM PLATE TESTS AND PRESSUREMETER TESTS

Undrained shear strength

(1) Plate loading tests

The theoretical relationship for the bearing capacity of a deep foundation in clay is given in terms of the undrained shear strength (Terzaghi, K, 1943) by:

$$q_u = N_c \cdot c_u + \gamma_s z \quad (1)$$

where q_u = ultimate base pressure

N_c = the theoretical bearing capacity factor for the particular type of foundation

$\gamma_s z$ = total overburden pressure

c_u = undrained shear strength.

Approximate values of N_c for a deep circular foundation can be obtained in terms of the shear modulus, G , of the soil and its undrained shear strength by considering the base failure of the foundation as being analogous to the expansion of a spherical cavity. The limiting pressure at which continuous expansion of a spherical cavity occurs in a uniform isotropic elastic-plastic material and in which no volume change occurs, can be determined by the expression obtained by Bishop, R F, Hill, R and Mott, N F (1945):

$$p_L = \frac{4}{3} \left[1 + \log_e \frac{G}{c_u} \right] \cdot c_u + \gamma_s z \quad (2)$$

where p_L = limit pressure

$\gamma_s z$ = pressure in the ground prior to expanding the cavity

G = undrained shear modulus of the soil;

$G = E_u/3$ where E_u is the undrained Young's modulus.

This solution, for an expanding spherical cavity, was modified by Gibson, R E (1950) to give an expression for the bearing capacity of a deep foundation.

He took into account the differences in the failure mechanism of the two cases and obtained the following equation:

$$q_u = \left\{ \frac{4}{3} \left[\log_e \frac{G}{c_u} + 1 \right] + \cot \alpha \right\} c_u + \gamma_s z \quad (3)$$

where the $\cot \alpha$ term allows for the shear stress along the interface of the elastic-plastic zone which forms under the foundation or along the surface of a pointed pile. For a flat base loaded to give an undrained failure in clay $\alpha = 45^\circ$ and

$$q_u = \left\{ \frac{4}{3} \left[\log_e \frac{G}{c_u} + 1 \right] + 1 \right\} c_u + \gamma_s z \quad (4).$$

Comparing this expression with eq (1), values of N_c are obtained which range from about 5 for a material with G/c_u ratio of 7 to between 9 and 10 for one with G/c_u ratio greater than 150. The higher values are in agreement with the approximate values of N_c for piles penetrating into an ideal rigid plastic medium obtained by Meyerhoff, G G, (1951 and 1961). For flat-bottomed piles he obtained $N_c = 9.34$ when he neglected the effects on the base resistance of the shear forces acting on the vertical sides of the pile and $N_c = 9.74$ when the full effects of these shear forces were taken into account.

In the case of the loading tests on circular plates in boreholes, the shear forces acting on the vertical sides of the plate are small, and the value of $N_c = 9.34$ is appropriate, provided that the plate has the same diameter as the borehole. This value is also in good agreement with values obtained from model tests in reconstituted London Clay (Marsland, A, 1972). At present the most reliable values of N_c for undisturbed London Clay are those deduced from the ultimate bearing capacities measured in the loading tests on 865 mm

diameter plates at a depth of 6.1 m at the present site and the corresponding shear strengths measured in triaxial tests on 98 mm diameter samples. At this depth, good undisturbed samples were obtained and the fissures are sufficiently closely spaced for strengths measured on 98 mm diameter specimens to provide a reasonable measure of the large scale strength. Values of N_c varying from 8.7 to 9.65 with an average of 9.25 were obtained from these tests.

In this paper, the undrained shear strengths of the stiff London Clay have been determined from the plate tests using the relationship

$$c_u = \frac{q_T - \gamma_s z}{N_T} \quad (5)$$

where q_T = total load on the plate divided by the base area of the plate

N_T = 9.6 which is the relevant plate factor based on a theoretical bearing capacity factor of $N_c = 9.25$ with an allowance for adhesion to the vertical sides of the plate, equal to 60 per cent of the shear strength of the clay (thickness of plate = 0.15 x diameter of plate).

The values of c_u derived from the values of q_T measured on the large plate tests and using eq (5) are plotted against depth in Figure 5.

(2) Pressuremeter tests

In a pressuremeter test, the soil surrounding the central cylindrical measuring cell is deformed under conditions which correspond closely to plane strain.

The equivalent relationship to eq (2) for the limiting pressure required

to expand an infinitely long cylindrical cavity, derived by Bishop, R F, Hill, R and Mott, N F (1945) and Gibson, R E and Anderson, W F (1961), is:

$$p_L = \left[\log_e \frac{G}{c_u} + 1 \right] c_u + p_0 \quad (6)$$

where p_0 = horizontal in-situ stress prior to drilling.

The term $(\log_e \frac{G}{c_u} + 1)$, which is analogous to N_c in eq (1), will be defined as the pressuremeter constant N_p .

The stress-strain curves for saturated clays in undrained shear depart from the ideal elasto-plastic curve assumed in the derivation of eq (6) in that they are continuous curves with no well defined yield point. The shear modulus, defined either as a tangent or as a secant modulus does not remain constant but decreases continuously with increasing strain. Ladanyi, B (1963) obtained solutions for the expansion of cavities using the stress-strain curves obtained from triaxial tests on several clays. He showed that the limit pressure, p_L , depended on the shape of the stress-strain curve as well as on the ratio of the shear strength to some suitable shear modulus. However, on comparing his solutions with those obtained from eqs (2) and (6), he concluded that reasonable agreement could be obtained provided that the secant modulus over the range of deviator stress $0 \rightarrow \frac{1}{2} (\sigma_1 - \sigma_3)_f$ was used as the value of the shear modulus, G , in these equations.

In order to determine the values of c_u using eq (6), it is necessary to determine p_L and also to have reasonable estimates of p_0 and G .

(a) Estimation of p_L

The limit pressure p_L cannot be determined directly by measurement in a pressuremeter test since by definition it is the pressure at which continuous expansion of the cavity occurs.

The standard pressuremeter is inserted into an open borehole and as a consequence the start of the test is at a lower pressure than the in-situ horizontal stress, p_o . Gibson, R E and Anderson, W F (1961) assumed that the volume change which occurred during unloading from p_o to zero was directly proportional to the change of pressure. This implies that the $p - V$ curve is also linear during reloading from zero pressure to p_o . Using this assumption and taking the start of the tests from a pressure $p = 0$, and the corresponding volume $V = V'_o$ they obtained an expression which gave the pressures in the later stages of the tests when plastic deformation plays an ever increasing part as

$$p = p_L + c_u \log_e \left[\frac{\Delta V}{V} - \left(1 - \frac{\Delta V}{V} \right) \frac{p_o}{G} \right]$$

*A better formula than
this, original since G can be
determined off the plot of
 p_o estimated as a P17*

where ΔV = the increase in volume from the initial volume V'_o

V'_o = initial volume at $p = 0$

V = total volume at pressure p . ($V = V'_o + \Delta V$.)

Unfortunately the $p - V$ curves obtained from the pressuremeter tests on London Clay (see Figure 4) show that the initial reloading curve is very non-linear and this affects the relationship given in eq (7). The effects of reducing the stress from p_o to zero can be significantly reduced by taking the starting point of the test at a pressure $p = p_o$. The corresponding starting volume V_o can be read from the $p - V$ curve and subsequent volume changes measured

from this new initial volume. The new expression for p which replaces eq (7) is:

$$p = p_0 + c_u \log_e \left(\frac{\Delta V}{V} \right) \quad (8)$$

where V = total volume at pressure p

$$\Delta V = V - V_0$$

V_0 = volume at pressure p_0 .

This equation implies that the curve obtained by plotting p against $\log_e \frac{\Delta V}{V}$ should be essentially linear for the later stages of a test, provided that the assumption of an elasto-plastic material is reasonable. This means that extrapolation of the curve to $\log_e \frac{\Delta V}{V} = 0$, which corresponds to continuous expansion of the cavity ($\frac{\Delta V}{V} = \frac{\Delta V}{V_0 + \Delta V} = 1$, whence $\Delta V = \infty$), will provide a good estimate of p_L . That this is the case for London Clay can be seen from the curves in Figure 6. Values of p_L determined in this way are given in column (2) of Table 1. While this procedure requires the determination of p_0 it was found that p_L varied by only about 1% for a 20% change of p_0 .

(b) Estimated of p_0

It has already been indicated that a reasonable estimate of p_0 is important in the determination of c_u . The value of p_0 can not be determined directly from tests using a pressuremeter in a pre-drilled borehole. The choice of p_0 must be made by inspection of the pressure volume curve bearing in mind the following points:

- (1) p_0 should lie on the most linear part of the curve
- (2) Very small positive or negative increments of pressure from p_0 should provide a similar magnitude of volume change and p_0 will lie close to the point of inflexion (ie zero curvature) of this essentially linear portion of the pressure-volume curve.

- (3) Since in an elastic-plastic material a pressure increase of c_u above p_o causes plastic yield to occur in the clay adjacent to the probe, a marked increase of curvature should occur when the pressure exceeds $p_o + c_u$. It should be noted that the value of c_u at which this occurs is the peak value for the clay immediately adjacent to the probe as given by the Palmer analysis outlined later in this paper.

Since p_o must be determined in order to estimate c_u , condition (3) above implies some iteration whereby an initial estimate of p_o is made, the value of c_u is calculated, and then a check is made to determine whether the value of $p_o + c_u$ obtained corresponds to the pressure at which the curve becomes significantly non-linear. An example showing the successive stages of this process for a typical test is given in Figure 7. An initial estimate of p_o is made from available information on the consolidation history of the soil. For highly overconsolidated London Clay it was expected that K_o would be substantially greater than unity and an initial value of $K_o = 2.4$ was used (taken from Skempton, A W, 1961) and this gave a $p_o = 452 \text{ kN/m}^2$. A volume V_o corresponding to $p_o = 452 \text{ kN/m}^2$ was taken from the measured $p - V$ curve and the corresponding $p - \log_e \Delta V/V$ curve was plotted ($\Delta V/V = \Delta V/(V_o + \Delta V)$). The value of the peak shear strength obtained from this curve was approximately 180 kN/m^2 . It can be seen, Figure 7a, that the $p - y_1$ curve for this value of p_o is linear well past the pressure $p_o + c_u$ (y_1 is the radial extension of the borehole; see eq (12) below for definition). This process was repeated for other values of p_o in Figures 7b and 7c, where the points at which a marked increase in curvature became apparent are marked with an asterisk. The value of p_o for which this point most nearly corresponds to the pressure $p_o + c_u$ was chosen as the correct p_o . In this case it was approximately equal to 511 kN/m^2 as shown in Figure 7b. Values of p_o determined in this way are given in column 3 of Table 1 and plotted against depth in Figure 8a.

Values of p_o can also be determined from laboratory measurements of K_o combined with measurements of in-situ densities and pore water pressures using the relationship:

$$p_o = K_o (\gamma_s z - u) + u \quad \left| \quad K_o = \frac{\sigma_h - u}{\gamma_s z - u} \quad (9) \quad \neq \gamma_s = \right.$$

where $\gamma_s z$ = total overburden pressure

u = porewater pressure.

Values of K_o have been estimated from laboratory tests on London Clay from different locations by Skempton, A W (1961) and Bishop, A W, Webb, D L, and Lewin, P I (1965). These values have been used together with measurements of in-situ densities and pore water pressure made at the present site to provide estimates of p_o . The p_o depth profiles determined in this way are included in Figure 8a and it can be seen that they are very similar to those determined by the iterative procedure. The K_o depth profiles are shown in Figure 8b. Recent tests have been made down to a depth of 11 m at the present site using a Camkometer (Windle, D and Wroth, C P, 1977). These gave values of K_o ranging from 2.0 to 3.5 which are very similar to those obtained by the iterative procedure.

