Interpretation Problems Related to Self-Boring Pressuremeter Tests in Sensitive Clays

Problèmes d'interprétation reliés aux essais au pressiomètre autoforeur dans les argiles sensibles

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ABSTRACT

Theoretical analyses of the self-boring pressuremeer test (SBPMT) in clay have developed to the point where in principle it is possible to determine soil parameters like shear modulus, undrained shear strength, and in situ horizontal geostatic stress, without resorting to empirical relationships. However, it has also become clear that factors such as initial installation disturbance and strain rate effects, for example, still occur and affect the initial part or more of the pressure-strain expansion curve.

These factors are even more accentuated in the overconsolidated sensitive clays of Eastern Canada which are known for their strain softening response as observed in laboratory triaxial tests on undisturbed specimens. However, their true strain softening behaviour has practically never been observed in stress-strain curves derived from experimental pressuremeter expansion curves.

The present paper reports on the analysis of SBPMTs carried in two sensitive clays of Quebec and shows that none of the stress-strain curves derived from the pressure-strain expansion relationships using either the classical pressure versus the logarithm of the strain procedure or the differentiation technique was characterized by a true undrained peak shear strength followed by strain softening. Most of the tests showed a nonlinear pseudo-elastic perfectly plastic behaviour.

RESUME

Les analyses théoriques de l'essai au pressiomètre autoforeur (SBPMT) dans l'argile ont évolué au point qu'il est en principe possible de déterminer les paramètres du sol comme le module de cisaillement, la résistance au cisaillement non drainé et la contrainte géostatique horizontale in situ, sans recourir à des relations empiriques. Cependant, il est également devenu évident que des facteurs tels que le remaniement initial dû à l'installation et les effets du taux de déformation, par exemple, se produisent toujours et affectent la partie initiale ou une plus grande partie de la courbe d'expansion pression-déformation.

Ces facteurs sont encore plus accentués dans les argiles sensibles surconsolidées de l'Est du Canada, qui sont connues pour leur réponse de radoucissement par déformation telle qu'observée dans les essais triaxiaux en laboratoire sur des spécimens non remaniés. Cependant, leur véritable comportement de ramollissement n'a pratiquement jamais été observé dans les courbes contrainte-déformation dérivées des courbes expérimentales d'expansion pressiométriques.

Le présent article présente des analyses des SBPMTs effectués dans deux argiles sensibles du Québec et montre qu'aucune des courbes contrainte-déformation dérivées des relations d'expansion pression-déformation utilisant soit la procédure classique pression versus logarithme de la déformation ou la technique de différenciation n'a été caractérisée par une vraie résistance au cisaillement non drainé maximale suivie d'un ramollissement. La plupart des tests ont montré un comportement pseudo-élastique non linéaire parfaitement plastique.

Keywords: Strain softening, sensitive clays, self-boring pressuremeter tests, stress-strain curves.

1. Introduction

The self- boring pressuremeter test (SBPMT) is unique among in situ soil tests in that it allows the derivation of the complete stress-strain curve of clay from the experimental pressure-expansion curve. The interpretation of SBPMTs in clay is generally carried out using a total stress approach by means of either a classical ideally elastic-perfectly plastic (Cassan 1960; Gibson and Anderson 1961; Windle and Wroth 1977) or a non-linear plastic analysis (Baguelin et al. 1972; Ladanyi 1972; Palmer 1972). In addition, Denby and Clough (1980) and Ferreira and Robertson (1992) proposed a hyperbolic strain-hardening model for the stress-strain curve. Pye (1995) presented a comparison of the elastic-perfectly plastic and hyperbolic theories for interpretation of loading-unloading branches of the SBPMT and showed that a small systematic difference existed between the two models for the undrained shear strength of soft clay.

As a consequence of the models used for the analysis of SBPMTs in clay, only the non-linear analysis of Baguelin et al. (1972), Ladanyi (1972), and Palmer (1972) is able to show a strain-softening response if the clay is really characterized by such behaviour. Thus, as the overconsolidated sensitive clays of Eastern Canada are known for their strong strain-softening responses as shown by conventional undrained triaxial tests on undisturbed specimens, then such behaviour should be apparent from the interpretation of SBPMTs in these clays. For example, Aubeny et al. (2000) found that the stress-strain curves of Boston Clay derived from SBPMTs, using Palmer' approach, were indeed characterized by extremely high peak strengths at very small strain followed by strain softening. However, such response, which was attributed to initial disturbance, was not considered as being an intrinsic property of the clay. A similar conclusion was reached by Eden and Law (1980) and Law and Eden (1982) concerning self-boring pressuremeter tests carried out in two sensitive clays of Eastern Canada.

