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The use of a self-boring pressuremeter to determine the undrained properties of clays

by D. WINDLE†, BA, PhD, and C. P. WROTH*, MA, PhD, MICE

Introduction

MODERN DEVELOPMENTS, such as the finite element method, have resulted in increasingly sophisticated techniques of analysis for problems in soil mechanics. These improved methods have high-lighted the problems inherent in conventional sampling and laboratory testing. Frequently, these testing procedures cannot supply suitably accurate parameters either for sophisticated techniques of analysis or even, as in the case of the North Sea oil development, for conventional design calculations.

It has been shown that the action of 'sampling' causes significant disturbance due both to mechanical deformation and to the inevitable difference in stress history between a sampled element of soil and a similar element in the field (Davis and Poulos, 1966). Marsland (1973) has shown that large differences exist between moduli measured with large diameter plate bearing tests or back-analysed from field observations and moduli measured in the laboratory from carefully taken samples. Sangrey (1968) notes that temperature fluctuations can cause irreversible pore pressure changes and reviews the work of other people who have also observed this. The temperature of the ground is rarely the same as in a normal laboratory and furthermore, it is standard practice to coat samples with hot wax!

Kirkpatrick and Rennie (1975) investigated stress relief effects on blocks of kaolin that had been prepared in the laboratory. Samples were taken from the blocks and kept for a period of time before being tested in the usual manner in a triaxial apparatus. The results suggested that for kaolin, only 20% of the suction pore pressure remained after 50 days. Kaolin is a permeable clay and changes occurring during storage of less permeable natural clay samples are likely to occur more slowly. However, substantial reductions in negative pore pressure might still be expected.

Rowe (1972), in his Rankine lecture, illustrated the enormous importance that soil fabric has in determining the behaviour of the soil in engineering situations. This fabric consists of fissures, organic growth, sand or silt partings etc. and is often not adequately represented in normal samples. Rowe suggested taking large samples for both consolidation and triaxial testing, as well as making very detailed records of the soil fabric. There is no substitute for making such records, but in-situ tests can reduce the number of large samples that should be taken.

The above is not a comprehensive survey of the literature on the problems of conventional sampling and laboratory testing. It merely shows that there are many problems. A growing awareness of these problems has led to an increasing interest in all forms of in-situ testing.

There are also many difficulties with in-situ testing. For instance, it is not possible to follow a given stress path and it is only by testing very quickly that undrained conditions can be simulated, which introduces undesirable rate effects. However, many of the problems of disturbance are overcome and, in general, a large volume of soil is being tested and thus the fabric is more accurately represented. This is not to imply that in-situ testing can ever remove the need for careful laboratory testing, particularly in the cases, where the samples can be reconsolidated back to accurately known in-situ stress levels. It can, however, replace some routine laboratory testing on grounds of both improved accuracy and more favourable economics.

There are many forms of in-situ testing presently available, all with some further advantages and disadvantages. Tests such as the standard penetration test and the Dutch cone test are cheap and simple to perform and in the latter case can provide a very useful continuous profile. However, in order to derive engineering parameters such as shear strength or deformation modulus, empirical correlations must be used. These correlations are subject to a lot of scatter and uncertainty. This is because the SPT blow count or cone resistance is affected by many factors, none of which can be taken fully into account when producing such a correlation. The more expensive plate loading tests or pressuremeter tests, however, do produce data that can be analysed more rigorously to produce engineering parameters directly.

Pressuremeter tests are of particular interest because they are less expensive than plate bearing tests, yet still provide direct measurements of moduli and shear strength, using well-developed theories. As well as this, they also provide an estimate of the in-situ horizontal stresses.

The pressuremeter was developed by Menard (1957) and has subsequently been modified by several workers such as McKinlay and Anderson (1975). However, these conventional pressuremeters are inserted in pre-drilled holes and thus test the disturbed soil in the borehole wall. Consequently, the results of the test are very sensitive to the method of forming the borehole. This drawback to what is otherwise a useful tool for site investigation has led to the development of self-boring pressuremeters.

This article describes such an instrument that has been developed at Cambridge University over the last seven years. It is capable of being inserted into the ground in such a way that the ground suffers minimal disturbance. Once the Cambridge instrument is inserted, then pressuremeter tests can be performed on virtually undisturbed soil. It is possible to obtain a number of soil parameters from the results of these tests (Wroth and Hughes, 1973, 1974).

A very similar series of instruments has been independently developed in France by the Laboratoire des Ponts et Chaussées. This article only includes results obtained from the Cambridge self-boring pressuremeter but similar results have been obtained with the French instrument (Baguelin *et al*, 1974).

Description of the instrument and its operation

The probe, which is illustrated in Figs. 1 and 2, is essentially a miniature cylindrical tunnelling machine that is jacked steadily into the ground. The soil entering the shoe is cut into small pieces by the central rotating cutter and is then flushed to the surface. The flushing fluid, which is normally either water or drilling mud, is pumped down the inside of the cutter rods and up the annular space between these inner rods and the outer rods.

There is a cylindrical membrane fitted over the outside of the instrument which can be expanded against the undisturbed soil as in a conventional pressuremeter test. This membrane is made of neoprene and protected by an outer flexible stainless steel sheath. The radial expansion of the membrane is measured at its mid-plane by three separate pivoted levers which are kept in contact with the membrane by spring cantilevers. Mounted on each spring are electrical resistance strain gauges which accurately indicate the movement of the lever, and hence of the membrane. The gas pressure applied to expand the membrane and the pore pressure in the soil in contact with the membrane are also measured by electrical transducers mounted inside the instrument. The resulting electrical signals are relayed to the surface via a multicore cable that passes inside the armoured nylon tube which supplies the gas pressure.

The drilling rig to insert the probe must provide a vertical force on the outer rod and rotation of the inner rod on which the cutter is fixed. Provision must also be made to pump water down the central rotating shaft. Care must be taken to ensure that the minimum of vibration is transferred down the drill rods to the probe as otherwise initial disturbance could be

†Fugro-Cesco International BV, Leidschendam, The Netherlands
*Reader in Soil Mechanics, Cambridge University Engineering Department.

