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INFLUENCE OF CUTTING SHOE SIZE IN SELF-BORING PRESSUREMETER TESTS IN SENSITIVE CLAYS

by K. T. Law and W. J. Eden

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Influence of cutting shoe size in self-boring pressuremeter tests in sensitive clays

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This paper examines the influence of the cutting shoe size of a pressuremeter apparatus in the light of soil behaviour after some unloading and loading. An oversized cutting shoe creates a gap between the borehole and the pressuremeter probe, causing a stress release with a probable consequence of overestimating the shear strength of the soil. An undersized cutting shoe imposes a certain load to the surrounding soil prior to the pressuremeter test, thus introducing errors in measuring the stress-strain relationship of the soil.

To gain a quantitative idea of such an influence, Cambridge self-boring pressuremeter tests were carried out on Leda clay at two sites in the Ottawa region. It was found that in the case of an oversized cutting shoe, both the modulus and shear strength were overestimated by about 30 and 80% respectively. In the case of an undersized cutting shoe, the shear resistance was reduced at small strains. In both cases, however, the stress-strain relationship beyond a moderate strain (5%) showed little dependence on the cutting shoe size.

Cet article présente une étude de l'influence de la dimension de la trousse coupante d'un pressiomètre autoforé, tenant compte du comportement du sol après déchargement et rechargement. Une trousse coupante surdimensionnée crée un espace entre la paroi du forage et la sonde pressiométrique, produisant un relâchement des contraintes qui a pour conséquence une surestimation de la résistance du sol. Une trousse coupante trop petite impose un certain chargement du sol autour de l'appareil avant l'essai, introduisant ainsi des erreurs dans la mesure de la courbe effort-déformation du sol.

Dans le but d'obtenir des indications chiffrées sur ces influences, des essais au pressiomètre autoforé de Cambridge ont été réalisés dans l'argile Leda sur deux sites de la région d'Ottawa. On a constaté que, dans le cas d'une trousse coupante surdimensionnée, le module et la résistance au cisaillement sont surevalués par 30 et 80% respectivement. Dans le cas de la trousse coupante de petit diamètre, la résistance au cisaillement à faible déformation est diminuée. Dans les deux cas, la relation contrainte-déformation au delà d'une déformation de 5% est peu affectée par la dimension de la trousse coupante.

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Introduction

To correctly evaluate the engineering behaviour of the sensitive clays in eastern Canada, the disturbance caused by sampling or *in situ* testing must be taken into account. The magnitude of the problem of disturbance has been demonstrated by Eden (1970) when it was shown that undrained strengths determined from block samples were twice that of strengths measured on tube samples at the same site. Various methods have been devised to minimize the disturbance, e.g., by reconsolidating the samples to the *in situ* stress condition (Bozozuk 1972; Bjerrum 1973). Such procedures raise questions about their validity, particularly if the sample undergoes a significant volume change upon reconsolidation.

In situ testing with the field vane or static cone always causes some disturbance due to the insertion of the test device. The Menard pressuremeter test requires that a hole be prebored before insertion of the pressuremeter. The self-boring pressuremeter

(Baguelin *et al.* 1972; Wroth and Hughes 1973) offers the opportunity to reduce the disturbance to a minimum. From the test information, the entire stress-strain relationship can be derived through interpretation of the pressure-strain relationship measured directly by the apparatus.

The Division of Building Research has acquired a Cambridge self-boring pressuremeter and tests have been conducted at a number of sites. This paper describes one aspect of the investigations, that of the influence of small differences between the membrane diameter and the diameter of the borehole formed by the cutting shoe of the pressuremeter. Other factors influencing pressuremeter tests are considered in a companion paper (Eden and Law 1979).