Many factors exert an influence on the stress-strain behaviour of the soil obtained from the pressuremeter test. Based on field and laboratory test results, Eden and Law (1980) showed 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 prior to the performance of an expansion test is a significant factor in the interpretation of the test results. 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, as shown by Law and Eden (1980, 1982) in self-boring pressuremeter tests carried out in two sensitive clays of the Ottawa region. In addition, Prevost (1976) demonstrated theoretically how to recover the true stress-strain curve from a pressure-expansion relationship affected by either overcutting or overpushing (See also Silvestri 2004).

Several investigations have been carried out in the sensitive clays of Eastern Canada by means of SBPMTs (See, for example, Law and Eden 1982; Hammouche 1995). Law and Eden (1982) illustrated the dramatic effect of the diameter of the cutting shoe on the derived values of the lift-off pressure, the undrained shear strength, and the shear modulus. Hammouche (1995) carried out self-boring pressuremeter tests, hydraulic fracture tests, dilatometer tests, and vane shear tests in a lightly overconsolidated clay, and showed that undrained shear strengths deduced from the pressuremeter tests were about 40% higher than vane-derived values. Detailed analyses of SBPT results obtained by Hammouche (1995) and comparison with other in situ tests may be found in a number of publications (see, for instance, Silvestri 2018, Silvestri and Tabib 2013, 2015, 2024). Pelletier (2004) performed Ménard-type pressuremeter expansion/contraction tests in a medium stiff sensitive clay and showed that undrained shear strengths derived using an ideally elastic-perfectly plastic soil model were overestimated by about 100% compared to values deduced from self-boring pressuremeter tests. Silvestri and Abou-Samra (2008) attributed the overestimation of the undrained shear strengths found by Pelletier (2004) was the result of unloading and remoulding of the boreholes prior to the performance of prebored pressuremeter tests.

The present paper shows that shear stress-shear strain curves derived from self-boring pressuremeter tests in strain-softening sensitive clays rarely show a strain-softening response, unless the borehole cavity is overbored or disturbed prior to the performance of the expansion test.

2. Theory

2.1. Pressuremeter Relationships in Undrained Clay

By considering plane strain and undrained conditions, it has been found that the total horizontal pressure p_o applied by the pressuremeter membrane during the expansion test is given by (Cassan 1960; Gibson and Anderson 1961; Baguelin et al. 1972; Ladanyi 1972; Palmer 1972)

$$p = p_o + \int \tau \frac{d\varepsilon}{\varepsilon} = f(\varepsilon_o) + p_o \quad (1)$$

where ε_o is the radial (or tangential) strain induced at the wall of the cavity, τ is the shear stress generated in the soil, ε is the radial strain corresponding to τ, and p_o is the initial total horizontal pressure acting on the pressuremeter membrane, prior to the performance of the expansion test. Eq. (1) is based on the assumption that the clay remains homogeneous during the expansion and that the analysis is carried out in terms of total stresses. Eq. (1) is valid provided radial strains are small (i.e., $\varepsilon_o < 10\% - 15\%$). Differentiation of Eq. (1) with respect to ε_o leads to the shear stress-radial strain curve of the clay:

$$\tau = \varepsilon_o f'(\varepsilon_o) = \varepsilon_o \frac{dp}{d\varepsilon_o} \tag{2}$$

Where $f'(\varepsilon_o) = df/d\varepsilon_o$ represents the slope of the experimental pressure-expansion relationship, $dp/d\varepsilon_o$. In a pressuremeter test, $(p - p_o)$ in Eq. (1) is a function of ε_o and Eq. (2) thus theoretically allows the unknown function τ to be determined from the experimental expansion curve. It must be recalled that the foregoing analysis remains valid only if the soil around the pressuremeter was undisturbed by the installation procedure, such that $p = p_o = \sigma_{ho}$, where σ_{ho} is the existing horizontal pressure for $\varepsilon_o = 0$ at the start of the test.