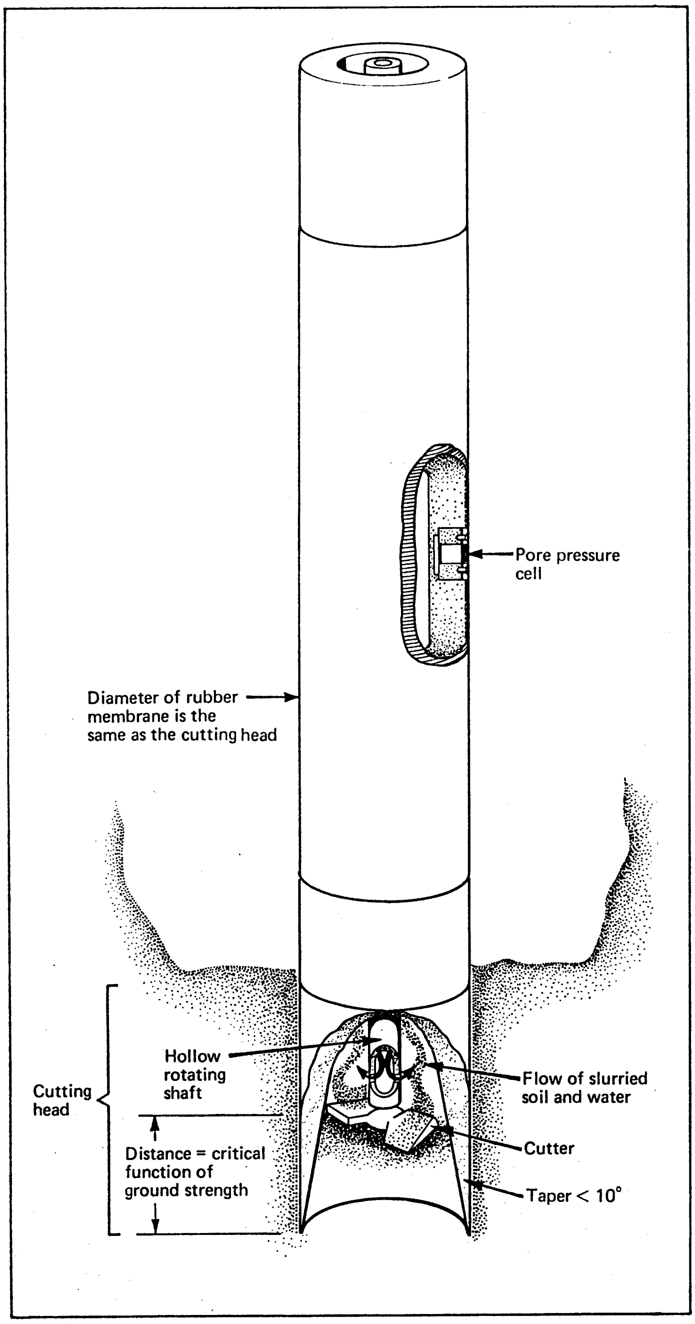


Fig. 1. Schematic diagram of the self-boring pressuremeter before insertion

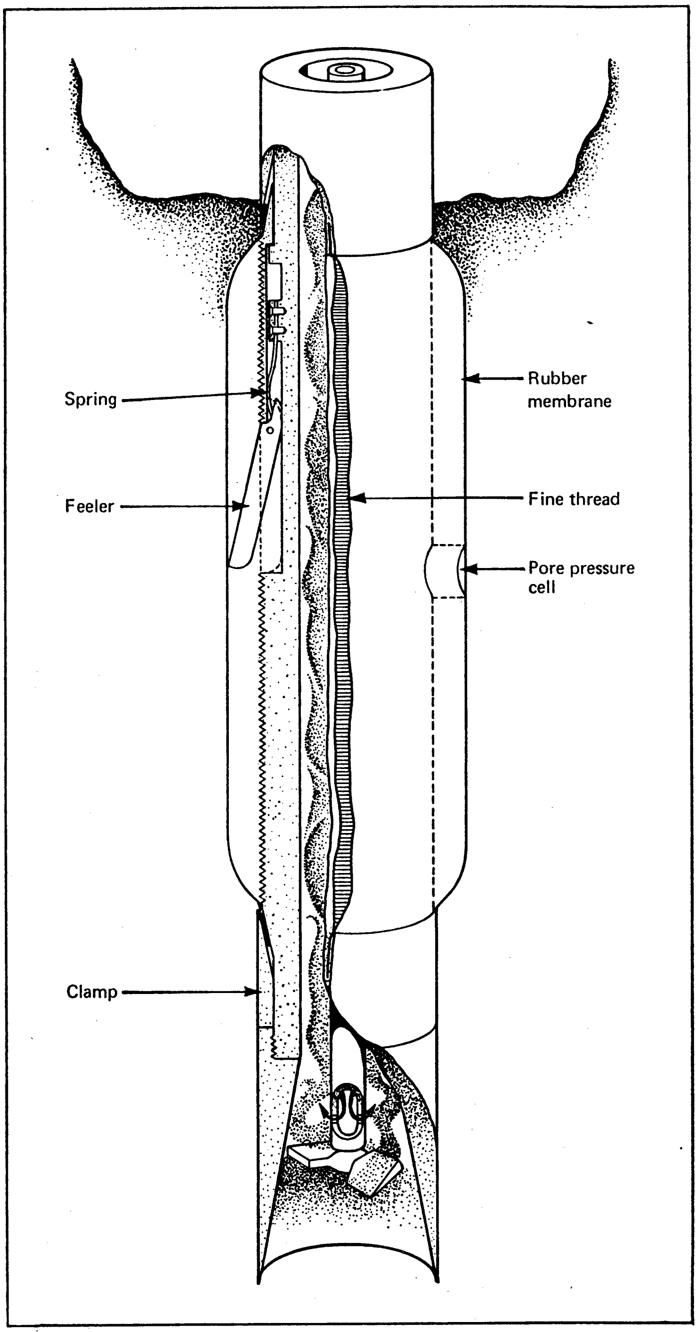


Fig. 2. Schematic diagram of the self-boring pressuremeter during an expansion test

excessive. A drilling rig which was specifically designed for this instrument is shown in Fig. 3. Rotation is provided by a hydraulic motor and vertical movement by hydraulic jacks reacting against ground anchors. The motor and jacks are supplied with oil via a standard Pilcon control panel. This panel may be supplied with high pressure oil either from a purpose-built power pack as shown or from another source such as a suitable shell and auger or rotary drilling rig. The water or drilling mud is supplied from a standard Boyles 7-12 drilling pump. Both engines are only coupled to the frame with flexible pipes, and consequently vibrations are not transferred to the probe.

Other versions of the drilling equipment can be used in soft soils. Indeed, one version has successfully inserted the instrument to a depth of 27m in Boston Blue clay. The equipment shown in Fig. 3 is easily capable of inserting the instrument at a rate of 20cm/min in London clay.

The tests are performed by expanding the membrane with pressure supplied from a gas bottle at the surface. The electrical signals from the instrument are fed into operational amplifiers which produce outputs proportional to the separate averages of the three variables. Present practice is for these output signals to be read directly from a digital volt meter, although equipment for automatic acquisition of the data is now under development.

The tests can either be conducted manually by applying discrete increments of pressure at given intervals of time or automatically, using an electronic control system which provides strain-controlled tests. It has been standard practice to conduct stress-controlled pressuremeter tests, as described above, and thus the test results presented later in the article are from this type of test.

However, as experience with the use of electronic equipment in the field was gained, it became feasible to construct an electronic control system for conducting strain-controlled pressuremeter tests. This device, developed in conjunction with Cambridge Insitu, monitors the strain output signal and adjusts the frequency of small increments of pressure to provide the desired strain rate. The authors are of the opinion that tests conducted at a constant rate of strain (as is common in other forms of soil testing) are superior. Consequently, it is not planned to conduct any further stress-controlled undrained tests with the Cambridge instrument.

The complete equipment* is normally operated by an engineer and a technician with tests conducted at 1m intervals down a vertical profile. For commercial purposes this could be achieved under favourable conditions at a rate of about one test per hour. It is suggested, however, that for many applications the equipment will

*Camkometers are manufactured by the licencees Cambridge Insitu, of Little Eversden, Cambridge CB3 7HE, from whom the complete equipment, or any part of it, is available commercially worldwide.

be used in conjunction with a conventional rotary drilling rig. This would provide increased flexibility such that (*i*) the borehole could be drilled and lined in a conventional manner through layers such as gravel, (*ii*) claystones could be dealt with, and (*iii*) other routine tests could be performed in the same borehole.

Degree of disturbance during insertion

Hughes (1973) was able to observe the degree of disturbance caused by the insertion of the self-boring pressuremeter under controlled laboratory conditions using the radiographic technique developed at Cambridge (Arthur, James and Roscoe, 1964).