Interpretation of Undrained Pressuremeter Test

The method of interpreting an undrained pressuremeter test was described in three papers (Baguelin *et al.* 1972; Ladanyi 1972; Palmer 1972) published in

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three major geotechnical journals at about the same time. Working independently, these authors arrived at almost the same solution. The solution is for the expansion of a thick-walled cylinder, in which the material of the cylinder is considered to be an assemblage of a great number of thin concentric cylinders all responding to a common stress-strain law. At a given expansion of the bore any such thin cylinder will be strained to a given point on a common stress-strain curve, with no change in volume. For small strains under plane strain condition, the maximum shear stress, τ , is given by

$$[1] \quad \tau \simeq \epsilon \frac{dP}{d\epsilon}$$

where P = total radial pressure and ϵ = circumferential strain. For brevity in the following, ϵ will simply be called strain.

Equation [1] is sometimes written in another form to facilitate the interpretation of pressuremeter tests in which the volume increase ratio $\Delta V/V$ is measured:

$$[1a] \quad \tau \simeq \left(\frac{\Delta V}{V} \right) \frac{dP}{d(\Delta V/V)}$$

It can be easily shown that [1] and [1a] are identical for small strains.

Theoretically, for a homogeneous soil, [1] applies to any point in the soil provided that both P and ϵ refer to the same point. In the Cambridge pressuremeter, P and ϵ are measured at the probe and soil interface; the shear stress τ , therefore, refers to that point.

Equation [1] can be rewritten as

$$[2] \quad \tau = \frac{1}{2.3} \frac{dP}{d \log_{10} \epsilon}$$

Both equations can be solved graphically as shown in Fig. 1 *a* and *b*. In the following sections, either one or the other solution will be used depending on which one is more illustrative.

The strain ϵ under no volume change can be related to the shear strain γ :

$$[3] \quad \gamma = 2\epsilon$$

The initial slope on a τ versus ϵ curve will therefore be equal to $2G$, where G is the initial shear modulus of the soil. The modulus of compression, E , is obtained from:

$$[4] \quad E = 2(1 + \nu)G$$

where ν = undrained Poisson's ratio.

Many factors exert an influence on the stress-strain

FIG. 1. Graphical solutions for shear stress τ , from pressuremeter test: (a) pressure vs. strain; (b) pressure vs. log of strain.

behaviour of the soil obtained from the pressuremeter test. Some of these factors have been studied by Hartman and Schmertmann (1975) using the finite-element analysis. Based on field and laboratory test results, Eden and Law (1979) show the importance of anisotropy and stress path in pressuremeter tests in sensitive clay. In order to correctly interpret the pressuremeter test, therefore, the effect of these factors has to be considered. In conducting the tests reported herein, however, the important influence of anisotropy, stress path, and rate of loading have been held constant by maintaining the same procedures in each test series.

Mechanical disturbance generated even in the self-boring process is another significant factor in the test. It has a two-fold effect. First, a softened annulus zone of soil around the pressuremeter may be produced. Baguelin *et al.* (1975) point out that such a zone will lead to a reduction of the initial modulus but an increase in shear strength if the results are interpreted based on the assumption of isotropic and homogeneous soil. In severe cases, for instance, the shear strength may be overestimated by 100%. Secondly, disturbance leading to a stress change may be caused by a difference between the sizes of the cutting shoe and the membrane when mounted on the probe. The effect of this stress change is presented in the next section.

FIG. 2. Paths followed by soil around pressuremeter during tests with different cutting shoe sizes.

Effects of Self-boring with Different Cutting Shoe Sizes

The behaviour of the clay under consideration is dependent on the strain imposed on it prior to a shear test. Let R be the ratio of the cutting shoe diameter and the membrane diameter (when mounted on the pressuremeter). At $R = 1.00$, and assuming no other source of disturbance is present, the surrounding soil experiences no change in strain and hence no change in stress condition during insertion of the pressuremeter or probe. An ideal test can then be conducted. The membranes as supplied by the manufacturer, however, were in general slightly different from that of the cutting shoe, i.e., $R \neq 1.00$.