All these values of p_o are significantly higher than those determined from pressuremeter tests using the values of p at which the $p - V$ curves become reasonably linear and where the creep falls to a low constant value, as illustrated in Figure 4. Individual values of p_o determined in this way showed a large amount of scatter and in some cases gave values lower than the overburden pressure which is unrealistic for highly overconsolidated London Clay.

(c) Estimation of c_u

Estimates of c_u can be obtained from eq (6) by a simple iterative process using values of p_L and p_o determined by the methods described in (a) and

(b) above, together with appropriate values of the shear modulus G . Values of G have been determined from the linear portion of $p - V$ curves obtained in pressuremeter tests by the method outlined in a subsequent section of this paper.

Estimates of c_u using this procedure have been determined using values of G obtained from the reload cycles of the pressuremeter and are given in column 6 of Table 1. However, estimates of G for stiff fissured clays determined from pressuremeter or other small field or laboratory tests can differ considerably from those relevant to large scale loading conditions.

This difficulty can be eliminated when comparing results of plate loading and pressuremeter tests in an isotropic material by using the same value of G/c_u in eqs (4) and (6), and by equating the term $\{^{4/3} \left[\log_e \left(\frac{G}{c_u} \right) + 1 \right] + 1 \}$ in eq (4) to values of N_c obtained empirically. It should be pointed out that this is only strictly true, even for isotropic soils, when both tests deform a volume of ground which is sufficiently large to be truly representative of the large scale properties.

In the present investigations, a reliable value of $N_c = 9.25$ was determined and thus we have

$$^{4/3} \left[\log_e (G/c_u) + 1 \right] + 1 = 9.25$$

giving

$$N_p = \log_e (G/c_u) + 1 = 6.18 \quad (10)$$

Substituting eq (10) in eq (6) gives, for the pressuremeter tests in London Clay, the expression:

$$c_u = \frac{p_L - p_o}{6.18} \quad (11)$$

Values of c_u determined using eq (11) are given in column 7 of Table 1. It can be seen that the values determined using this equation are much more consistent than those given in column 6 of Table 1, which were obtained using eq (6). In view of this and the simplicity of eq (11) its use is considered preferable when reliable empirical values of N_c are available. The values of c_u determined using eq (11) are plotted against depth in Figure 5, where they are compared with values of c_u determined from the large in-situ plate tests. (pH - Nc)

The values of c_u determined from pressuremeter tests using eq (11) are greater than the values determined from large plate tests using eq (5). While agreement is reasonable for tests in the brown weathered clay and the top of the blue clay, the differences become large at greater depths. At a depth of 24 m the values determined from the pressuremeter are approximately double those obtained from the plate tests. The magnitude of the difference between the values determined from the two types of test are closely related to the spacing and orientation of the fissures with respect to the direction of loading and the size of the loading device. At shallow depths the fissures are closely spaced with widely varying orientations. The spacing of the fissures increase with depth and below a depth of about 20 m they tend to become more predominantly horizontal and vertical.

Since the horizontal stresses are greater than the vertical stresses a simple intuitive approach based on sliding of frictional blocks would lead to an expectation of higher strengths from the pressuremeter than the plate tests. To illustrate this effect the undrained shear strengths are plotted against the corresponding effective normal stresses in Figure 9. For the pressuremeter the effective horizontal stress $K_0(\gamma_g z - u)$ has been used while for the plate tests a value mid-way between the horizontal and vertical effective stress $\frac{1}{2}(1 + K_0)(\gamma_g z - u)$ has been used. The curves for the strengths determined from both the pressuremeter and plate tests using elasto-plastic theories are very close except at the lower levels. It is realised that the closeness of the agreement may be somewhat fortuitous, but it suggests a fruitful line of investigation.

The values of c_u determined from both eqs (6) and (11) are affected by the value of p_0 . In the present case underestimates of the value of p_0 could have given values of c_u about 25 per cent greater than the values determined using the correct p_0 .

(d) Direct determination of the stress-strain curve and maximum shear stress

Over the last few years, several workers (Palmer, A C 1972; Ladanyi, B 1972; Baguelin, F, Jezequel, J F, Le Mee, E and Le Mehaute, A, 1972) have developed methods of deriving the stress-strain curves for clays from the pressure volume curves measured in pressuremeter tests. Since these methods are very similar, the one outlined by Palmer, A C has been used in this paper. The only assumptions he made were that the loading produces axi-symmetric deformation in the plane at right angles to the axis of the borehole and that the material is incompressible. The latter assumption implies that the soil is fully saturated and that no drainage occurs during the test.

Palmer, A C shows that the shear stress (τ) in the soil immediately adjacent to the wall of the expanding borehole is given in terms of the radial extension of the borehole (y_1) and the corresponding pressure gradient dp/dy_1 by

$$\phi(y_1) = 2\tau = y_1 (1 + y_1)(2 + y_1) \frac{dp}{dy_1} \quad (12)$$

where

$$y_1 = \frac{\text{increase in radius of borehole due to increase in pressure } (p - p_0)}{\text{radius of borehole at reference state } (p = p_0)}$$

The reference state $p = p_0$ is the horizontal pressure in the ground prior to boring the hole for the insertion of the pressuremeter. At small strains, eq (12) approximates closely to

$$\tau = y_1 \frac{dp}{dy_1} \quad (13)$$

which leads to a simple geometric method of calculating the shear stress directly from the curve of p against y_1 (Wroth, C P and Hughes, J M O, 1973). At larger strains, it is more convenient to express the shear stress in terms of the increase in volume, ΔV , from the reference state and the current volume, V , of the measuring cell at the measured pressure. Thus

$$\tau = \frac{dp}{d(\log_e \frac{\Delta V}{V})} \quad (14)$$

giving the shear stress as the slope of the curve of p against $\log_e (\frac{\Delta V}{V})$.

The maximum slope gives the maximum shear stress or the peak shear strength of the soil. By differentiating the curve at a number of points, a shear stress-strain curve for the soil immediately adjacent to the wall of the probe can be obtained. Typical examples obtained from the present tests

in London Clay are given in Figure 9 and the peaked nature of the curves is clear. Values of the maximum shear strengths determined in this way are given in column 8 of Table 1 and are plotted against the depth of the tests in Figure 5.

It can be seen that these values are appreciably higher than those determined from the ultimate bearing pressures measured on the 865 mm diameter plates using eq (5). These large differences are due partly to the anisotropy and partly to the non representative nature of the clay in the small elements of clay immediately adjacent to the probe. They are also significantly higher than the values determined from the limit pressures using eqs (6) and (11). These differences can also be explained in terms of scale effects since a much larger volume of clay has been sheared by the time the probe is fully inflated. The peak shear strengths determined from the stress-strain curves correspond to the small-scale strengths which approach the strengths of the intact clay between the fissures. The same effect has been observed in comparisons between the strengths determined from laboratory tests on small samples, standard static cone penetration tests and large in-situ loading tests (Marsland A, 1971(a), 1971(b) and 1974). The relevant strengths for estimating the stability of foundations in fissured London Clay are the large scale operational strengths such as those obtained by back analyses of failures or from loading tests on large diameter plates.

The peak values of c_u determined from eq (14) are also sensitive to the values of V_o , (corresponding to p_o) used in calculating the values of $\log_e \Delta V/V$ where

$\Delta V/V = \Delta V/(V_o + \Delta V)$. For the example given in Figure 7 the percentage change in the estimated c_u was about 50 per cent of the corresponding percentage change in p_o . Since in this case the total overburden pressure was 260 kN/m^2 (p_o corresponding to $K_o = 1$) and $p_o \approx 510 \text{ kN/m}^2$ the potential errors for overestimating the peak values of c_u , which could result from underestimates of p_o , are considerable.

In overconsolidated clays, not only the peak values but the complete stress-strain curve could depend upon the scale and nature of the clay fabric relative to the size and orientation of the pressuremeter probe. It is therefore considered inappropriate to use values of c_u based on the peak and post-peak values determined from these curves.

In spite of the limitations, correlations based on overall failure as given by eq (11) are to be preferred, since these values are less sensitive to variations in interpretation.

(e) Direct comparison of failure states of foundation and pressuremeter tests

An alternative approach, which bypasses the separate calculation of c_u , is to relate the bearing capacity of a foundation directly to the limit pressure measured by the pressuremeter. Menard L F (1963) gives an empirical relationship:

$$q_u = q_o + K (p_L - p_o) \quad (15)$$

where q_u = 'ultimate' bearing capacity of the foundation measured at a settlement of 10 per cent of the width or diameter

q_o = vertical overburden pressure = $\gamma_s z$

p_L = average pressuremeter limit pressure at that depth

p_o = lateral in-situ earth pressure at the corresponding depth

K = the bearing factor dependent on the soil type and the shape of the foundation.

For the base of a bored pile in a cohesive soil, Menard suggests a value of $K = 1.8$. By comparing eqs (4) and (6) with the appropriate values of $N_c = 9.25$ and $N_p = (p_L - p_o)/c_u = 6.18$, one obtains:

$$\begin{aligned} q_u &= \frac{9.25}{6.18} (p_L - p_o) + q_o \\ &= 1.5 (p_L - p_o) + q_o \end{aligned} \quad (16)$$

This value of $K = 1.5$ is slightly lower than that suggested by Menard even though the empirical value of $N_c = 9.25$ obtained from the large plate tests was based on maximum bearing pressures which occurred at settlements equivalent to about 15 per cent of the plate diameter. The values of K determined from the ratios of $(q_u - q_o)/(p_L - p_o)$ determined from the plate and pressuremeter tests are even lower. They decrease from 1.25 at a depth of 6.1 m to 0.75 at a depth of 24.4 m. At the shallow depths the difference is largely due to the fact that the pressuremeter measures the shear strength on a vertical face which, for a heavily overconsolidated clay is larger than the strength in other directions along which some of the shear strength is mobilised in the plate test. At the deeper levels the effects of the small size of the diameter of the probe in relation to the spacing of the fissures, is the predominant effect.

SHEAR MODULUS

(1) Plate loading tests

The settlement of a rigid punch loaded at the surface of an elastic half-space has been given as (Timoshenko and Goodier, 1951):

$$\rho = \frac{P (1 - \nu^2)}{BE} = \frac{P (1 - \nu)}{2 BG} \quad (17)$$

where ρ = settlement of punch

P = total load applied to punch

B = diameter of punch

E = elastic (Young's) modulus of half-space

G = shear modulus of half-space, $G = E/2(1 + \nu)$

ν = Poisson's ratio

For the case of a plate test conducted at the bottom of an open borehole, a depth factor must be used (Burland, J B 1969; Marsland, A 1971(c)) to take into account the stiffening effect of the soil above the test level. Writing the average pressure on the plate as $q = P/(\pi B^2/4)$ we obtain

$$\rho = \frac{q}{G} \cdot B \frac{\pi}{8} (1 - \nu) \cdot f(z) \quad (18)$$

where

$$f(z) = \text{the depth factor} = \frac{(\text{settlement of loaded plate at depth } z)}{(\text{settlement of loaded plate at surface})}$$

For undrained loading tests ($\nu = 0.5$) at depths greater than ten times the plate diameter the depth factor is approximately equal to 0.85.

The shear modulus may thus be calculated from a plate test using the following equation:

$$G = \frac{q}{\rho} \cdot \frac{\pi}{8} \cdot B(1 - \nu) \cdot f(z) \quad (19)$$

Equation (19) is only strictly applicable to a linear elastic half-space.