When initial disturbances are present, reliable results may nevertheless be obtained, as shown either by Prévost (1976) by means of the hysteretic Masing model or Jefferies (1988) using an image-matching technique. Prévost (1976) indicated, for example, that if the borehole is overcut or overbored by a radial strain ε_{ob}, then (i) the apparent shear stress-strain curve which is found from application of Eq. (2), exhibits a discontinuity (or a peak) at $\varepsilon_o^* = -2\varepsilon_{ob}$, where $\varepsilon_o^* = \varepsilon_o - \varepsilon_{ob}$ represents the radial strain measured from the beginning of the test, and (ii) the apparent shear resistance τ^* for $\varepsilon_o^* \leq -2\varepsilon_{ob}$ is equal to twice the value of the true shear resistance τ for $\varepsilon_o = \varepsilon_o^*/2$. It should be also noted that the position of the discontinuity on the apparent stress-strain curve allows determination of ε_{ob} and then the true stress-strain curve of the clay. Prévost (1976) also showed that initial overpushing yields an apparent shear stress-strain curve that is lower than the true stress-strain curve. As a consequence, only the presence of a softened annulus of clay and overcutting will cause an overestimation of the undrained shear strength. In addition, overcutting causes the appearance of a peak strength, followed by strain softening, even if the stress-strain curve is of the strain-hardening type (Baguelin et al. 1975; Prévost 1976).

2.2. Expansion Curve of True Strain-Softening Material

Consider the shear stress-shear strain curve of a hypothetical strain-softening clay shown in Fig.1. The relationship is given by (Prévost and Hoeg 1975; Ladd et al. 1979; Prapaharan et al. 1990):

$$\tau = A\varepsilon\,(B\varepsilon_o + 1)/(C\varepsilon_o^2 + 1) \tag{3}$$

where ε is expressed as a percentage and A, B, and C are material constants. These constants are such that (i) the peak shear strength S_u occurs at a radial strain $\varepsilon = \varepsilon_p = \left[B + (B^2 + C)^{\frac{1}{2}}\right]/C$, (ii) the slope at zero strain is A, and (iii) the residual shear resistance $S_{u\,resid}$ occurs at $\varepsilon = \infty$ and is equal to AB/C.

The corresponding pressure-expansion curve is given by:

$$\varepsilon_p = p_o + \left[AB/\left(2(3C)^{\frac{1}{2}}\right)\right] ln(1 + C\varepsilon^2) + \left[A/(3C)^{\frac{1}{2}}\right] tan^{-1}\left(\varepsilon C^{\frac{1}{2}}\right) \tag{4}$$

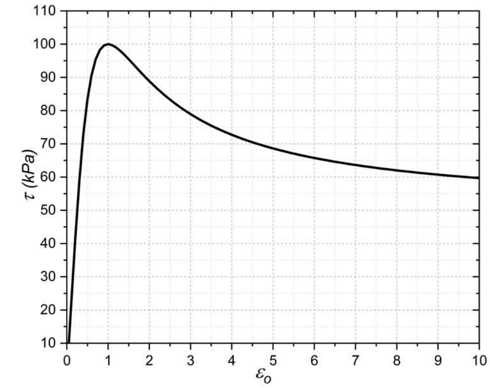

Figure 1. Shear stress-radial strain curve of strain-softening clay.

For illustration purposes, let $A = 200$, $B = 0.5$, and $C = 2$. These values correspond to S_u=100 kPa at $\varepsilon_p = 1.0\%$ and $S_{u\,resid} = 50\,kPa$. The pressure-expansion curve is illustrated in Fig. 2. Indeed, as shown by Prévost (1976), the location of ε_p for the peak shear strength on the shear stress-radial strain relationship should correspond to an inflection point on the pressure-expansion curve, which is difficult to observe in Fig. 2.

This example illustrates the difficulty in deriving stress-strain curves from pressure-expansion curves that show an intrinsic strain-softening response. The difficulty lies in the differentiation procedure of the experimental pressure-expansion curves that is used to obtain the stress-strain relationships. Indeed, it is much easier to obtain pressure-expansion curves by integration of pre-determined stress-strain curves.