When $R > 1.0$, a gap is created between the soil and the membrane, thus reducing the *in situ* horizontal pressure to that due to the water partially or wholly filling the borehole. This is represented by path AB in Fig. 2, where point A corresponds to the *in situ* condition. Reloading by inflating the membrane is shown by path BC where C represents the situation where the inflated membrane diameter is equal to that of the cutting shoe. The clay is now pushed back to the original position before self-boring. This position is called the absolute zero strain reference in this paper. Repeated loading on this clay will cause point C to be lower than point A. Further loading leads to point D beyond which the path will be identical to that of the ideal test.

When $R < 1.0$, the clay has to be pushed laterally to make room for the membrane, now larger than the cutting shoe. An initial load is thus introduced, bringing the pressure from point A to point E. If a waiting period is applied, the relaxation process will

FIG. 3. Shear stresses from ideal test and from test with oversized cutting shoe.

lower the pressure to point F, where inflation of the membrane begins. Since there is little information on the effect of relaxation, it is not certain what path the clay will take upon further loading. It is reasonable to assume that merging with the ideal test will occur at some point G.

As the pressure-strain curve varies with R , so does the deduced stress-strain relationship. A comparison between the test with $R > 1.0$ (test H) and the ideal test is illustrated in Fig. 3, with the corresponding shear stresses at any given strain denoted by τ_h and τ_i respectively. Beyond point D, τ_h is equal to τ_i as the two curves merge into one. Before point D, however, the slope of the curve from test H may be steeper than that of the ideal test, resulting in τ_h being higher than τ_i . One important implication is thus apparent: it is possible that the shear strength may be overestimated. Figure 4 further illustrates this point. In it the pressure is replotted against the logarithm of strain. As shown earlier, the shear stress is proportional to the slope of this curve and therefore the peak strength is given by the maximum slope. A series of three possible locations of point D is considered.

In Fig. 4a, point D is well below the point of maximum slope (point M) or the peak strength of the ideal test. The dashed portion corresponds to the part of test H before merging with the ideal test curve. This portion does not have a slope exceeding that at point M. The peak strength of this test will occur after the merging and therefore will be the same as the ideal test.

As point D approaches point M (Fig. 4b), the slope of the dashed portion increases, so much so that at

FIG. 4. Effects on shear strength from test with oversized cutting shoe: (a) merging well below M, same strength from both tests; (b) merging near M, same strength from both tests; (c) merging beyond M, higher strength from test with oversized cutting shoe.

point D it is equal to that at point M. The consequence is that peak strengths from both tests will still be equal, with test H reaching the peak strength at a smaller strain.

As point D advances further (Fig. 4c), the dashed portion gives a higher slope, resulting in a higher shear strength. This will occur at a strain either smaller or larger than the ideal test depending on the location of point D.

It is therefore obvious that the position of point D has an important influence on the deduced shear strength. For the clay being considered, the failure strain is generally small and the possibility of overestimating the strength is quite high.

Another interesting point related to the above consideration should also be noted. It concerns the method of interpretation of the ordinary pressuremeter test without self-boring. According to Baguelin *et al.* (1978), it is assumed that the start of the straight line portion of the pressure curve marks the original position of the soil prior to making the borehole. This point is regarded as representing the zero reference strain for the test. When test H is performed, it provides a check on this assumption as the absolute zero strain can be evaluated. Based again on Fig. 3,

the start of the straight line portion approximately corresponds to point B, which is below point C, the point where the soil is actually pushed back to its original position. There is therefore a shift of the reference strain, with the result that the measurement of the strength will be affected. The shear strength deductions for the two reference strains, assuming both occur at point N, are shown in Fig. 3. A higher value will be obtained using the method of interpretation by Baguelin *et al.* (1978). This will further aggravate the tendency to overestimate the strength by performing test H. The amount of overestimation will of course depend on the shift of reference strain, the initial slope of the pressure curve, and the failure strain. Tests have been conducted to gain a quantitative idea of this aspect and are reported in a later section.

The effect on the measured clay behaviour in the case of $R < 1.0$ cannot be ascertained at this point as little is known about the process of relaxation in this clay. This effect was also studied in the present test series.