In practice a shear modulus for non-linear materials, such as soil, may

be obtained by choosing a sensible range of the bearing pressure q over which to measure the settlement of the plate. It is common practice to take secant moduli over the stress range from zero to either a third or one half the total ultimate bearing capacities, q_u (line OA in Figure 1). Values of G over the range of q from $0 \rightarrow q_u/2$ measured in large plate tests are plotted with depth in Figure 11. The values determined from both the initial loading and the re-loading following the unloading from a bearing pressure equal to $\frac{1}{2}q_u$ (line BC in Figure 1) show an approximately linear increase with depth. The ratios of the values obtained from re-load cycles to those from the initial loading cycle also increase with depth, indicating that fissures and discontinuities in the clay open much more rapidly at the lower levels where the relief of stress during drilling is greatest.

As a consequence of the curved nature of the load-settlement curves the values of the moduli depend upon the range of stress over which the settlements are measured. Values of the tangent moduli taken at bearing pressures corresponding to the existing overburden pressures (line DE in Figure 1) are of particular interest when making comparisons with the values determined from pressuremeters. These values are also plotted in Figure 11. At shallow depths these tangent moduli are higher than the secant moduli taken over the bearing pressure range of 0 to $\frac{1}{2}q_u$. However, at greater depths where large foundations are often founded, the values are in close agreement. It is also of interest to note that the tangent moduli taken at bearing pressures.

equal to the existing overburden are almost constant with depth. In view of the increase in strength with depth this is contrary to expectations and is a further indication of the more rapid expansion of the clay at the lower levels, during the period that the clay remains unloaded.

Secant moduli determined from the loading curves over the range of bearing pressure from the existing overburden (q_o) to a pressure corresponding to the overburden plus one third of the net bearing pressures $q_o + \frac{1}{3}(q_u - q_o)$ are also of interest in connection with pressuremeter tests and some foundation problems. Values determined in this way are also plotted against depth in Figure 11. As would be expected these values are generally lower than those for the secant moduli determined from the bearing pressure/settlement curves over the range 0 to $\frac{1}{2}q_u$. There is also a tendency for the secant moduli taken from $q_o + \frac{1}{3}(q_u - q_o)$ on the initial loading curve to decrease with depth. This is possibly the clearest indication that the moduli determined from the plate test at the lower levels are lower than they should be as a result of the expansion which takes place during the period of unloading.

(2) Pressuremeter tests

For the case of the expansion of a cylindrical cavity in an infinite linear elastic medium, the expression for the shear modulus is given by Gibson R E and Anderson W F 1961 as:

$$G = v_m \cdot \frac{\Delta p}{\Delta V} \quad (20)$$

where Δp and ΔV are increments of pressure and corresponding volume change in the elastic region

V_m is the average volume of the cavity over the pressure increment Δp .

For non-linear materials, the mathematical definition of the shear modulus is the tangent of the shear stress-strain curve at the origin:

$$G = \left(\frac{d\tau}{d\gamma} \right)_{\gamma = 0} \quad (21)$$

where τ = shear stress

γ = shear strain

For the case of the expansion of a cylindrical cavity at small strains, the change in shear stress in the soil adjacent to the probe is $\Delta\tau \approx \Delta p$ and the corresponding shear strain is $\gamma \approx 2y_1$. Substituting in eq (21) the tangent modulus at the reference state where $p = p_0$ is given by

$$G = \frac{1}{2} \left(\frac{dp}{dy_1} \right)_{y_1 = 0} \quad (22)$$

Since for small strains, $y_1 \approx \frac{1}{2} \frac{\Delta V}{V_0}$, eq (20) and (22) are equivalent for the tangent modulus.

For engineering purposes a secant modulus measured in the plate over the appropriate pressure range is more relevant. Equation (22) may be re-written in terms of increments of pressure p and strain y_1 to give a secant modulus of

$$G = \frac{1}{2} \left(\frac{\Delta p}{\Delta y_1} \right) \quad (23)$$

where Δp and Δy_1 are measured over a suitable range. In order to select a suitable range, it is necessary to consider the shear stress levels in both the plate and pressuremeter tests.

The stresses under a loaded plate can be determined in the elastic range using the Boussinesq solution. Timoshenko S P and Goodier J N (1951) have shown that the maximum shear stress occurs on the centre line below the plate and is approximately equal to 0.3 times the average pressure (q) applied to the plate. Since for an elastic-plastic material plastic deformation commences when the maximum shear stress exceeds the shear strength c_u , a plastic zone starts to develop when $q \approx 3.33 c_u$. It has already been shown in eq (5) that for plate tests in London Clay the applied pressure at failure ($q_u - \gamma_s z$) is approximately equal to $9.6 c_u$. Thus a plastic zone will start to develop under the plate when

$$q - \gamma_s z \approx \frac{1}{3} (q_u - \gamma_s z) \quad (24)$$

In the case of the pressuremeter the maximum shear stress occurs in the clay immediately adjacent to the wall of the probe and is approximately equal to the increase in pressure from the reference state, ie $\tau \approx p - p_o$. It follows that a plastic zone starts to develop near the wall of the probe when $p = p_o + c_u$.

Thus secant moduli obtained from plate tests over the range of bearing pressure from $q_o = \gamma_s z$ to $q = q_o + \frac{1}{3}(q_u - q_o)$ correspond to secant moduli determined from pressuremeter tests over the range of pressure from p_o to $p_o + c_u$.

From the typical curves obtained from pressuremeter tests shown in Figure 12 it can be seen that the curves of p against y_1 , are reasonably linear over this range and there is thus little difference between the tangent and secant moduli. Values of the tangent shear moduli taken from the pressuremeter

curves at $p = p_0$ are plotted with depth in Figure 11. These values are substantially lower than those derived from the plate tests. The theoretical studies of Hartman P J and Schmertmann J H 1975 show that much of this may be attributed to remoulding of the clay at the wall of the borehole during drilling. The re-loading cycles of the pressuremeter tests appear to give more reasonable values of the shear modulus, although many of these are still appreciably lower than those determined from the plate loading tests.

CONCLUSIONS AND RECOMMENDATIONS

Comparisons have been made of the estimates of undrained shear strengths and moduli of stiff fissured London Clay determined from pressuremeter tests using a 60 mm diameter probe and loading tests on a 865 mm diameter plate down to a depth of 24 m. The plate loading tests deformed a volume of soil which contained sufficient fissures and other planes of weakness to be fully representative of the full-scale. It is considered that the estimates of the undrained shear strengths and shear moduli determined from these tests are the operational values which are relevant for use in foundation design. They have been used as a basis in evaluating the pressuremeter as a design tool. These comparisons have shown that:

- (i) undrained shear strengths determined from the limit pressures, extrapolated from the $p - \log_e \frac{\Delta V}{V}$ curves obtained from pressuremeter tests, using elasto-plastic theory give the best agreement with the operational values determined from the large plate tests. Even so the strengths determined from the pressuremeter tests are higher than the operational values. The ratio of the pressuremeter to the operational values determined from the plate tests increases with depth from about 1.1 at a depth of 6 m to 1.5 at 18 m and reaches 2.0 at a depth of 24.

It is considered that the large differences at the lower levels are mainly due to the small diameter of the pressuremeter probe in relation to the spacing of the fissures and other discontinuities. Some of the differences could also be due to the fact that in the highly over-consolidated London Clay the horizontal effective stresses are appreciably greater than the vertical effective stresses. As a consequence the undrained strengths mobilised along failure surfaces which are entirely in the horizontal plane, as in the pressuremeter tests, are larger than those mobilised along failure surfaces in other planes. A crude allowance can be made for this effect by assuming that the undrained shear strength is proportional to the in-situ normal effective stress along the potential shear planes. The effect of scale, on the other hand, can only be overcome by the use of much larger diameter probes.

- (ii) The peak undrained shear strengths determined directly from the slope of the $p - \log_e \frac{\Delta V}{V}$ curves, using the Palmer type analysis, are appreciably higher than those determined from the limit pressures. The ratios of these peak values to the operational strengths determined from the plate tests increase from about 1.4 at 6 m to 2.0 at 20 m, and below this depth are greater than 3.0. These appreciably larger differences reflect the very small volume of soil being sheared at the low strains at which the peak shear strength is developed in the soil adjacent to the probe. As a consequence the peak values determined by this method tend towards the strength of the intact material between the fissures which is not representative of the overall bulk operational strength.

- (iii) Values of p_o cannot be measured directly by pressuremeters installed in pre-drilled boreholes. Even so estimates of p_o in reasonable agreement with values determined indirectly from laboratory tests and by direct measurements using self-boring pressuremeters (Windle D and Wroth C P 1977) have been obtained by a new iterative procedure. The procedures normally used for estimating values of p_o from the $p - V$ curves obtained from pressuremeter tests seriously underestimate the true values of p_o . While the values of p_L is not affected significantly by the values of p_o and the corresponding V_o , the values of c_u determined by both methods depend on good estimates of p_o or V_o .
- (iv) In order to obtain good estimates of the limit pressure, p_L , it is necessary to expand the pressuremeter probe to a volume approximately twice the original volume.
- (v) The limit pressures measured by a pressuremeter can be related directly to the ultimate bearing capacity of a foundation by choosing a suitable factor of K . For a uniform isotropic stiff clay the theoretical value of K is approximately 1.5. However, for stiff fissured clays the value of K depends upon the composition and fabric of the clay. In the case of London Clay, the empirical value of K is about 1.25 near the surface and decreases with depth to 0.75 at 24 m.
- (vi) The shear moduli determined from the pressuremeter tests were substantially lower than those obtained from the plate loading tests. This was especially true for the values determined from the initial loading. This, together with the large difference between the value

determined from the initial loading and the re-loading cycles show that considerable softening had occurred at the side of the borehole during drilling and insertion of the probe. The moduli determined from the re-load cycles are much closer to the operational values determined from the large plate tests. The importance of this effect has been confirmed by the higher values of G determined from tests using self-drilling pressuremeters (Amar S, Baguelin F, Jezequal, J F and Le Mehaute, A 1975; and Windle, D and Wroth, C P, 1977).

As a consequence of the findings during the present investigations it is recommended that:

- (i) Shear strengths derived from pressuremeter tests in stiff clays should be determined from the limit pressures using elastoplastic theory. In order to obtain the operational strength these values must be reduced by a factor which depends on the nature and scale of the clay fabric relative to the size and orientation of the pressuremeter probe.
- (ii) Detailed comparisons should be made between pressuremeter and large in-situ plate tests in a wide range of clays. It is essential that these comparisons are made over the full depth of soil likely to be loaded by foundations. Empirical relationships obtained from tests carried out in weathered clay at shallow depths seldom apply to the clay at greater depths. Comparisons between pressuremeter tests and other small in-situ and laboratory tests in overconsolidated clays can be extremely misleading since they are all subjected to appreciable scale effects.

- (iii) Detailed investigation should be made into the effects of increasing the size of probes in a wide range of clay fabrics. The use of larger probes should reduce the differences between the values determined from pressuremeter and large in-situ plate tests.
- (iv) Consideration of scale effects due to discontinuities and higher effective horizontal stresses in over consolidated clays suggest that moduli determined from 'ideal' pressuremeter tests should be higher than the operational moduli determined from plate tests. When pressuremeter tests are carried out in pre-drilled boreholes in stiff clay these effects are more than balanced by the softening which occurs during drilling and insertion of the probe. In this case it is reasonable to take values of moduli determined from the reload cycles as a basis for design. As self-drilling instruments causing less disturbance are developed these balancing effects will be reduced and moduli greater than the operational value may be anticipated. The limited investigations so far carried out using a self-drilling instrument in the stiff clay at the present site show that the moduli determined from the initial loadings are comparable with those determined from the large plate tests.
- (v) Since the rate of loading will effect both the pressuremeter and the large plate tests it is important to include tests to study this effect in future investigations.