Fig. 4 shows special reproductions of two X-radiographs demonstrating the very small amount of disturbance. The upper picture shows an area within a block of normally consolidated kaolin before insertion of the instrument. The lower picture shows the same area after the insertion of a prototype instrument with the clay remaining stressed. Accurate measurement of the positions of the lead shot showed that, in this case, (*i*) radial displacement of the soil adjacent to the instrument was less than 0.5% of the radius of the instrument and (*ii*) the axial movement of the

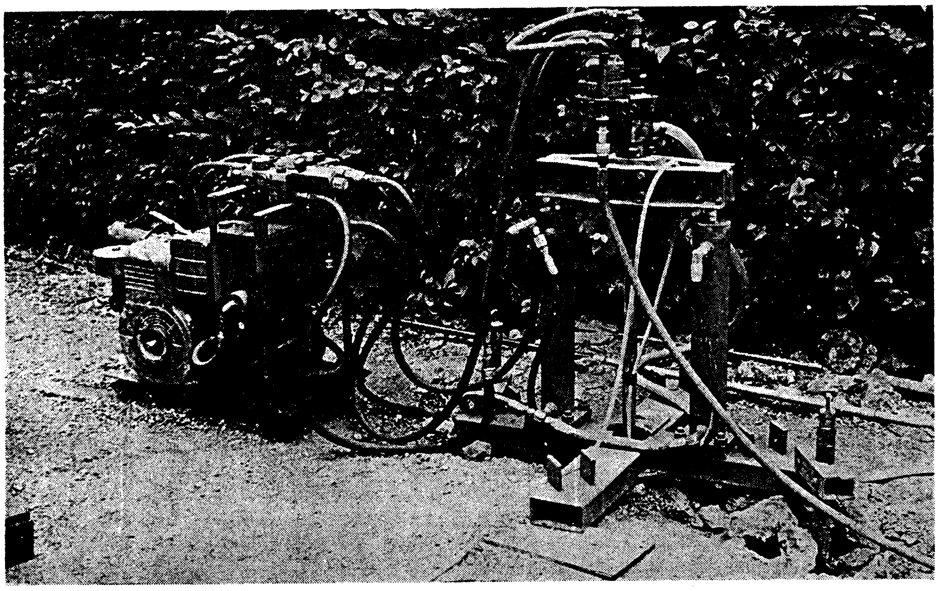


Fig. 3. View of drilling equipment

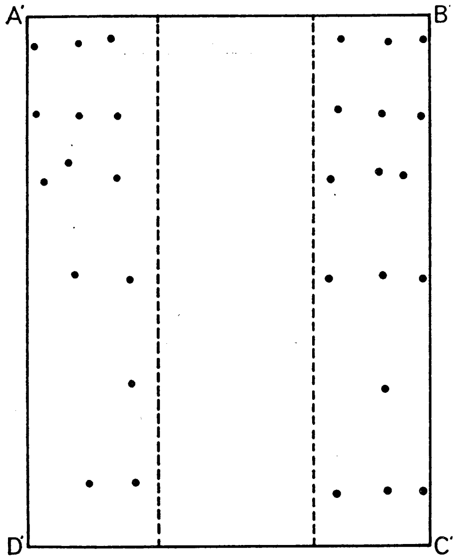
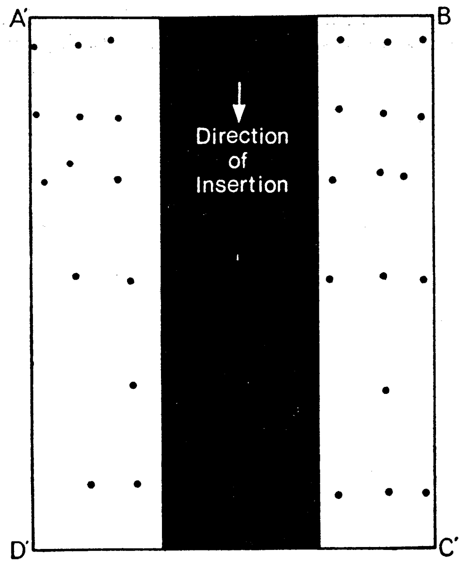


Fig. 4. Reproductions of two radiographs showing minimal disturbance of the clay, the upper picture before insertion of the instrument and the lower, after insertion



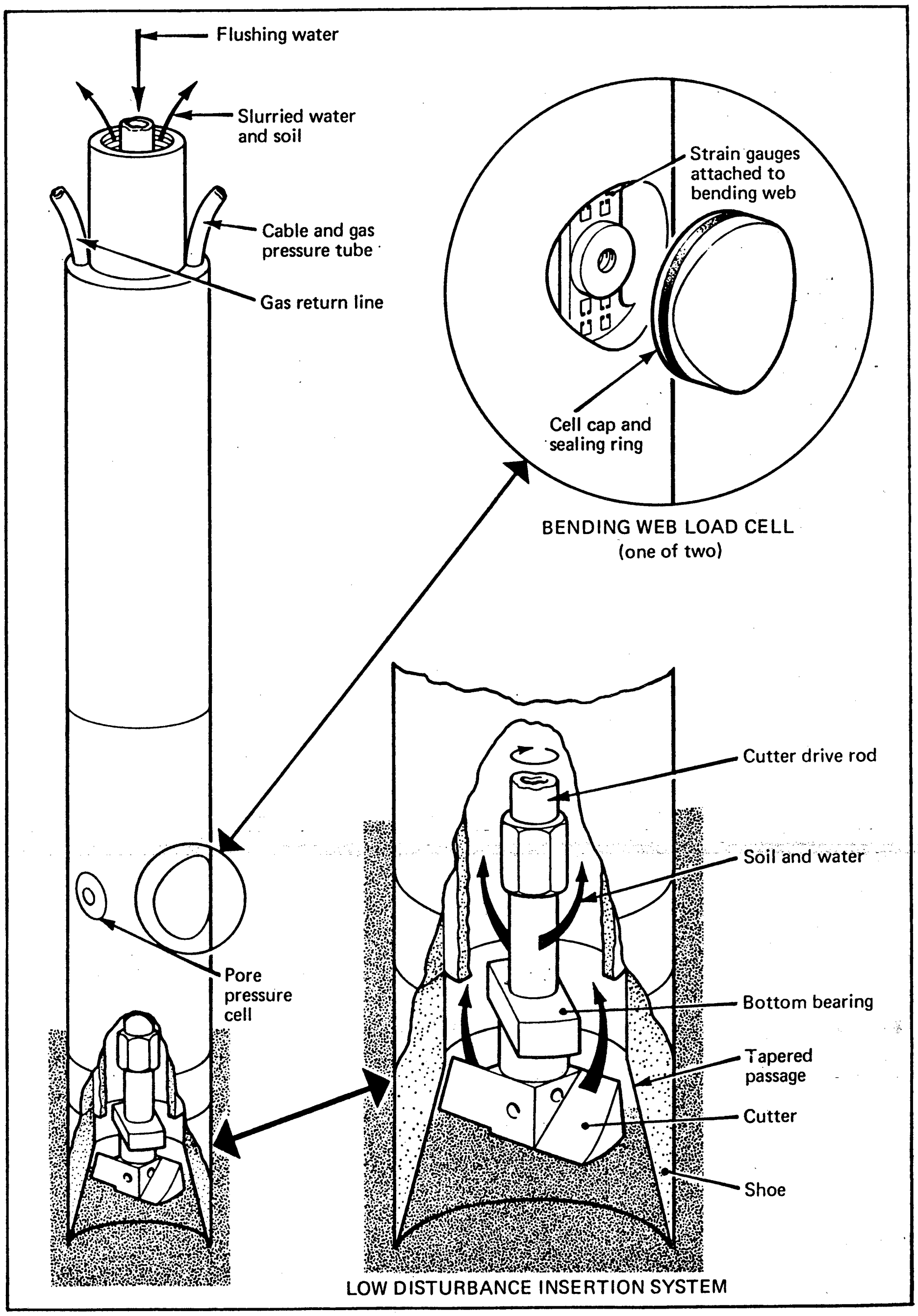
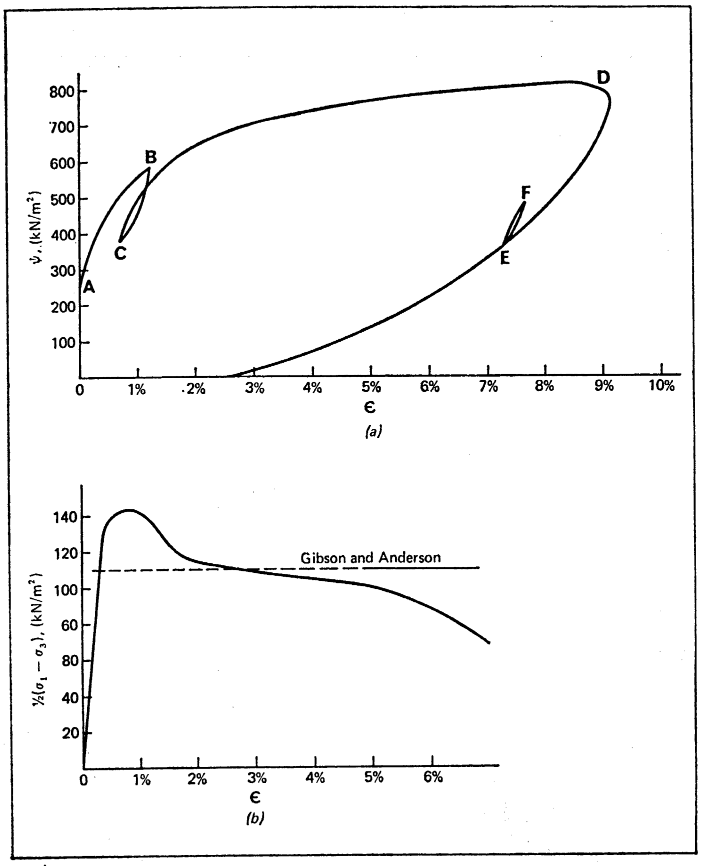