Experimental Study

Description of Equipment

Because the principle of operation of the Cambridge self-boring pressuremeter is fully described by Wroth and Hughes (1973, 1974), only the essentials are reported here. The instrument probe is 80 mm in diameter and about 0.90 m long. It consists of a cutting shoe inside of which is a rotating cutter blade. Above the shoe is the membrane portion, about 0.61 m long. At the centre of the membrane portion are located the sensors for measuring strain, effective pressure, and gas (total) pressure. All sensors are bonded strain gauges, with lead wires passed up through the gas supply tube to the surface. Dry nitrogen gas is used to expand the membrane.

Some modifications to the original equipment were made so that it could be used with a Mobil B-40 hydraulic powered drill rig mounted on a truck chassis. They are described in detail by Eden and Law (1979). Pertinent to the present investigation is the removable cutting shoe made from hardened tool steel. By using different cutting shoes, the influence of its size could be studied.

Test Procedures

The test procedures generally comprised five steps: (1) measurement of membrane resistance; (2) self-boring to depth; (3) waiting for the pore pressure to stabilize; (4) loading the soil by inflating the membrane; and (5) complete unloading.

Before inserting the probe into the ground, the resistance of the membrane was measured by slowly

inflating the membrane in air and recording the pressure at different strains. For the first test, the hole was augered down to about 1.50 m above the intended depth. The probe was then lowered into the hole and allowed to self-bore at a rate dependent on the nature of the soil. The guiding factor was to obtain a homogeneous slurry without clay lumps in the cuttings washed out with water. A typical rate of 0.1 m/min was used. When the pressuremeter had bored to the desired depth the pore-water pressure, as registered by the effective pressure sensor, was allowed to come to equilibrium. The membrane was then inflated incrementally at a rate of 9.8 kPa/min, which was close to that used by Wroth and Hughes (1974). When the strain reached around 7%, unloading began at about 50 kPa/min. After the gas pressure was removed, and if the strain sensors indicated a complete return of the membrane to the pretest position, the probe was advanced to the next depth for another test. In some cases, suction existed around the probe immediately after the test and a vacuum was applied to hold back the membrane before the advance.

Test Program

Two Leda clay sites were studied. The first was an open field within the National Research Council of Canada (NRCC) grounds at Ottawa. The soil is a stiff gray clay, lightly overconsolidated with an overconsolidation ratio of about 2.2, and a sensitivity of around 50. Its natural moisture content was approximately 80% with plastic and liquid limits equal to 25 and 54% respectively. Tests were conducted using cutting shoes larger than, equal to, and smaller than the membrane. Two depths, 8.8 and 10.3 m, were chosen; tests were conducted at one after about a 1 h wait and at the other after 18 h. At each depth several tests with $R = 1.00$ were performed to study repeatability of results. Menard-type pressuremeter tests were also conducted at the same site by Schmertmann (J. Schmertmann, personal communication, 1972), which allowed a comparison to be made of the two types of tests. In the Menard pressuremeter tests, the boreholes were made with drilling mud techniques, which seemed to produce satisfactory results.

The second site was in South Gloucester, 21 km southeast of Ottawa. The subsoil at this site had been studied in detail by Bozozuk (1972). Essentially, it is a soft sensitive deposit with an overconsolidation ratio equal to 1.5, a sensitivity reaching 100, a natural moisture content of 70%, a plastic limit of 25%, and a liquid limit of 55%. Pressuremeter tests at various depths were conducted using cutting shoes equal to and smaller than the membrane.

FIG. 5. Results of pressuremeter test, NRCC site, 10.3 m depth.

Test Results and Discussion

As the sensors are located in the centre portion of the pressuremeter, the only correction required for data reduction is due to the membrane resistance. The pressure P_{mr} , in overcoming the resistance at any strain ϵ , can be adequately described by a fourth degree polynomial:

$$[5] \quad P_{mr} = \sum_{n=0}^4 a_n (\log \epsilon)^n$$

where the a 's are the coefficients of the polynomial and are to be estimated using an ordinary least squares approach.

NRCC Site

At the 8.8 m depth the tests with $R = 1.0$ show two different pressure curves, one slightly higher than the other. The corresponding pressure curves were used, therefore, in establishing the effect of R . At 10.3 m, the test results showed good repeatability.