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TABLE 1 - SUMMARY OF RESULTS FROM PRESSUREMETER TEST

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Depth m	P_L kN/m^2	P_o kN/m^2	Reload Moduli		Values of c_u kN/m^2			N $1 + \log_e \frac{q}{c_u}$
			G MN/m^2	E MN/m^2	$\frac{P_L - P_o}{1 + \log_e \frac{q}{c_u}}$	$\frac{P_L - P_o}{6.18}$	Peak Palmer	
Borehole 1	3.99	206	-	-	-	91	143	4.55
	7.01	363	19.5	58.5	108	108	134	6.19
	9.75	439	11.6	34.8	121	109	154	5.57
	13.11	511	8.0	24.0	156	125	172	4.94
	16.15	580	20.1	60.3	152	145	174	5.89
	19.20	639	11.6	34.8	175	147	200	5.19
	21.95	696	17.5	52.5	217	189	401	5.39
	25.30	758	(12.4)	(37.2)	(343)	255	464	(4.59)
Borehole 2	3.96	206	18.0	54.0	74	78	120	6.49
	7.01	363	24.4	73.2	108	112	149	6.42
	10.06	442	10.3	30.9	108	97	143	5.56
	13.11	511	12.0	36.0	150	131	167	5.38
	16.15	580	12.7	38.1	157	137	180	5.39
	19.20	639	21.3	63.9	175	164	184	5.80
	22.25	699	28.1	84.3	237	222	295	5.77
	25.30	758	25.7	77.1	227	210	247	5.73

TABLE 2

EMPIRICAL RELATIONSHIP BETWEEN ULTIMATE BEARING PRESSURE AND LIMIT PRESSURE

Depth m	865 mm dia. plate tests				Pressuremeter tests			$K = \frac{q_u - q_o}{p_L - p_o}$
	q_u kN/m^2	q_o kN/m^2	$q_u - q_o$ kN/m^2		p_L kN/m^2	p_o kN/m^2	$p_L - p_o$ kN/m^2	
6.1	920	120	800		970	330	640	1.25
12.2	1095	240	855		1215	490	725	1.18
18.3	1310	360	950		1564	580	885	1.08
24	1510	480	1030		2100	740	1360	0.75

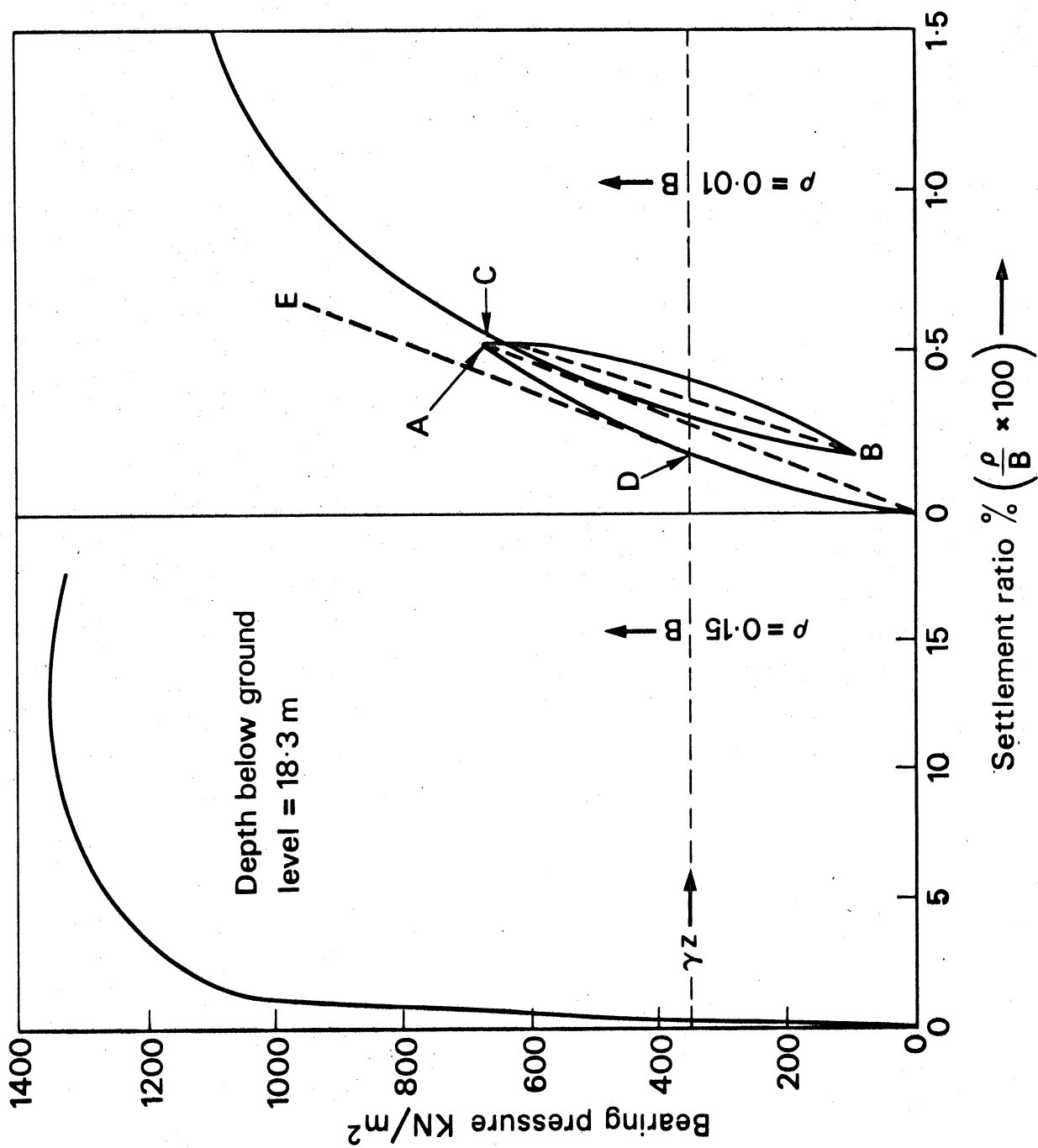


Figure 1 Typical load settlement curves from load tests on a 865 mm diameter plate

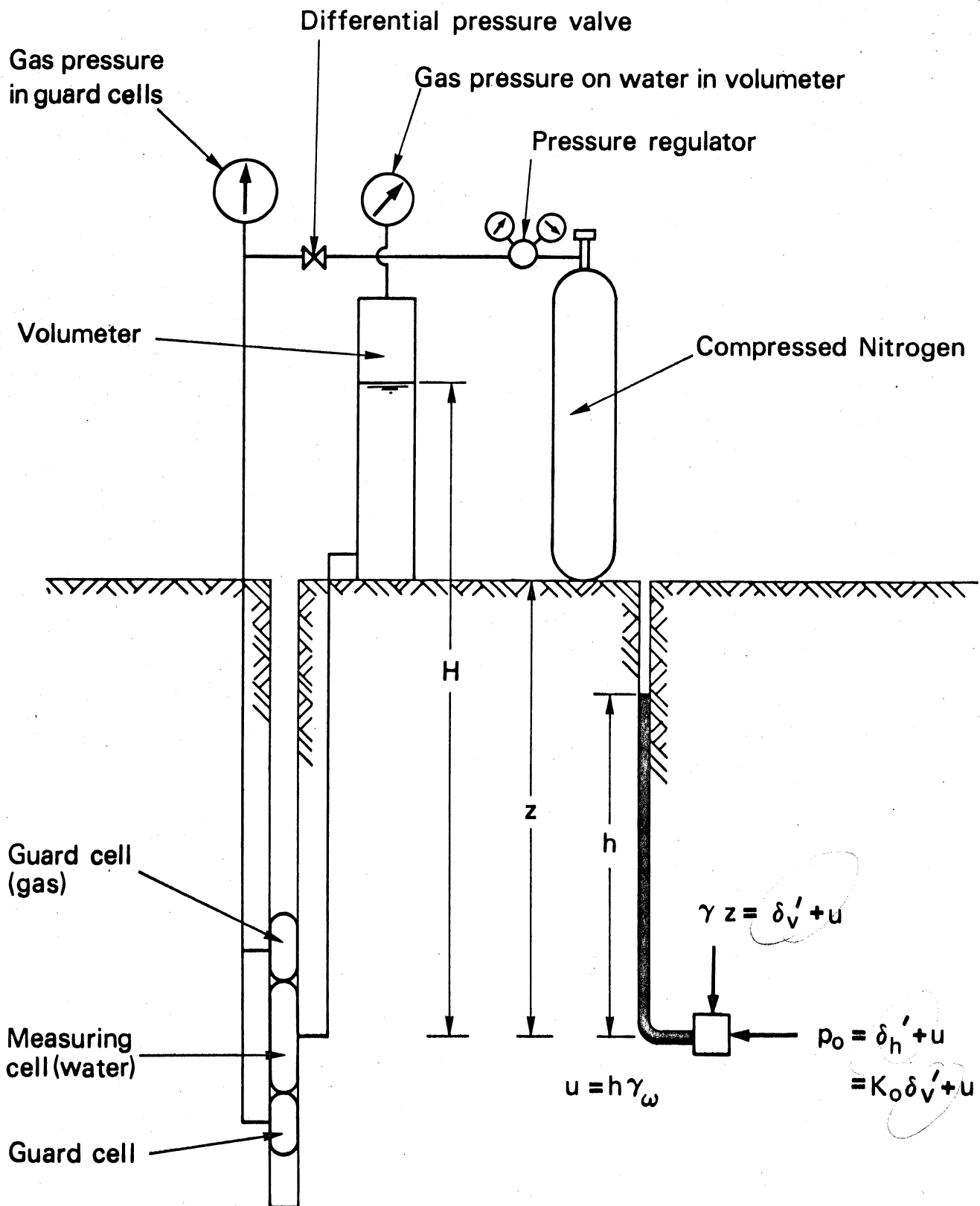
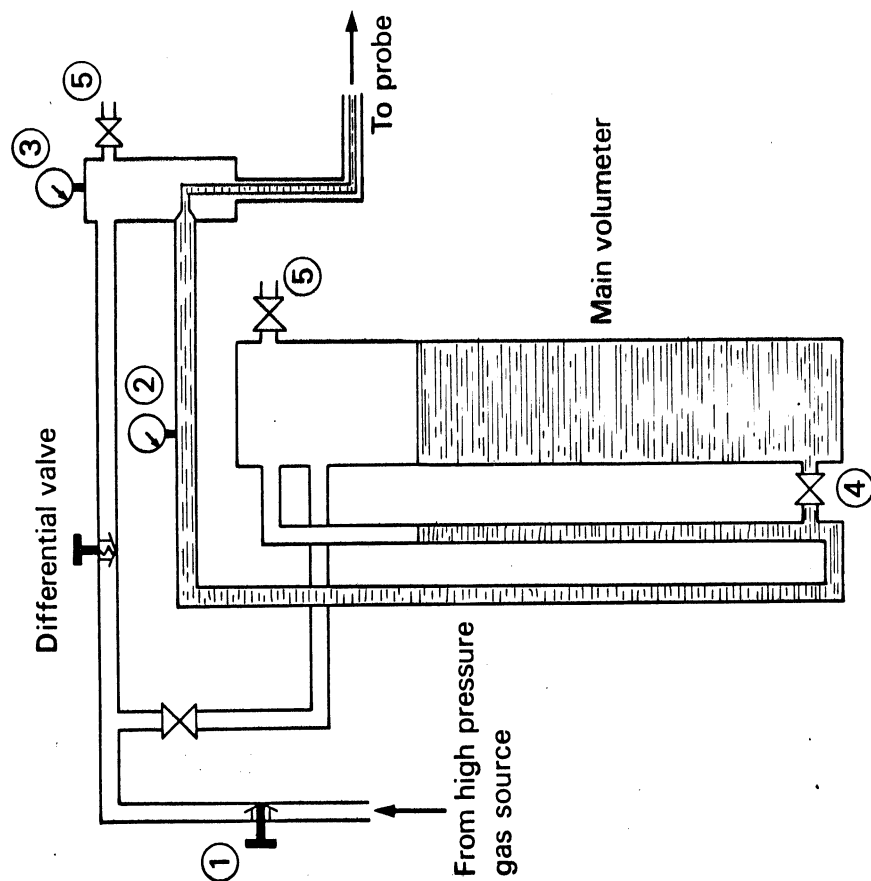
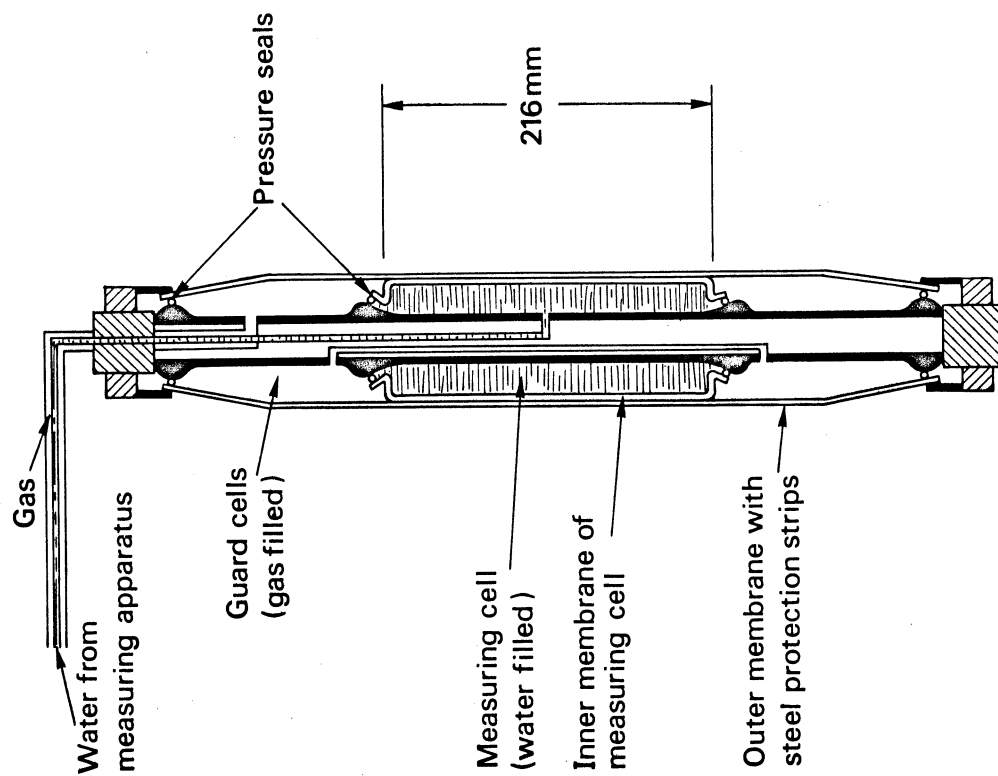