Fig. 5. Self-boring lateral stress cell



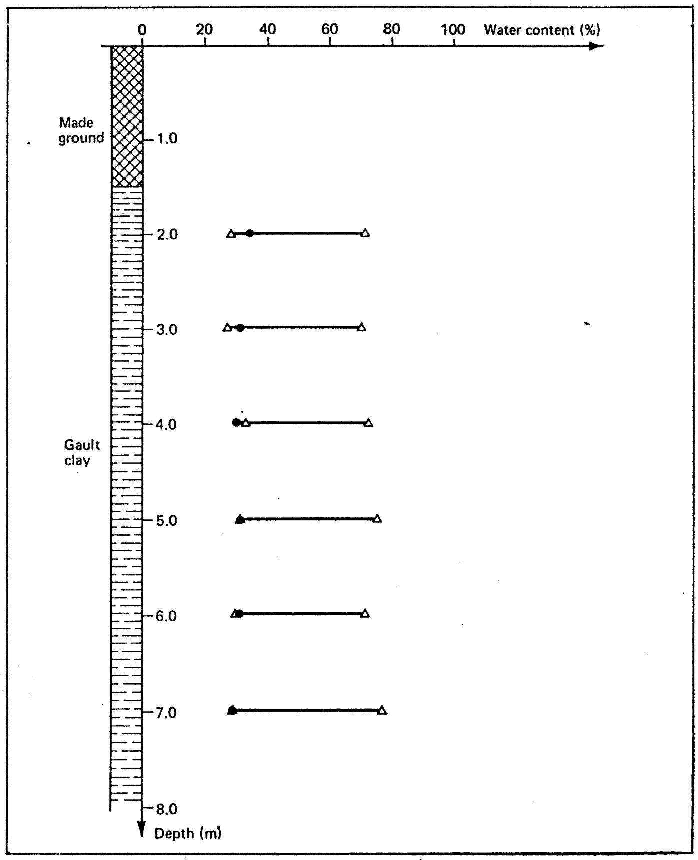


Fig. 6. Results of pressuremeter test in Gault clay. Fig. 7 (right). Borehole log and index test results for Gault clay at Madingley

soil was confined to a very thin zone, probably within 1 or 2mm from the surface of the instrument and too narrow to be observed.

Both the magnitude and the direction of the radial disturbance are dependent on the position of the rotating cutter relative to the leading edge of the cutting shoe. If the cutter is too near the leading edge, stress relief on the base of the hole allows the soil beneath to move radially inwards, and upwards; whereas if the cutter is too far back the effect is analogous to driving in a solid shaft and the displacements are radially outwards and downwards. The strength and in-situ stresses of the soil also influence these movements, and although it is possible to optimise the position of the cutter for a given clay, it is unlikely to be the same for all soils.

Unfortunately, it is not, at present, possible to examine heavily overconsolidated clays in the above manner because of the very large pressures required to simulate the maximum overburden pressure. The cutter setting for the use of the self-boring pressuremeter in Gault and London clay was determined by inserting the self-boring lateral stress cell (shown in Fig. 5) into Gault clay using a number of cutter positions.

After the instrument was drilled to a particular depth, the output of the lateral stress cell was monitored for periods up to 160 hours. Several cutter positions were tested and a position was selected that produced no observed change of lateral stress with time. The same cutter position was confirmed to be appropriate for the top 8m in London clay.

Interpretation of the undrained expansion test

Baguelin *et al.* (1972), Ladanyi (1972) and Palmer (1972) have all separately shown that the shear stress τ at the wall of the expanding cavity during a pressuremeter test is given for small strains by

$$\tau = \tfrac{1}{2}(\sigma_r - \sigma_\theta) \simeq \varepsilon \frac{d\psi}{d\varepsilon} \qquad \dots (1)$$

where cylindrical coordinates are used,

ψ is the pressure applied by the membrane, and

ε is the radial expansion of the membrane divided by the initial radius.

This analysis has been applied to the results of a typical pressuremeter test conducted in the Gault clay, shown in Fig. 6*a*. The resulting stress-strain curve in Fig. 6*b* indicates the initial shear modulus and the sensitivity of the clay as well as its peak and residual§ undrained shear strengths. It would be possible to obtain from this an average value of strength over a range of strain. However, it is considered easier to use the method of Gibson and Anderson (1961), which idealises the behaviour of the soil as linearly elastic with shear modulus G and perfectly plastic after the undrained shear strength c_u has been reached. This leads to the expression

$$\psi = \sigma_l + c_u \ln\left\{\frac{\Delta V}{V} - \left(1 - \frac{\Delta V}{V}\right)\frac{\sigma_h}{G}\right\} \dots (2)$$

where V is the current volume of the membrane

σ_l is the limit pressure

σ_h is the initial in-situ lateral stress.

For a self-boring pressuremeter, where the borehole does not have to be reloaded back to the original in-situ stress state, this equation reduces to

$$\psi = \sigma_l + c_u \ln\left(\frac{\Delta V}{V}\right) \qquad \dots (3)$$

Eqn. 3 relates the two observed parameters ψ and $\frac{\Delta V}{V}$ and can be used to derive a value for the undrained shear strength c_u. It is, however, only valid when the soil has gone plastic, that is when $\psi > \sigma_h + c_u$.

This method has been used to produce the value for c_u of 110kN/m² in Fig. 6*b*, which provides a reasonable average for the stress-strain curve derived by means of Eqn. 1. This average value and also the residual strength obtained from the stress-strain curve should be more directly comparable with shear strengths derived from plate or cone tests than is the peak strength.