A comparison of the pressure-strain relationship is shown in Fig. 5 for $R = 1.011$ and $R = 1.0$ at the 10.3 m depth where tests were conducted with a waiting period of 18 h. As discussed earlier, a divergence between the curves is seen in the initial portion, which is also true at 8.8 m where a 1 h waiting period was employed. The point where the linear portion of the

FIG. 6. Shear stress - strain curves from pressuremeter tests (oversized cutting shoe), NRCC site, 10.3 m depth.

curve starts can be quite clearly defined at this depth though some judgement is required at 8.8 m. The deduced shear stress curves are shown in Fig. 6. A significant difference can be seen for strain less than about 5%. Corresponding differences are also found for the initial modulus E , and the peak strength S_u , as shown in Table 1. Of particular interest are the high strengths obtained with $R = 1.011$. Based on the absolute zero reference and the reference at the start of the linear portion, the strengths are about 15 and 80% higher respectively than that of the test with $R = 1.00$. There is also a corresponding increase in the modulus, but the increase is less consistent and averages about 30%.

All these strengths are plotted in Fig. 7 and compared with those from the Menard pressuremeter tests (J. Schmertmann, personal communication, 1972). The Menard pressuremeter strength is slightly higher than the highest self-boring pressuremeter strength obtained with $R > 1.0$ and strain reference shifted from the absolute zero strain. For this type of clay, therefore, the Menard pressuremeter test will overestimate the *in situ* strength.

The foregoing experimental observation on the effect of an oversized cutting shoe is in line with the description in the earlier section. On examining the experimental results, the effect of a softened annulus zone around the probe should be recalled. This effect was studied with special reference to the present setup. It was found that for tests in sensitive clay there was an apparent error of about 2% in the shear

TABLE 1. Results of the Cambridge self-boring pressuremeter tests

Test No.	Depth (m)	Size ratio of cutting shoe to membrane R	Shear strength S_u^* (kPa)	Initial modulus $E^\dagger$ (MPa)
National Research Council of Canada site				
77-4	8.8	1.011‡	139.3	30.4
77-4	8.8	1.011§	88.3	24.1
78-2	8.8	1.00	72.6	18.8
77-8	8.8	1.00	85.3	21.4
77-11	8.8	0.995	83.9	11.2
77-5	10.3	1.011‡	135.3	21.0
77-5	10.3	1.011§	88.3	19.6
77-10	10.3	1.00	78.5	17.7
77-12	10.3	0.995	69.6	14.1
South Gloucester site				
78-G1	3.3	1.00	30.4	9.8
77-G1	3.3	0.997	30.4	8.8
78-G2	4.8	1.00	43.2	14.7
77-G2	4.8	0.997	40.7	11.9
78-G3	6.3	1.00	47.5	14.7
77-G3	6.3	0.997	35.3	12.6
78-G4	7.9	1.00	45.6	26.7
77-G4	7.9	0.997	50.0	22.0

*The shear strength refers to the maximum shear stress ever reached in the whole test.

†As the shear stress - strain relationship near the origin cannot be accurately measured, the initial modulus in this case refers to the secant modulus at the point where the relationship begins to be well defined.

‡Zero strain reference at start of linear portion.

§Absolute zero strain reference.

FIG. 7. Pressuremeter strength profiles, NRCC site.

strength. Such an error is therefore not serious when interpreting the results.

Variability may be another factor to be considered when comparing test results. The worse case is at 8.8 m where a strength variation of 17% is found in different boreholes. Variability is, however, totally eliminated when two strengths are obtained from the same test data interpreted using different zero strain references. Furthermore, variability does not account

FIG. 8. Effect of undersized cutting shoe on pressuremeter test, NRCC site, 8.8 m depth.

for the strength discrepancy of 80% reported above when the zero reference strain is taken at the start of the linear portion of the pressure curves.