Figure 2 Diagram of pressure meter system



- ① Main pressure regulator
- ② Main cell pressure gauge
- ③ Guard cell pressure gauge
- ④ Tap for switching to fine volumeter
- ⑤ Bleed valves

(a) Volumeter : pressure circuits



(b) Partially expanded probe (type G-B)

Figure 3 Details of volumeter and pressuremeter probe

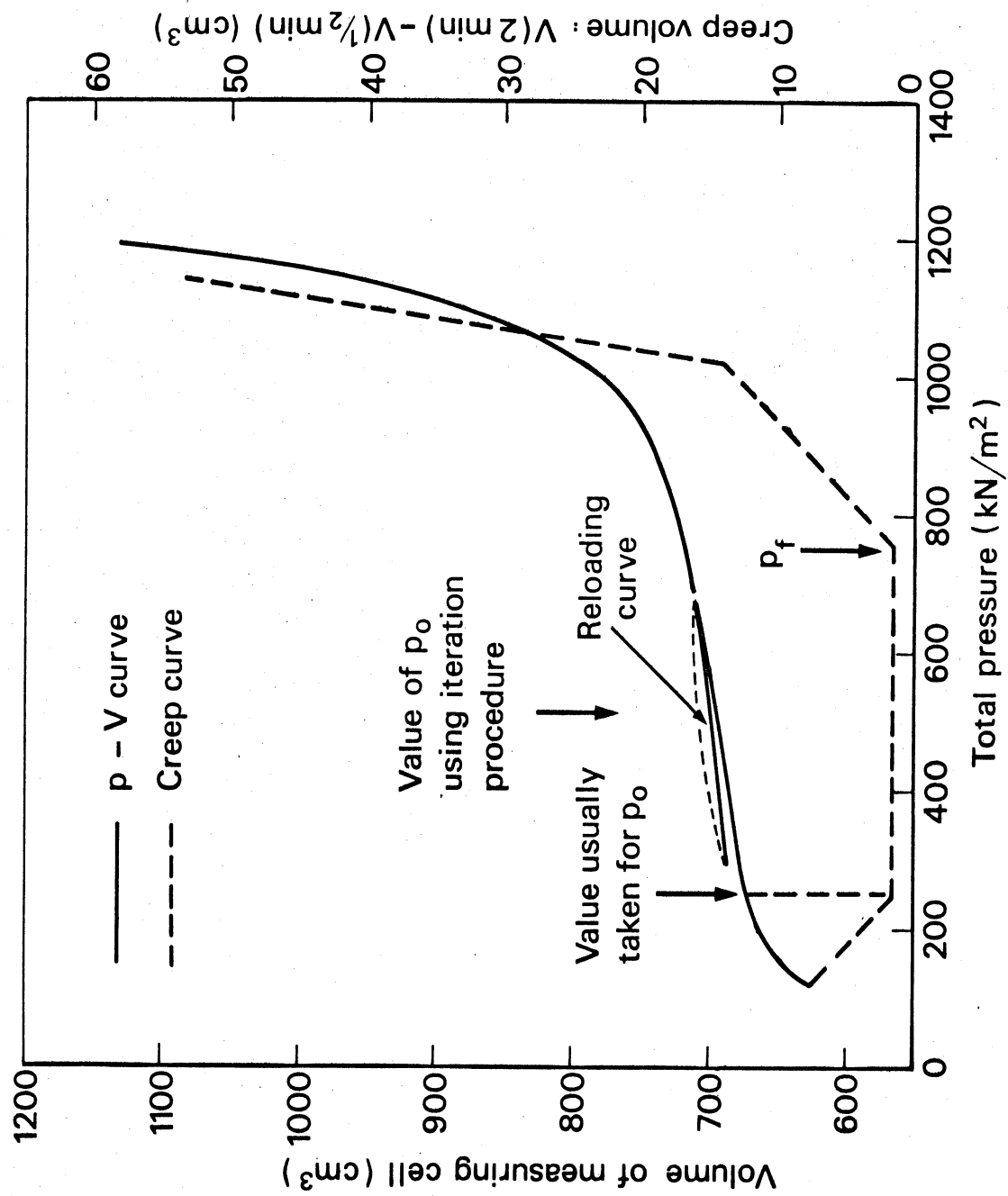


Figure 4 Typical P-V curve from pressuremeter test (depth 13.1 m, Borehole 2)

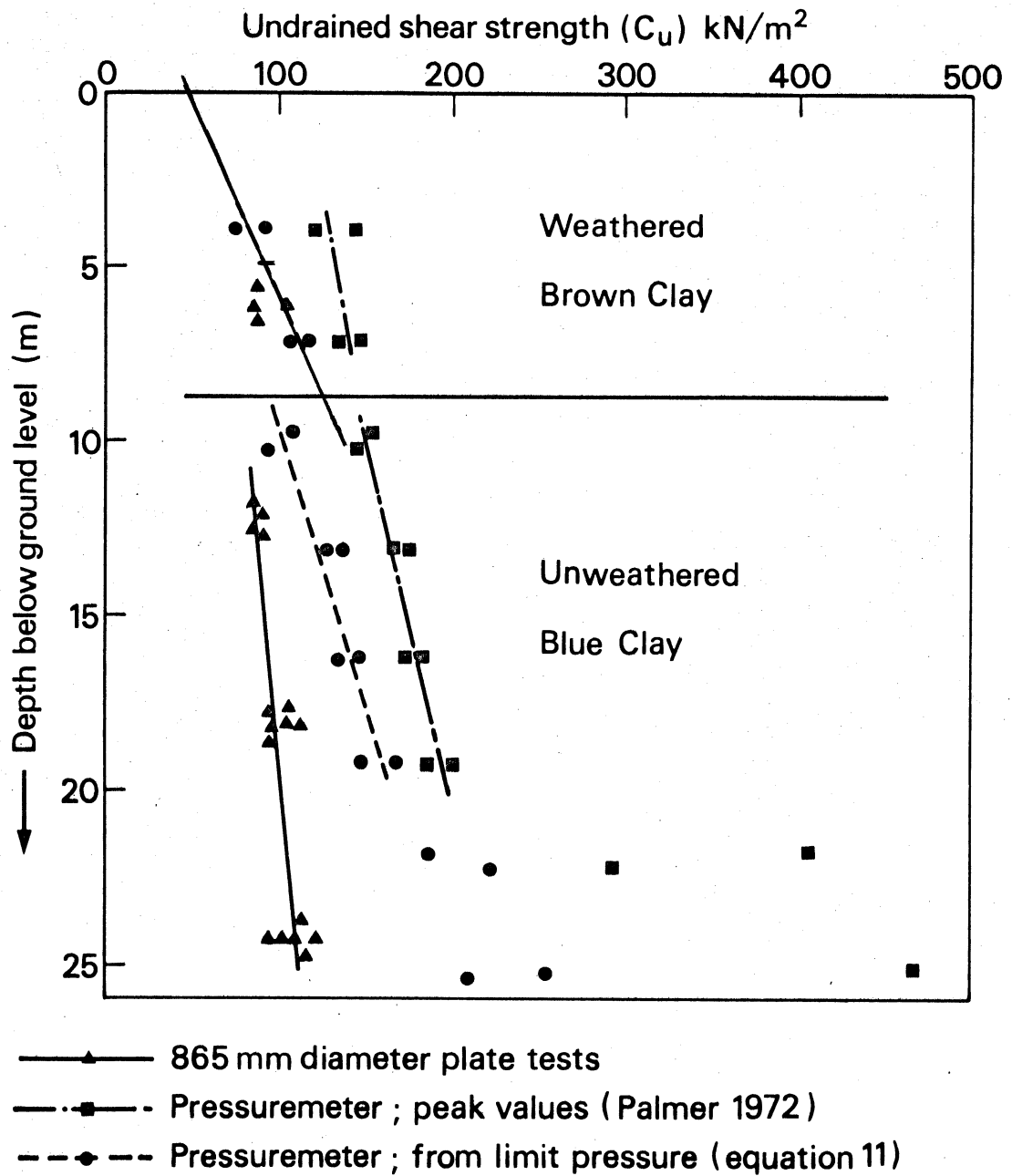


Figure 5 Comparison of undrained shear strengths determined from pressuremeter and large diameter plate tests