Values of undrained Young's modulus E_u can be obtained from the loading cycles such as BC or DE as follows. The gradient of the curve is

$$\frac{\Delta\psi}{\Delta\varepsilon} \simeq \frac{2\Delta\tau}{\Delta\gamma} = 2G = \frac{E_u}{1+\nu_u} \qquad \dots (4)$$

where G is the shear modulus,

γ is the engineering shear strain,

ν_u is the value of Poisson's ratio for the undrained case, assumed to be 0.5.

(For an isotropic elastic material, the shear modulus is independent of the drainage conditions, unlike the value of Young's modulus, and it is therefore a superior parameter to use. For the undrained case $E_u = 3G$).

It is also possible to derive a value for

§Although the term residual shear strength is used here and elsewhere in the article, it must not be confused with the concept of a residual value of ϕ' as used in long-term slope stability analysis. This value of residual strength is simply a value of undrained strength after a certain amount of shearing.

the initial modulus by using the expression on the initial loading part of the pressuremeter curve. However this value of modulus is very sensitive to even slight disturbance during insertion of the instrument, and is less representative than the modulus obtained from a reloading cycle. For the stiff London clay, reload moduli have been evaluated over a stress increment of 100kN/m² and initial moduli over a stress increment of 200kN/m². These correspond approximately to increments in engineering shear strain of 1 and 2%. It is simpler to use a stress increment with stress controlled test data and satisfactory agreement is obtained with the results of other tests. However, for the soft clay initial moduli have been evaluated over an increment in engineering shear strain of 1%, as the size of stress increment discussed above is clearly too large for this case.

Values of in-situ horizontal total stress σ_h can also be obtained from pressuremeter tests by observing the pressure at which the membrane starts to expand radially, e.g. point *A* in Fig. 6*a*. This method of determining the horizontal stress has been compared with the self-boring stress cell method and, at present, little difference can be detected between the two methods. The stress cell provides a superior measurement but the pressuremeter provides a value more quickly.

Comparison of results from stiff clays with results of other tests

The self-boring pressuremeter has been used at two sites of heavily overconsolidated clay (Windle and Wroth, 1977). One site, on Gault clay, is close to the new Cambridge geotechnical centrifuge at Madingley. A borehole log and profile of liquid and plastic limits and natural water content is given in Fig. 7.

The other site on London clay is at Hendon, North London. It has been used extensively by the Building Research Establishment and has been described by Marsland (1974).

(a) Shear strengths

Fig. 8*a* shows values of peak undrained shear strength from pressuremeter tests conducted at three different rates of loading in the Gault clay. Rates α, β and γ approximate to reaching 10% strain in 6, 12 and 24 minutes respectively, although the strain rates decrease with depth as the soil becomes stronger and requires more pressure to induce a given shear strain.

It can be seen that the different rates of loading have a marked effect on the peak shear strength and considerably more effect than would be expected. However the average or residual shear strengths plotted in Figs. 8*b* and 8*c* respectively show no such variation. This is borne out by an examination of the stress-strain curves. The tests conducted at faster rates show a sharper peak and considerably more brittleness or strain softening, whereas the tests at the slowest rate γ show virtually none.

Similar behaviour was also noted by Parry (1971) in undrained triaxial tests on soft clays. He observed that, if the tests were carried out rapidly, a sharp peak could be seen in a plot of shear stress versus shear strain. But at slower strain rates the curve showed no peak. He ascribed this to either structural features in the soil or pore pressure effects. Structural features of a similar nature will exist in stiff clays. Similar pore pressure effects might well occur during a pressuremeter test in an over-consolidated clay because measurements of effective stress indicate that substantial positive pore pressures are generated during such a test. This is a consequence of the total stress path induced in the soil being different from that induced in a triaxial test.

It can be seen that the values of average or residual strength for any one rate show more scatter than the values of peak strength. This is because the Palmer method is sensitive to operator interpretation which leads to "smoothing" of the data, and a suppression of the scatter.

The profiles of peak strengths from Fig. 8*a* are compared with peak strengths obtained from quick undrained triaxial tests in Fig. 9*a* and also with the results of a number of Dutch cone tests in Fig. 9*b*‡. The Dutch cone tests, of which a typical result is shown in Fig. 9*c*, were conducted with an electrical Fugro type

‡ The peak strengths are compared in order to provide figure continuity. It is appropriate to compare the pressuremeter average or residual strengths with the cone strengths, which can be done by comparing Figs. 8*b*, 8*c* and 9*b*.

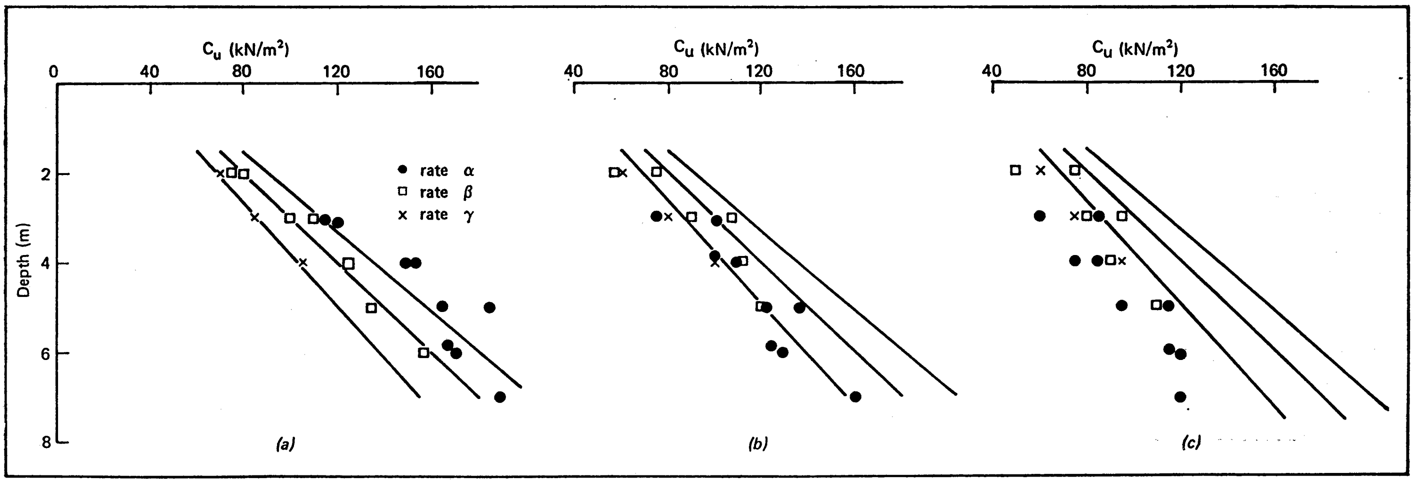


Fig. 8. Undrained shear strengths of Gault Clay measured by the pressuremeter— (a) peak values, (b) average values, and (c) residual values