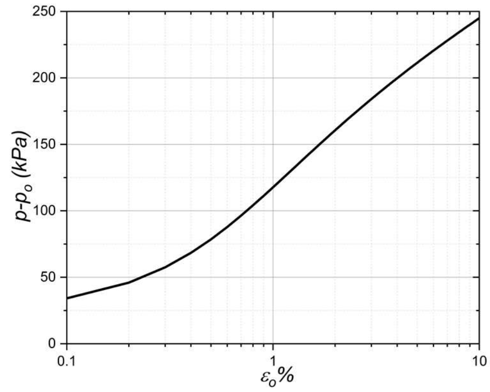

Figure 2. Pressure-expansion curve of strain-softening clay.

3. FIELD TESTS

Pressuremeter tests (SBPMTs) were carried out by means of a Cambridge self-boring instrument, Mark VIII model. Expansion tests were performed at a pressure rate of 18 kPa/min resulting in an average radial strain rate of approximately $1\%/min$. Two pore pressure gauges fixed to the flexible membrane allowed measurement of the pore water pressures generated both prior and during the expansion tests. Vane shear tests (VSTs) were carried out using a Nilcon vane (diameter $D = 65\ mm$, height $H = 130\ mm$).

3.1. Lightly Overconsolidated Sensitive Clay

Ten self-boring pressuremeter tests (L-5-1 to L-5-3, L-6-1 to L-6-2, L-7-1 to L-7-2, and L-7-4 to L-7-6; see also Table 1) were performed at the experimental site of Louiseville (Quebec), a town located 125 km northeast of Montreal, along Highway 40 on the north shore of the St. Lawrence River, as Reported by Hammouche (1995) and Silvestri (2003). This site has been studied over the past 40 years by research teams from Laval University and Polytechnique Montreal. The soil profile at Louiseville consists of a 60m thick deposit of lightly overconsolidated sensitive Champlain Sea clay. In the depth interval between 2 and $14m$, the natural moisture content decreases from 90% at $2m$ to 65% at $14m$, and the liquidity index from 1.6 at $2m$ to 1.1 at $14m$. The field undrained shear strength S_u varies linearly with depth, from 20 kPa at $2m$ to 55 kPa at $14m$. The clay is overconsolidated and the overconsolidation ratio (OCR) ranges from 5.6 at $2m$ to 2.4 at $14m$.

Typical pressure-expansion curves obtained are reported in Fig. 3, following the procedure of Windle and Wroth (1977). Examination of the curves shows essentially that all the relationships are linear in the plastic phase of the expansion.

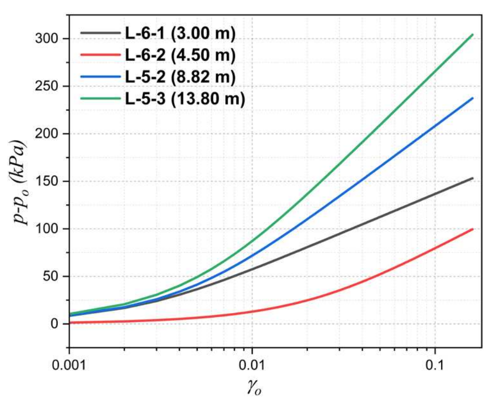

Figure 3. SBPT pressure-expansion curves of lightly overconsolidated clay at Louiseville.

Application of Palmer's (Palmer 1972) approach allowed the determination of the corresponding shear stress-shear strain curves shown in Fig. 4. Please recall that the shear strain γ equals twice the radial strain ε.

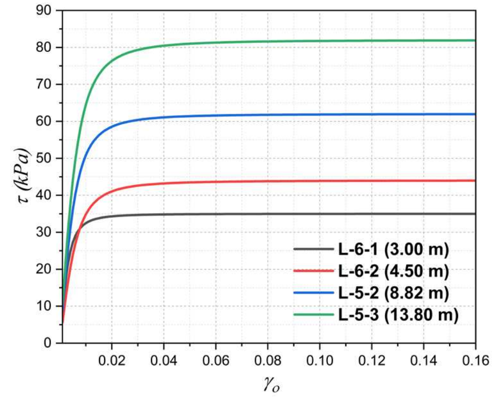

Figure 4. SBPT-derived shear stress-shear strain curves of lightly overconsolidated clay at Mascouche.