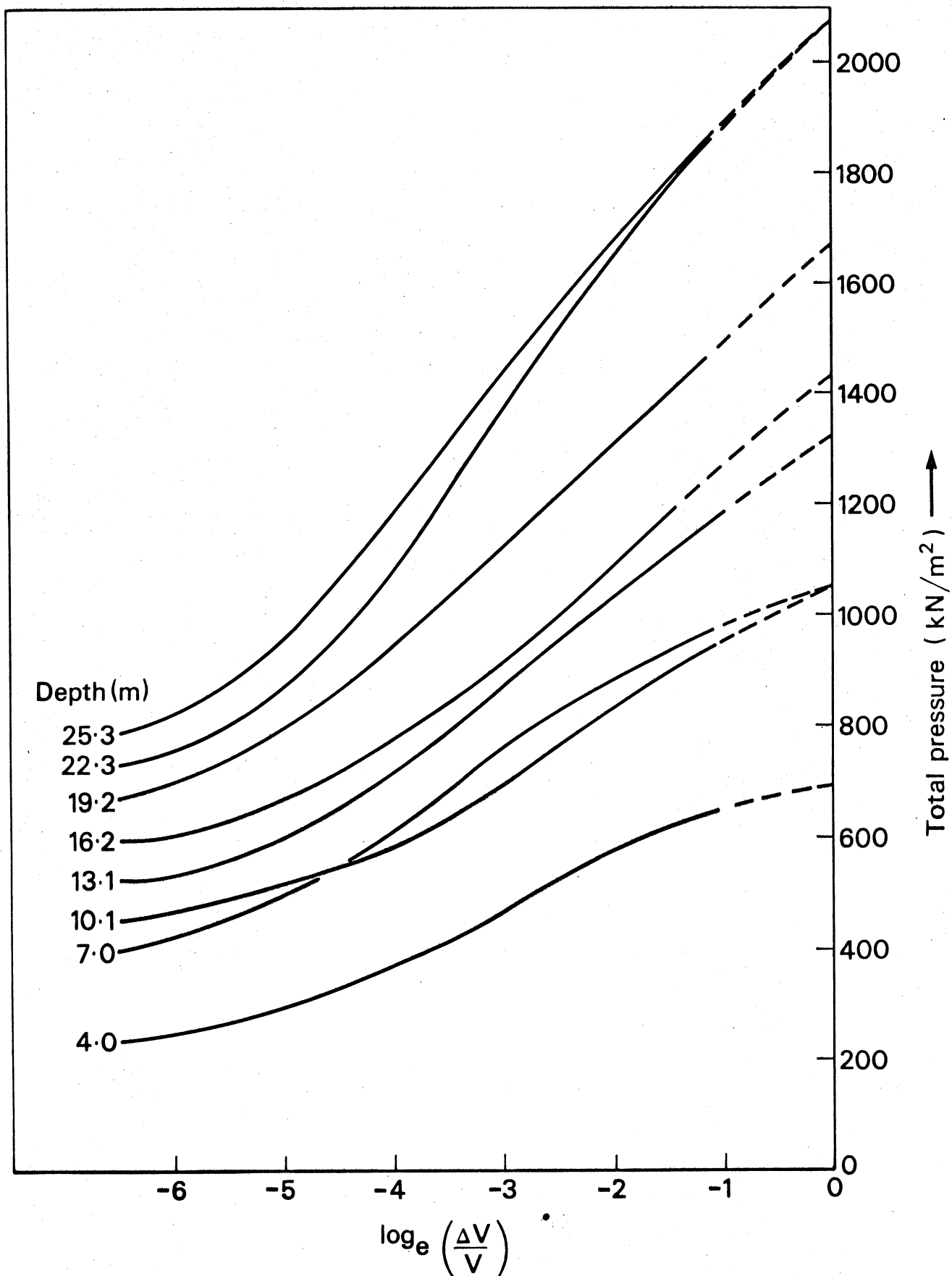


Figure 6 $p - \log_e \left(\frac{\Delta V}{V} \right)$ curves from pressuremeter tests (Borehole 2)

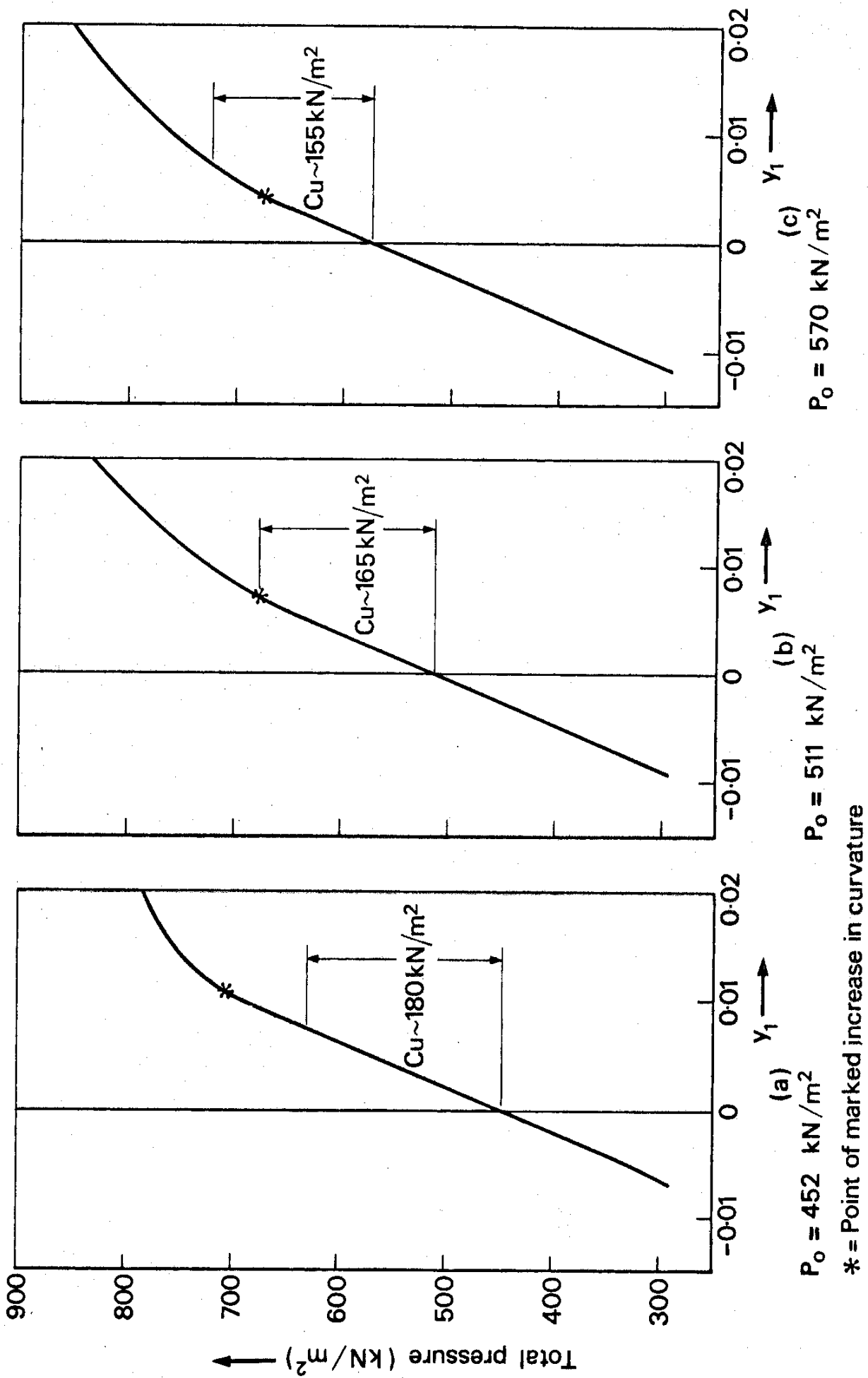
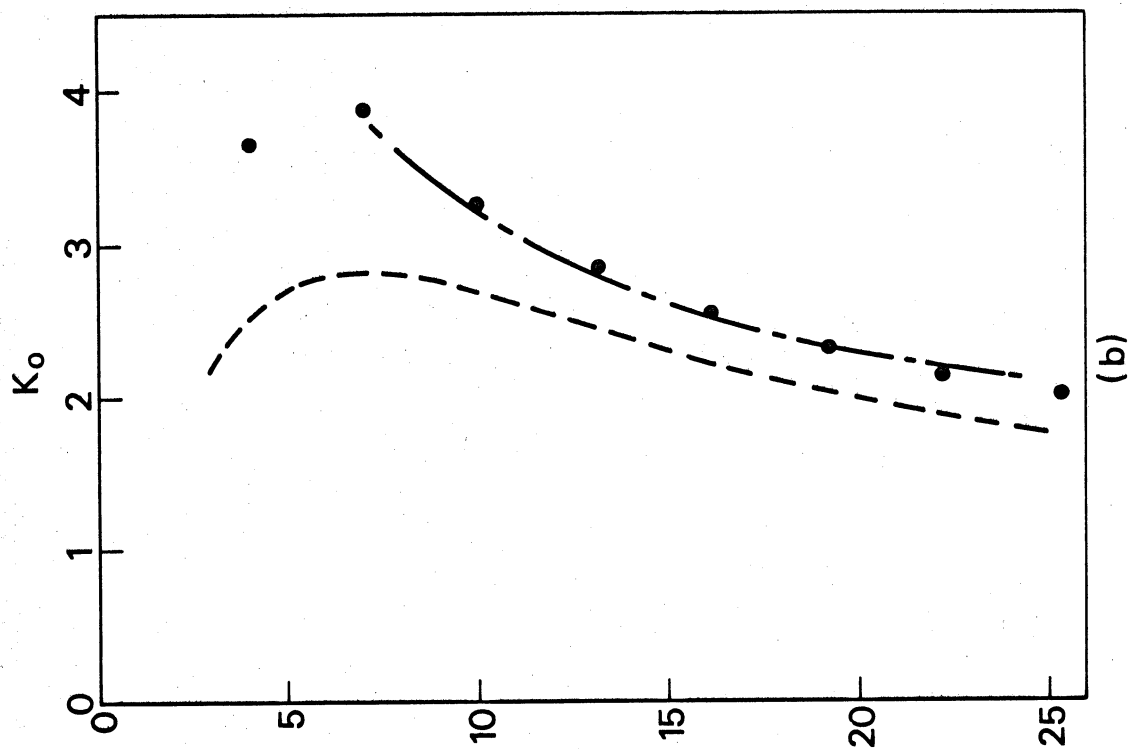
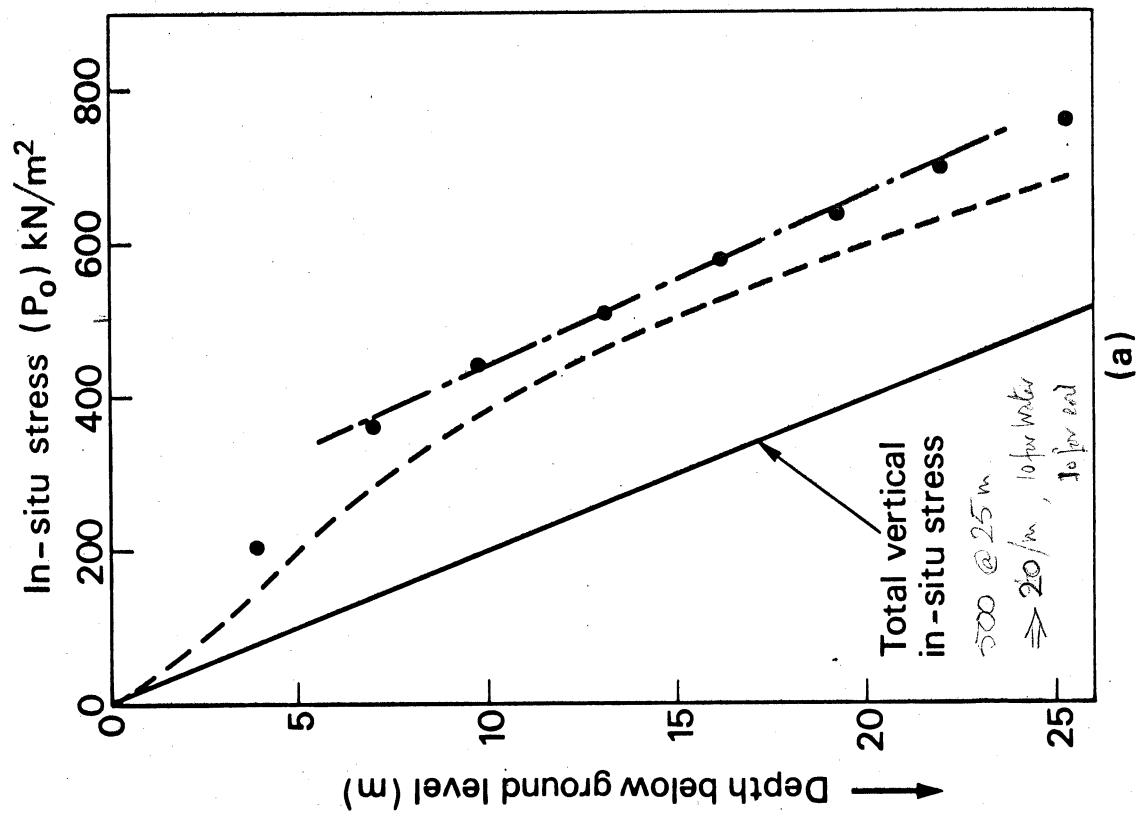