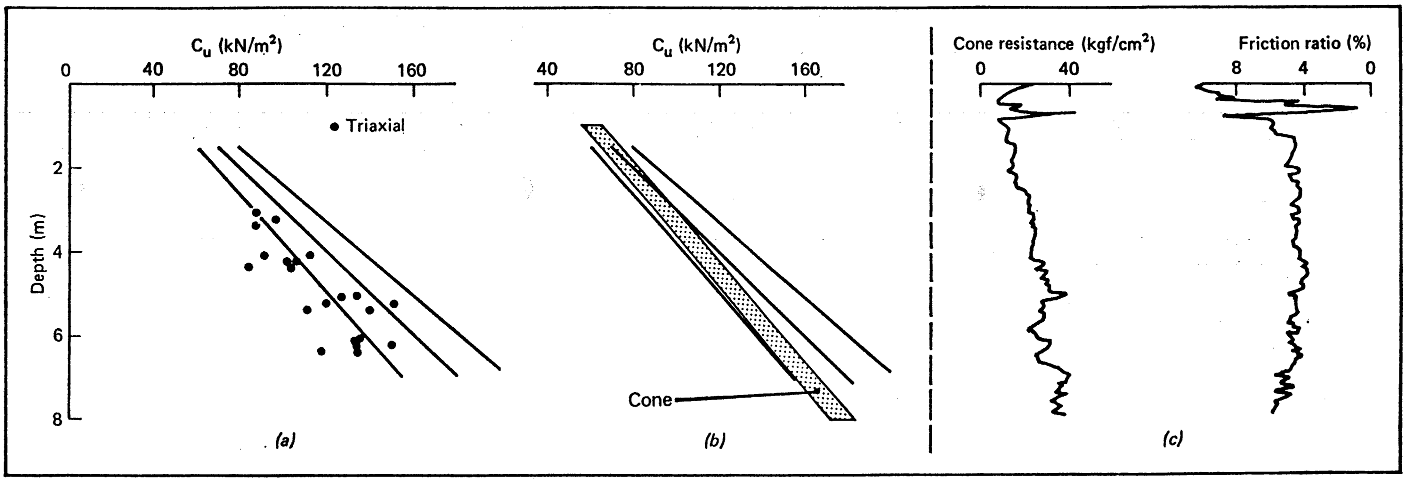
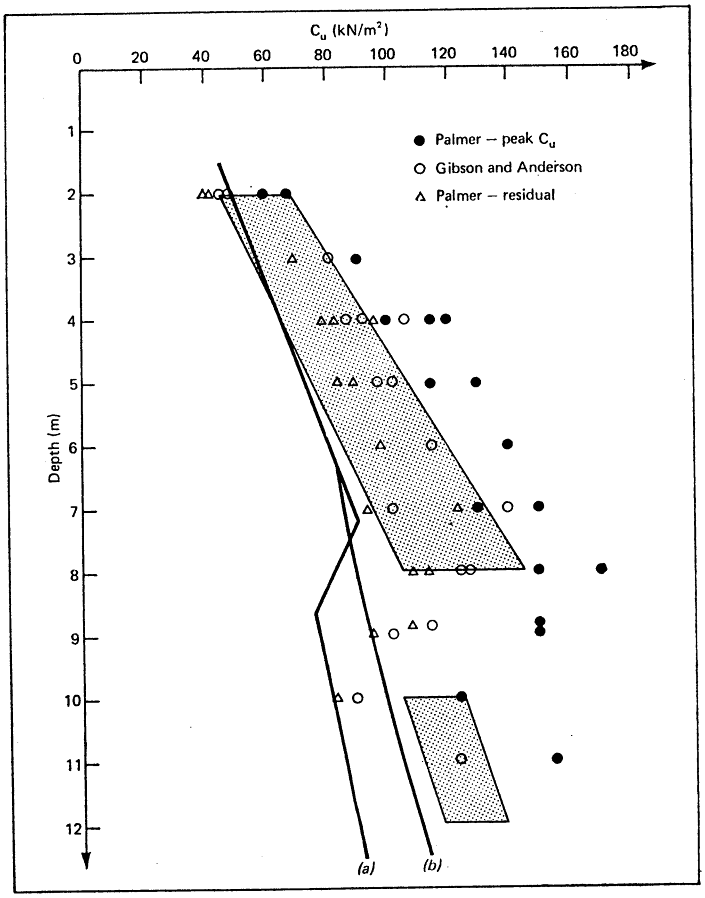


Fig. 9. Comparison of undrained shear strengths determined from pressuremeter test results and strengths from (a) triaxial tests and (b) Dutch cone tests, with (c) a typical Dutch cone test result



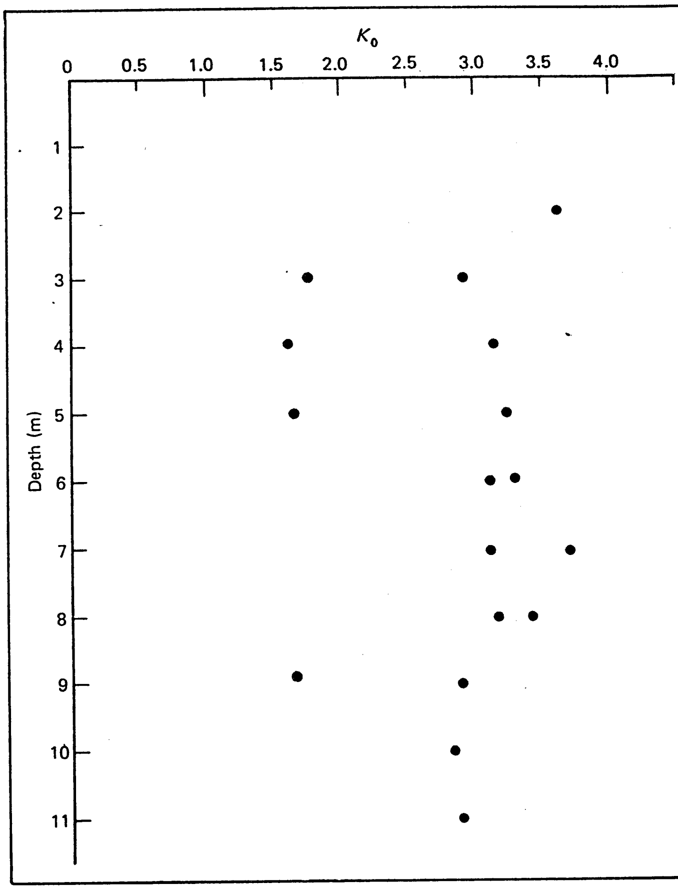
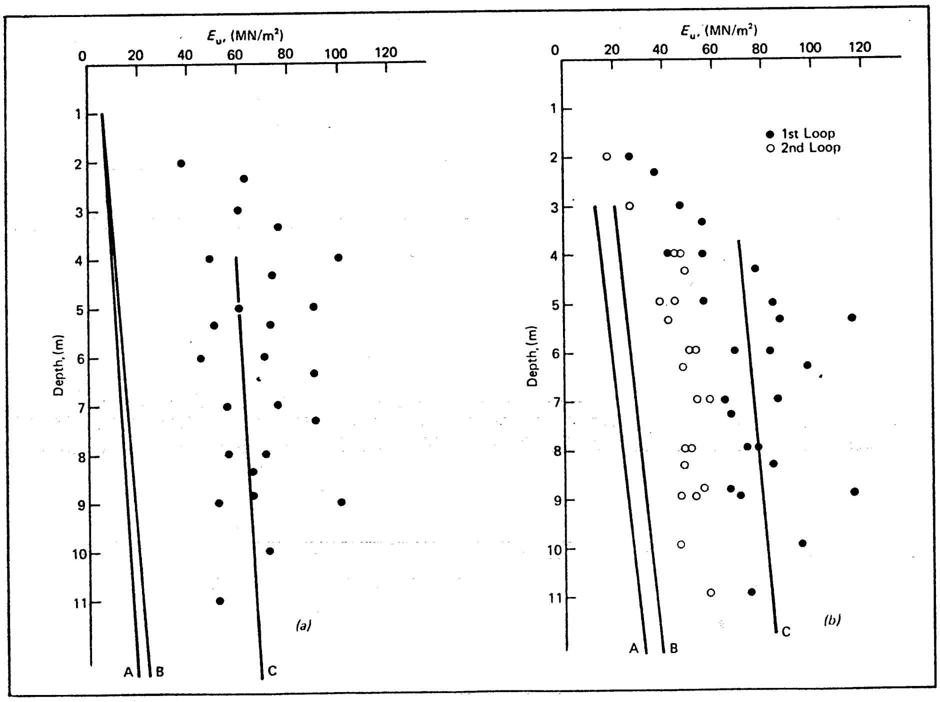


Fig. 10 (above). Comparison of undrained shear strengths of London clay determined by (i) the pressuremeter (individual points), (ii) the triaxial test and plate bearing test (lines (a) and (b)) and (iii) cone tests (stippled area)

Fig. 11 (below). Undrained moduli obtained from various tests in the London clay, (a) from first loading and (b) from reloading

Fig. 12 (above right). Values of K_0 in London clay obtained from pressuremeter tests



cone at the standard rate of penetration. The results were all very similar and show the site at Madingley to be laterally uniform. A cone factor or ratio of cone resistance to undrained shear strength of 20 was used to analyse the results. The agreement between the results of all three tests is very good.