Table 1 presents a comparison between the undrained shear strength and the shear modulus derived by means of the ideally elastic-perfectly plastic model and Palmer's approach (i.e., Eq. (2)). Regarding the undrained shear strength, both approaches are in good agreement. For the shear modulus, Palmer's approach allows finding the maximum value whereas the elastic-plastic approach gives the value of the shear modulus at the onset of the plastic response.

Examination of all the curves reported in Fig. 3 indicates that none of the relationships are characterized by an inflection point. As a result, all the stress-strain curves obtained by means of Palmer's approach (i.e., Eq. (2)) are of the nonlinear strain hardening type, as shown in Fig. 4.

As for the undrained shear strengths determined with the vane tests, it was found that the ratio $S_{u\ SBPMT}/S_{u\ Vane}$ was equal to an average value of 1.4.

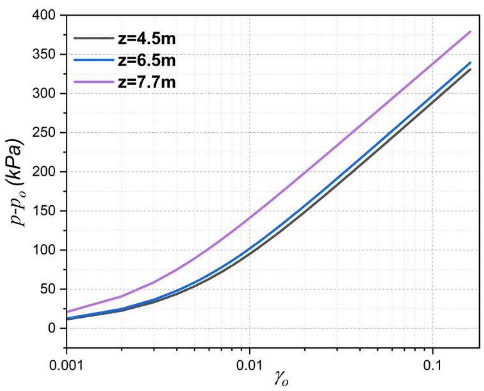

Figure 5. SBPT pressure-expansion curves of stiff overconsolidated clay at Mascouche.

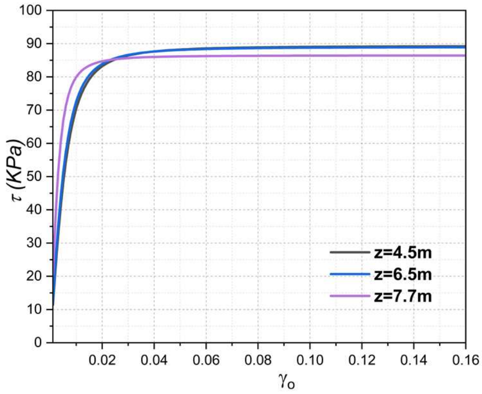

Figure 6. SBPT-derived shear stress-shear strain curves of stiff overconsolidated clay at Mascouche.

3.2. Stiff Overconsolideated Clay

Three self-boring pressuremeter tests were carried out by Hammouche (1995) using the same Cambridge, Mark VIII instrument at an experimental site located 25 km north of Montreal, along Highway 25, near the town of Mascouche (Quebec). The soil deposit is characterized by a $2m$ thick crust of fissured clay which is followed by a $7m$ thick layer of stiff sensitive clay, in which the pressuremeter tests were completed. The natural moisture content and liquid limit of the $7m$ thick layer of clay are practically constant at 65% and the plasticity index averages 37% The vane undrained shear strength increases from $60\ kPa$ at $3m$ to $100\ kPa$ at $9\ m$. The overconsolidation ratio (OCR) varies from 5.1 to 6.4.

Data obtained from the three SBPMTs are reported in Fig. 5 and Table 2. The corresponding shear stress-shear strain curves are shown in Fig. 6. Using the Tresca model (Windle and Wroth 1977), the undrained shear strength S_u and the shear modulus G at 4.5 m are equal to 90 and $8190\ kPa$, respectively. Concerning the tests performed at 6.5 and 7.7 m, the curves shown in Fig. 5 indicate that the corresponding values are $90\ kPa$ for S_u, and 8190 and $14430\ kPa$ for G, respectively. As for Palme's approach, the undrained shear strength varies between 87 and $89\ kPa$, whereas the maximum value of the shear modulus ranges from 12580 to $21050\ kPa$. As indicated by the curves reported in Fig. 5, the relationships are again essentially linear during the plastic response of the clay. As a consequence, it becomes quite difficult, if not impossible, to detect the presence od an inflection point, which would indicate that the stress-strain curve of the clay is characterized by a peak strength followed by strain softening. Again, the stress-strain curves reported in Fig. 6 are typical of a strain-hardening material.

Finally, comparison between the values of the undrained shear strengths obtained from the SBPMTs and those deduced from the field vane tests (VSTs) indicates that deduced undrained shear strength are between -10% and +24% of the results of the vane shear tests. Thus, for this clay, VST- and SBPMT-deduced undrained shear strengths appear to be quite similar.