Figure 7 Procedure for estimating in-situ horizontal stress. (Depth 13.1 m, borehole 2)



- Values from pressuremeter tests
- p_o and K_o from Skempton (1961)
- p_o and K_o from Bishop et al (1965)

Figure 8 Variation of total in-situ horizontal stress with depth

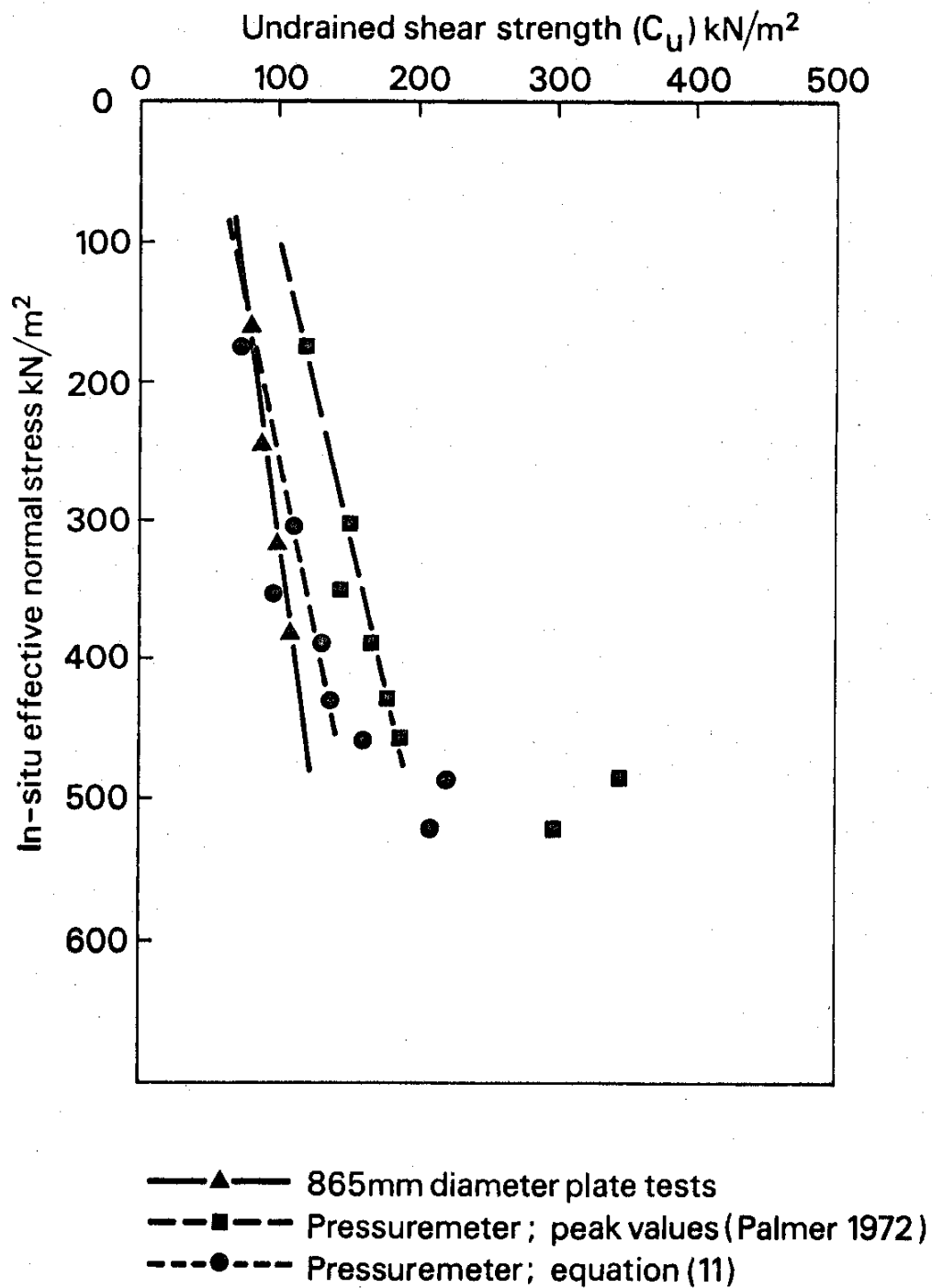


Figure 9 Comparison of undrained shear strengths plotted against in-situ effective normal stress

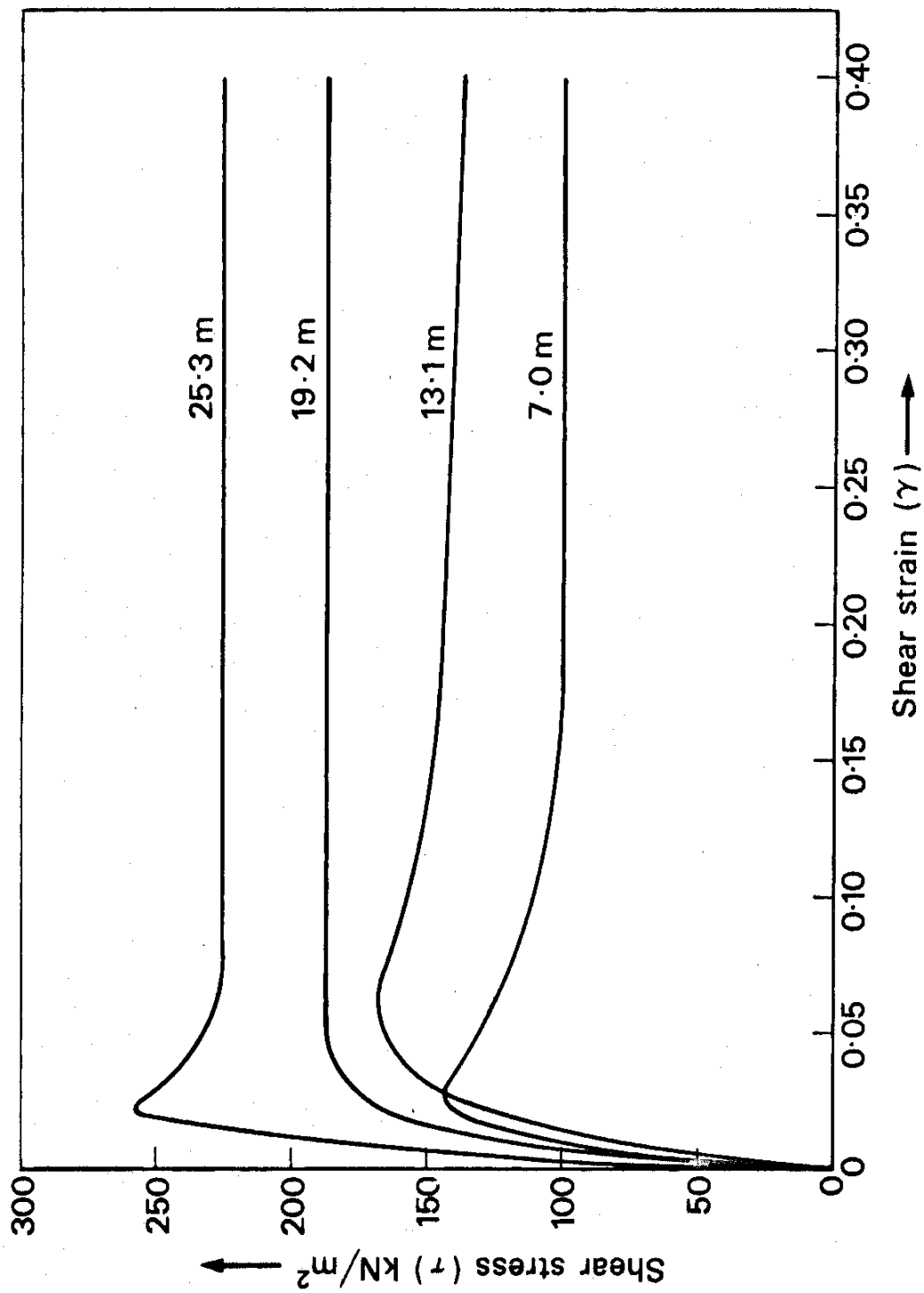
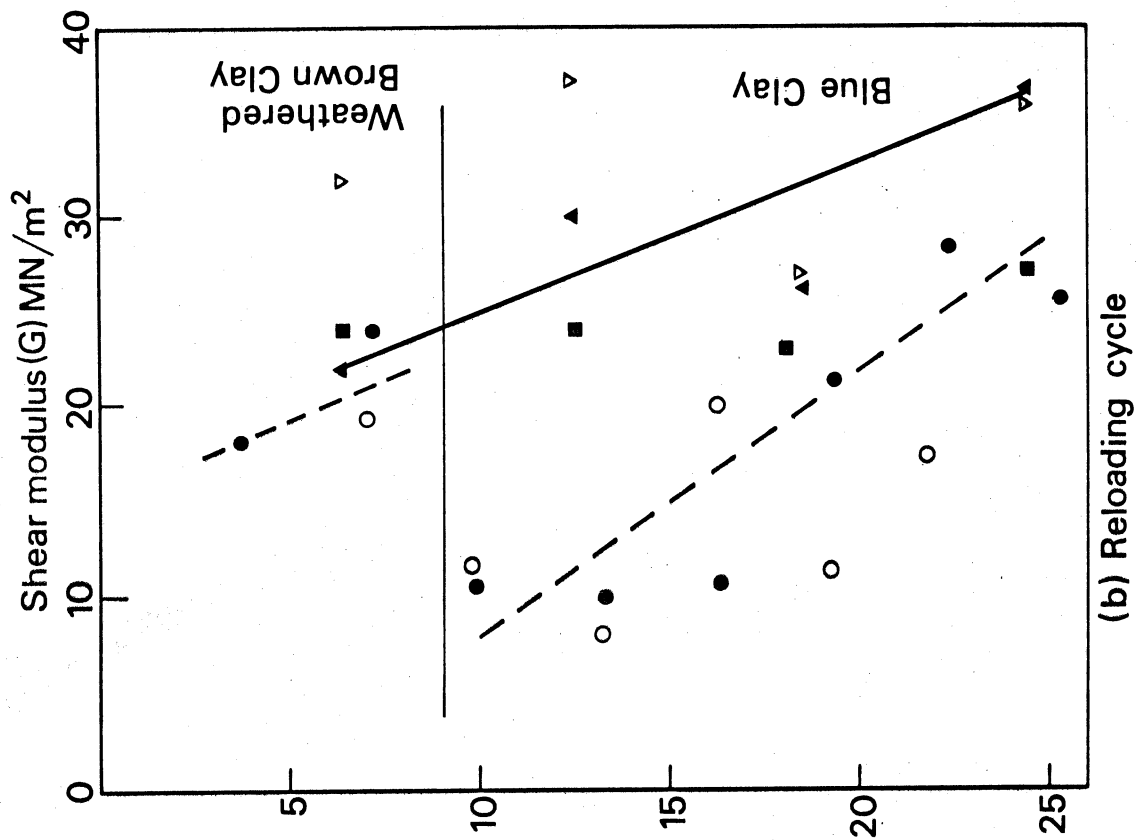
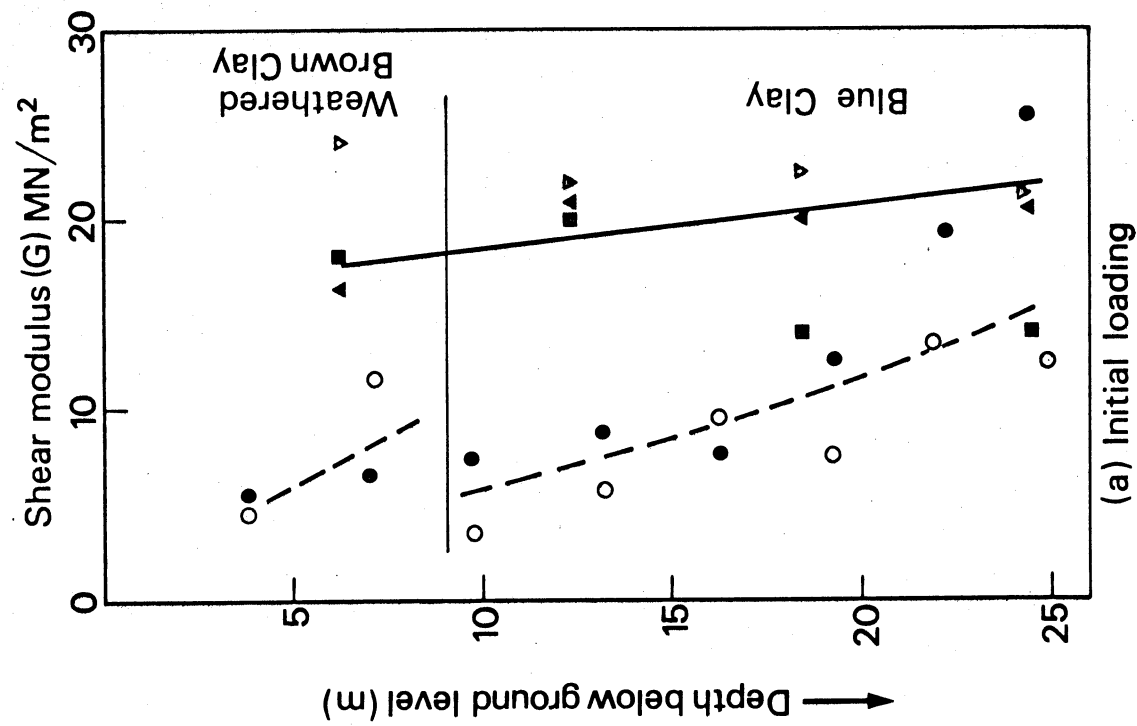
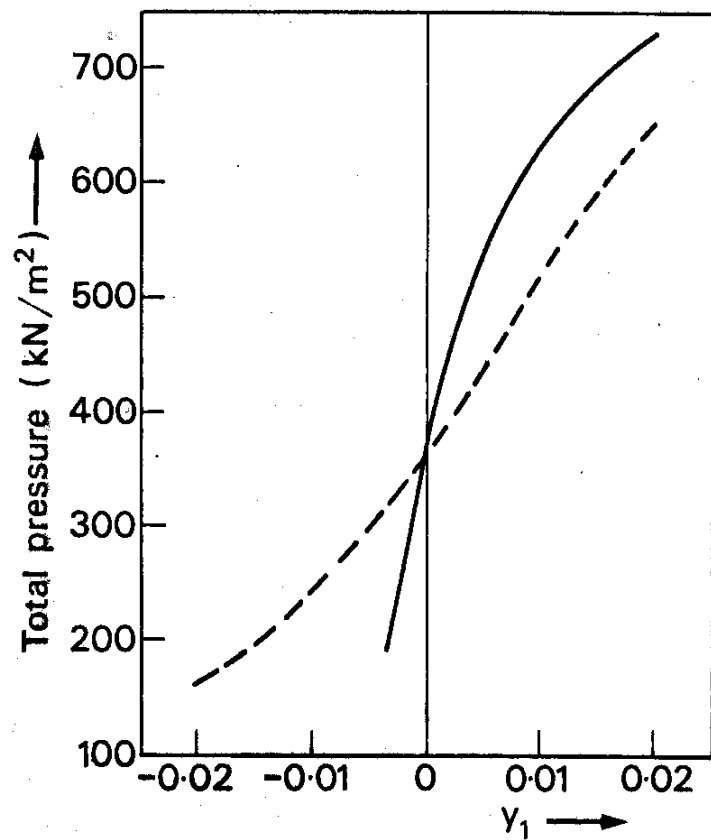