Fig. 10 shows similar values of undrained shear strengths of the London clay from the site at Hendon. The tests were run at the rate β giving a total test time of about an hour. The solid circular points ● represent peak shear strengths, the open circular points O represent average shear strengths and the triangular points △ represent residual shear strengths. Lines (*a*) and (*b*) represent the average of shear strengths determined by Marsland (1971) for the same site from triaxial tests on 38mm diameter specimens (line *a*); and from triaxial tests on 98mm diameter specimens as well as from large diameter plate bearing tests (line *b*). The stippled area represents the strengths derived from Dutch cone tests reported by Marsland (1974). The tests were again analysed using a simple cone factor of 20 as this is conventional practice. Marsland discusses the effects due to the different rates of testing. These will be important but cannot be taken into account as insufficient is known about them with respect to pressuremeter testing and the correlations with cone testing.

The residual strengths give the best agreement with Marsland's determinations, although they are consistently larger. The pressuremeter tests conducted at a constant rate of strain show that such tests produce lower values of residual strength. Had it been possible to conduct this type of test at Hendon then, almost certainly, values of residual strength would have been obtained that were lower than Marsland's lines *a* and *b*. Further discussion on this point will have to be left until constant rate of strain pressuremeter tests have been conducted at Hendon. However, the existing data indicate that satisfactory agreement between two types of test is likely.

(b) Young's moduli

Secant values of Young's moduli have been obtained for the results of tests in London clay. Fig. 11*a* shows values from the initial loading and Fig. 11*b* those from subsequent reloading loops such as *BC* or *EF* in Fig. 6*a*. They are compared with moduli obtained by Marsland (1973) from triaxial tests on 38mm diameter specimens (line *A*), triaxial tests on 98mm diameter specimens (line *B*) and 865mm diameter plate bearing tests (line *C*). The findings confirm Marsland's conclusions about sample disturbance. Surprisingly, in view of the anisotropy of London clay, the values agree closely with those from large diameter plate bearing tests.

Ward (1971) suggests a linear variation of the moduli from 50MN/m^2 near the top of the London clay to about 160MN/m^2 at its base, which agrees well with both the pressuremeter and the plate test values.

Moduli were also determined (open circular points O in Fig. 11*b*) from the reloading portion of loops such as *EF* shown in Fig. 6*a*. It was hoped that these would be similar to those obtained from loops such as *BC*. This would enable pressuremeter tests to be run without the first loop, which would enable the Palmer analysis to be applied with more accuracy. Unfortunately the moduli are consistently lower than those from the first loops. Interestingly they show considerably less scatter, probably because the soil at this stage of the test has undergone a certain amount of remoulding.

(c) In-situ horizontal stresses

Values of in-situ horizontal stress have been derived from the test results for London clay. At this site, the effects of under-drainage are thought to be negligible near the surface and consequently the borehole water level was used to calculate a value for the pore pressure at the test depth. This value has been used to determine values of K_0 which are plotted versus depth in Fig. 12. There is a certain amount of scatter in the values and this is probably partly due to variations in aspects of the drilling technique such as rate of advance. In spite of these difficulties, yet to be resolved, the use of self-boring instruments is one of the few methods of in-situ measurement of later stress in a deposit where K_0 is greater than 1.

Bishop, Webb and Lewin (1965) determined a value of 3.4 for K_0 at a depth of 10m at the Ashford Common Shaft. This value is of the same order as the results shown in Fig. 12.

Comparison of results from soft clays with results of other tests

The self-boring pressuremeter has been used at many sites of soft clay. The results from one site at Canvey Island are chosen to illustrate the basic trends. These trends are found to be the same for all the sites of soft clay tested to date.

The site consists of a surface layer of soft to very soft silty grey clay underlain by dense sand at about 8m below ground level. The clay is lightly overconsolidated and is typical of the soft estuarine clays found in the area. The profile of natural water content and Atterberg limits obtained for this layer is shown in Fig. 13. A number of pressuremeter tests have been carried out, again at rate β, in one vertical profile.

(a) Shear strengths

The peak undrained shear strengths are plotted against depth in Fig. 14*a*. Values of shear strength obtained from vane shear tests are plotted to the same scale, but separately (for purposes of clarity) in Fig. 14*b*. The mean line for the pressuremeter test results of Fig. 14*a* has been repeated in Fig. 14*b* and indicates consistently higher strengths than those from the vane shear test. This pattern has been confirmed for other deposits of soft clay both for this instrument and for the French self-boring pressuremeter. It is suggested that the higher strengths obtained by the self-boring pressuremeter may be due to (*i*) the greater disturbance caused by insertion of the vane and (*ii*) strain rate effects. In order to prevent drainage, the pressuremeter test has to be conducted fast with correspondingly high rates of strain, of the order of 0.8% per minute, for the soil adjacent to the membrane. It is only possible to compute a rate of relative displacement (and not of strain) for the vane test so that no direct comparison can be made.

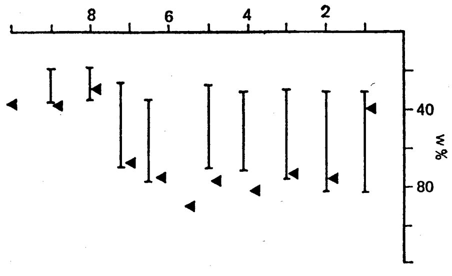


Fig. 13. Profile of natural water content and index tests for the soft clay at Canvey Island

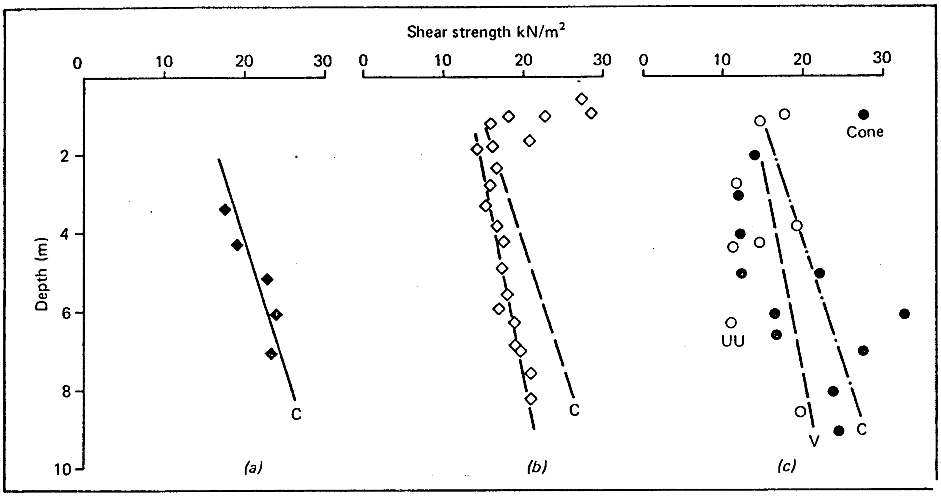


Fig. 14. Profiles of undrained strength obtained at Canvey Island from (a) Camkometer, (b) vane, and (c) Dutch cone and triaxial tests