Stress relief resulting from an oversized borehole in this clay is therefore the main factor in the observed high strength. This strength may exceed that from a test allowing no stress relief by up to 100% depending on how the borehole is prepared. Such being the case, the shear strength derived from the conventional pressuremeter test will be too high and will lead to unsafe designs.

The case with $R = 0.995$ at 8.8 m is compared with $R = 1.00$ in Figs. 8 and 9. The total pressure – strain curves are also shown in Fig. 8 with an indication of the initial strain caused by the undersized cutting shoe. In conforming to usual practice, this initial strain has been taken as the zero reference for obtaining shear stresses from tests with $R = 0.995$. The shear-strain curves shown in Fig. 9 reveal that the undersized cutting shoe gives rise to a reduction in shear resistance at small strain and has less effect at moderate strains. This observation holds true at the two depths where different waiting periods were employed. Other results are summarized in Table 1.

FIG. 9. Shear stress – strain curve from pressuremeter tests (undersized cutting shoe), NRCC site, 8.8 m depth.

FIG. 10. Shear stress – strain curves from pressuremeter tests, South Gloucester site.

South Gloucester Site

Test results similar to those from the NRCC site were obtained here. Figure 10 shows a comparison of the deduced shear stress – strain curves at various depths. In general, except at depth 4.8 m, an undersized cutting shoe ($R = 0.997$ in this case) again causes a reduction in shear resistance at small strain and produces little effect at high strain. At a depth of 4.8 m, where two distinct soil layers were found (Bozozuk 1972), the two curves are practically identical. This departure from the general observation may be due to the presence of two different soil layers. Other test results are also summarized in Table 1.

All the above tests with $R \leq 1.00$ yield a stress-strain curve less brittle than that observed in the laboratory (Bozozuk 1972; Law 1974). There are at least two reasons. The first reason lies again in the generation of a softened zone around the probe. This will cause the soil to reach the peak and postpeak strengths at larger strains than the undisturbed case (Baguelin *et al.* 1975). The decrease from the peak to postpeak strength will become more gradual and the brittleness less severe. A more important reason for the less brittle nature lies in the lack of drainage control during the test, which is assumed undrained. With the total radial pressure, pore pressure, and shear stress known, it is possible to compute the value of Skempton's pore-pressure parameter A (Skempton 1954). At moderate strains (about 5%) calculations show that A is close to zero or negative. This is appreciably lower than the undrained elastic response represented by $A = 0.5$ for the plane strain condition. Such low values are not consistent with current experience (Law and Bozozuk 1979) and one explanation is that drainage must have taken place at the soil-probe interface. This may be possible because of the presence of fissures and the steep pressure gradient there. The low pore pressure gives rise to a high effective pressure with the result that the soil is stressed near or within the normally consolidated range where more plastic behaviour prevails.

Summary and Conclusions

Loading and unloading prior to shearing of sensitive clay have been found to have an important influence on the shear strength and modulus obtained in the self-boring pressuremeter test. Loading or unloading can be caused by a minute difference between the sizes of the cutting shoe and the membrane. By using cutting shoes of different diameters, self-boring pressuremeter tests were conducted in both a stiff and a soft sensitive clay. The analysis of the results shows the following.

(1) An oversized cutting shoe allows the soil to unload prior to shear and this leads to a serious overestimation of the shear strength. The deduced shear strength was found to be 15 or 80%, depending on the choice of zero reference strain, higher than the strength from the ideal test with the cutting shoe and the membrane both of the same size.

(2) A similar comparison in the derived modulus shows that there is a corresponding increase that is less consistent. It amounts to about 30%.

(3) Based on a comparison with the self-boring pressuremeter tests, the Menard pressuremeter test

with the usual stress-strain relationship is found to overestimate the shear strength in stiff sensitive clay.

(4) An undersized cutting shoe imposes an initial load on the soil prior to shear, resulting in a reduction in shear resistance at small strains.

(5) Within the range studied all the stress-strain curves beyond a moderate strain (5%) show little dependence on the size of the cutting shoe.

(6) The use of different waiting periods (1 and 18 h) did not change the foregoing observations.

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