Table 1. Comparison of SBPMT results at Louiseville (Lightly overconsolidated clay)

Test No.	**Depth (m)**	**σ_{ho} (kPa)**	**Windle and Wroth (1977)**		**Palmer's approach**	
			S_u (kPa)	G (kPa)	S_u (kPa)	G_{max} (kPa)
L-5-1	5.82	111	46	5795	46	8695
L-5-2	8.82	140	55	5810	62	12000
L-5-3	13.80	245	82	8200	82	13160
L-6-1	3.00	52	34	7920	35	12230
L-6-2	4.50	98	44	4400	44	10000
L-7-1	3.00	61	37	3740	37	5740
L-7-2	4.50	99	52	2910	52	4570
L-7-4	6.00	110	51	5660	51	8600
L-7-5	9.40	145	63	9260	63	14210
L-7-6	14.00	235	63	9260	63	14980

Table 2. Comparison of SBPMT results at Mascouche (Stiff overconsolidated clay)

Depth z (m)	σ_{ho} (kPa)	S_u $Vane$ (kPa)	Windle and Wroth (1977)		Palmer's approach	
			S_u (kPa)	G (kPa)	S_u (kPa)	G_{max} (kPa)
$4.5m$	195	72	90	8190	89	12840
$6.5m$	275	80	90	8190	89	12580
$7.7m$	310	103	90	14430	87	21050

4. DISCUSSION

As shown by the pressure-expansion curves reported in Figs. 3 and 5, the absence of inflection points is an indication that the stress-strain curves of both the lightly and the stiff overconsolidated clays are of the nonlinear strain-hardening type. However, when these clays are tested in standard undrained triaxial tests, the resulting stress-strain curves are always characterized by peak strengths followed by strain softening as shown, for example, by Silvestri and Abou-Samra (2008, 2017). Thus, one would expect that the stress-strain curves deduced from self-boring pressuremeter tests would show the same response. The reason for the absence of inflection points in the pressure-expansion curves is two-fold. First, the stress paths which are followed by the clay during the expansion process turn to the right toward an increase in the mean effective stress on a Mohr-Coulomb diagram, whereas the stress paths which are followed in standard undrained triaxial tests turn to the left toward the origin, with a corresponding decrease in the deviatoric stress, as shown, for instance, by Law and Eden (1980) and Silvestri and Abou-Samra (2008). Second, when the peak undrained shear strength is reached in a standard undrained triaxial test, the specimen loses its integrity due to the development of planes of weakness. The specimen may also split in several pieces. However, such loss of integrity cannot develop during pressuremeter testing because the clay in contact with the probe is always confined by the surrounding material. As a consequence, the resulting stress-strain curves derived from pressuremeter tests are always less strain-softening than those obtained from standard undrained triaxial tests, as pointed out several years ago by Eden and Law (1980). In addition, stress-strain curves derived from self-boring pressuremeter tests and triaxial tests are found to be different since stress and strain paths, as well as boundary conditions are different.

5. CONCLUSIONS

When self-boring pressuremeter expansion curves are analyzed in terms of total stresses, it is often found that the deduced stress-strain curves are characterized by various degrees of scattering which can result in extreme values of undrained shear strengths at very small strains. This is caused by the necessity of obtaining by differentiation the slope of the experimental pressure-expansion curve. When an experimental pressure-expansion curve is differentiated point by point, even the slightest scatter in the experimental points may lead to unreasonable shear strength values. The adoption of the approach suggested by Windle and Wroth (1977) in which the applied pressure is plotted as a function of the logarithm of the radial or tangential strain reduces the scatter in the resulting stress-strain curve. However, it also decreases the possibility of finding stress-strain curves characterized by strain softening, as shown in the paper. A better approach would be to fit a polynomial to the experimental expansion pressure-radial strain relationship and then to apply Palmer's technique to obtain the stress-strain curve.

This procedure which was applied to the self-boring pressuremeter tests carried out in two deposits of sensitive clay of Quebec allowed the computation of the undrained shear strengths and the shear moduli of the soils. The shapes of the pressure-expansion curves appear to indicate that initial disturbance was not an important factor.

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