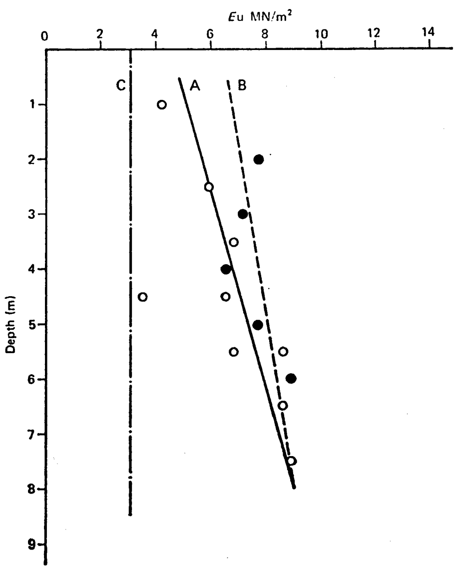


Fig. 15. Young's moduli of soft clay at Canvey Island and Mucking Flats

Results of Dutch cone tests, analysed using a cone factor of 9, and of unconsolidated undrained triaxial tests are shown in Fig. 14*c*. It is notable that in comparison with results from the Camkometer and the vane, there is much greater scatter for both the cone and triaxial tests, and that the average values of strength are lower. However, the strengths are not strictly comparable because of the different orientations of the possible failure planes in the various tests, so that if there is any anisotropy of strength the effects will be included in the results.

(b) Young's moduli

Secant values of initial moduli are shown (open circular points O) in Fig. 15 Also shown are values of initial moduli (solid circular points ●) from a nearby site of the same clay at Mucking Flats. The site has been described by Serota and Lowther (1976) and is essentially very similar to the site at Canvey Island. Line *A* is the result of a least squares regression analysis of all the results and line *B* is the result of a similar analysis of the data from Mucking Flats. Serota and Lowther (1976) quote some results of work performed at Imperial College. These results are shown as line *C* and consist of data from triaxial tests and also back-analysis by finite element computations of the observed deformations of the trial embankments.

The pattern of the results is as expected. The triaxial test results are low due to the inevitable sample disturbance. The embankment observations are known to include some plastic distortion, although in the computations the movements were assumed to be entirely elastic. This would, of course, considerably reduce the equivalent single value of *E* needed to match the observations.

(c) In-situ horizontal stresses

Values of K_0 have been derived from the test results and are shown plotted versus depth in Fig. 16. This time there is not a problem with under-drainage and standpipe observations of the water table confirm measurements taken with the pore pressure transducers. These values of K_0 of about 0.62 confirm that the clay is lightly overconsolidated.

Discussion on results presented

It can be seen that the strength values, particularly for the soft clay, are consistently higher than values obtained from other methods of testing. This means that these values of strength obtained from self-boring pressuremeter tests cannot be used in conventional design calculations as unsafe designs would be produced. This is because conventional stability calculations are based on empirical correlations, which are not always appropriate

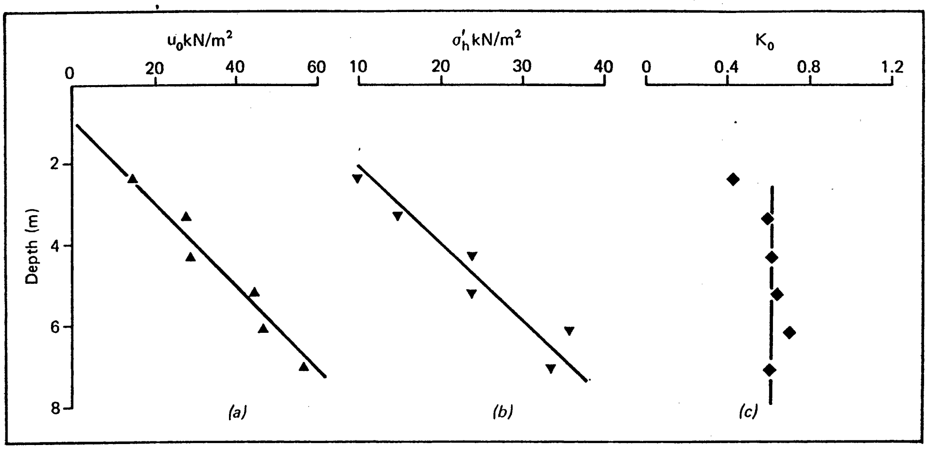


Fig. 16. Profiles of in situ stress at Canvey Island

as can be seen in the case of embankments on soft clay foundations.

The use of strengths derived from vane shear tests will produce unsafe designs as noted by Parry (1971). Bjerrum (1973) proposed a correction factor based on plasticity index. However, Ladd (1973) and La Rochelle *et al* (1974) have already noted serious non-conservative errors in Bjerrum's correction factor. It can be seen that problems with the use of approaches based on values of undrained shear strength are not restricted to values obtained from self-boring pressuremeter test results. There are good reasons for this, as the effects of anisotropy and progressive failure are not taken into account in normal stability calculations.

However, the values of Young's modulus and horizontal effective stress are of direct use. The values of horizontal effective stress agree well with the results of other methods of tests, and also with experience. This method appears to be accurate and substantially quicker than any other in-situ method of determining the lateral stress. Furthermore, it is the only in-situ method, presently available, for determining the horizontal stress in a deposit of stiff clay where cells, such as the Glötzl type, cannot be pushed into the ground. The values of Young's modulus are higher than equivalent values determined from other tests. However, work by Marsland (1973), Ward (1971) and others indicates that these higher values are more appropriate to the actual field behaviour.

Tests on sands

The equipment described has also been used to insert the instrument into sand. Tests have been conducted at the Wash, at Sizewell Power Station and in the micaceous residue at Kernick (Hughes, Wroth and Windle 1976).

The results are encouraging and indicate that the instrument works well in sands. There are a number of problems associated with the choice of cutter geometry and thus values of horizontal stress and initial modulus show a certain amount of scatter. Work is proceeding on this problem using laboratory samples of sand (Clarke 1976).

Advantages and disadvantages of this method of testing soils

Advantages

(*i*) It is possible to perform tests on virtually undisturbed soil. Although some slight disturbance is inevitable it can be very much less than the disturbance associated with so-called "undisturbed sampling".

(*ii*) It is possible to obtain a number of parameters from one test. These parameters are the undrained shear strength c_u, the shear modulus G or undrained Young's modulus E_u and the in-situ horizontal total stress.

(*iii*) Parameters can be derived from test results using well-developed theories of cavity expansion without recourse to empirical correlations.

(iv) In many cases, this method of testing produces very much less scattered data than other methods. This is due to the elimination of many of the variables associated with other forms of testing such as amount of disturbance during sampling or trimming, time between finishing the borehole and starting to test or sample, and effects of piping in a borehole in sand or soft chalk below the water table.

(*v*) It is possible to obtain results very quickly when necessary. The test data can easily be processed on site with the aid of a simple pocket calculator or slide rule.

Disadvantages

(*i*) The instrument will not penetrate gravel, boulder clay, claystones or similar material.

(*ii*) As in other forms of test, the orientation of the failure planes and the mode of deformation will usually be inappropriate to the field situation.

(*iii*) There is no control of the total or effective stress path. In practice, only two stress paths can be followed—one corresponding to an undrained test and the other corresponding to a fully drained test.

(*iv*) In order to reduce drainage effects, undrained tests must be performed at high rates of strain. This leads to undesirable rate effects which introduce small errors into the analysis of the data.

(*v*) The instrument is complex by present-day standards. The authors, however, believe that present developments in soil mechanics will force the site investigation industry to adopt more sophisticated techniques.

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