Figure 10 Stress - strain curves from pressuremeter tests (borehole 2)

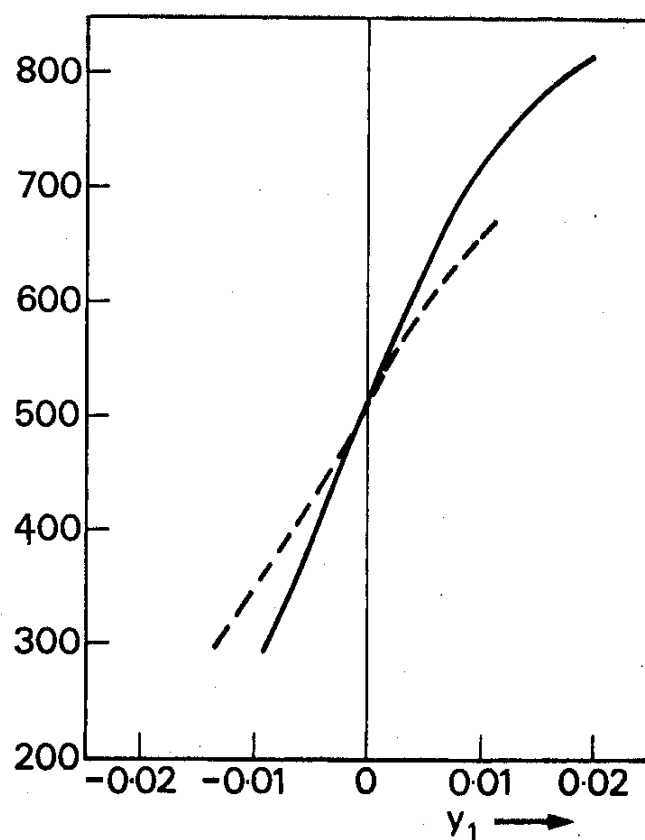


Pressure meter - Bore hole 1 ○ } tangent moduli
 Bore hole 2 ● } at p_0
 --- secant moduli at q_0
 --- secant moduli q_0 to $q_0 + \frac{1}{3}(q_u - q_0)$

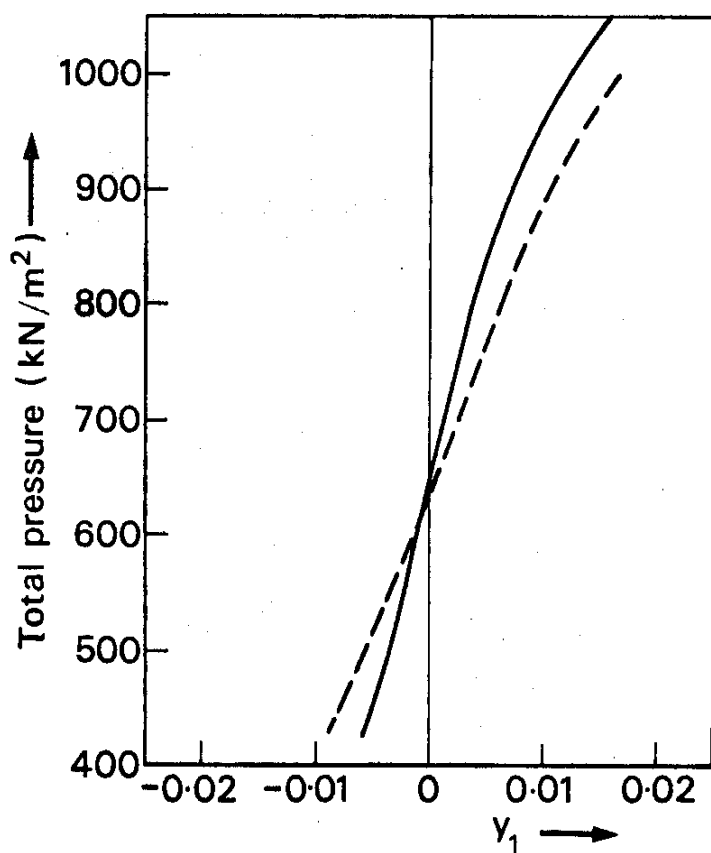
Figure 11 Comparison of moduli determined from pressuremeter and large in-situ loading tests



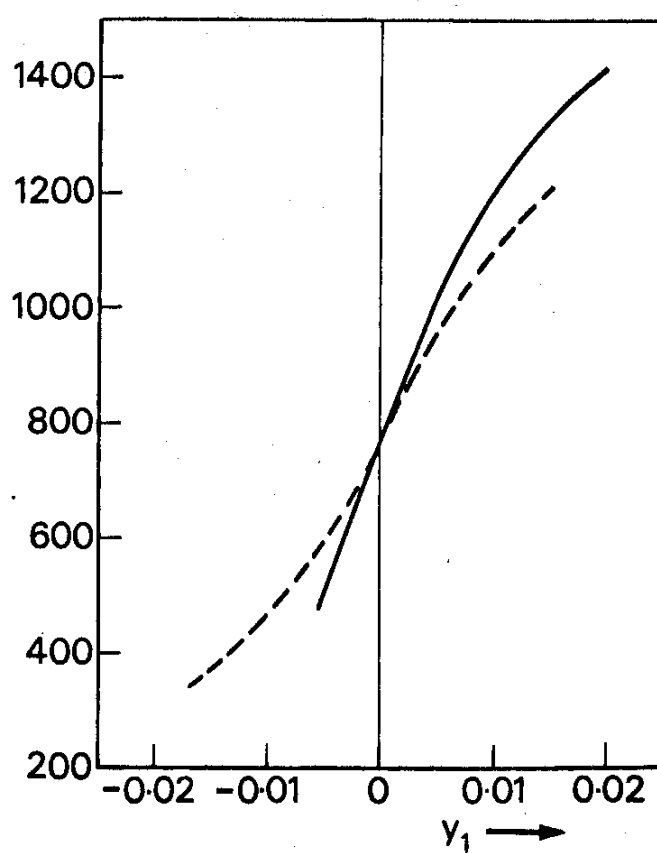
(a) Depth 7.0 m



(b) Depth 13.1 m



(c) Depth 19.2 m



(d) Depth 25.3 m

----- Initial loading

————— Reloading

Figure 12 Small strain $p - y_1$ curves from pressuremeter tests (borehole 2)