Geotechnical and Geophysical Site Characterisation 5 – Lehane, Acosta-Martínez & Kelly (Eds)
© 2016 Australian Geomechanics Society, Sydney, Australia, ISBN 978-0-9946261-1-0

Analysis of instrumented sharp cone tests performed in a sensitive clay of Quebec

V. Silvestri
Ecole Polytechnique, Montreal , Quebec, Canada. E-mail : vincenzo.silvestri@polymtl.ca
C. Tabib
Consultant, Montreal, Quebec, Canada

ABSTRACT: The objective of this paper is to compare field test results obtained in a stiff sensitive clay of Quebec using prebored sharp cone tests (ISCTs) with data obtained from prebored and self-bored pressuremeter tests (PMTs and SBPMTs), and vane shear tests (VSTs). Results are interpreted using both the elastic-plastic Tresca model and Palmer's approach. It is shown that while ISCTs and PMTs greatly overestimate the undrained shear strength compared to the SBPMTs and VSTs, the opposite trend is found for the shear modulus. It is thought that such behaviour is due to remoulding and unloading of the clay caused by drilling of the pilot holes for the ISCTs and PMTs. In addition, it has been found that the in situ coefficient of lateral pressure at rest, K_o, deduced from the SBPMTs is abnormally high. The primary cause has been attributed to the cemented nature of the clay.

1. INTRODUCTION

The most useful field tests in clay are the vane shear test (VST), the cone and the piezocone penetration tests (CPT and CPTu), the prebored and self-bored pressuremeter tests (PMT and SBPMT), and the flat dilatometer test (DMT). Although in each of these tests the soil undergoes different stress and strain paths, it is possible, by means of appropriate interpretation methods, to deduce soil parameters for design purposes.

The instrumented sharp cone test (ISCT) is a recent addition to the arsenal of in situ testing methods. The test consists in driving an instrumented low-angle truncated sharp cone (ISC) at a constant penetration rate in a prebored vertical pilot hole or a self-bored hole. The purpose of the test is to produce in the soil a quasi-cylindrical cavity expansion similar to that in a PMT or a SBPMT. The ISCT is thus intended to provide soil parameters like those obtained in a PMT or a SBPMT, but in the continuous manner of the CTP or the CPTu. Soil and water pressures generated by the steady penetration of the probe are measured by means of pressure transducers mounted on the wall of the cone. On account of the small apical angles (i.e., 1°-2°) of the ISC, pressuremeter interpretation methods are currently employed to analyse ISCT data for the determination of soil properties.

The objective of the present paper is to compare field test results obtained in a stiff sensitive clay of eastern Canada by means of prebored ISCTs with results deduced from PMTs, SBPMTs, and VSTs. It is shown that while both PMTs and ISCTs yield very similar results, they nonetheless lead to undrained shear strengths that are much higher than those deduced from SBPMTs and VSTs. The opposite trend is found to occur with the values of the shear modulus. Reasons are given for the observed response.

2. FIELD TEST RESULTS

ISCT

The instrumented sharp cone (ISC) used in the present investigation is shown in Fig.1. The device is equipped with five total pressure transducers installed at the positions indicated in the same figure. Because a plane strain undrained cylindrical cavity expansion is considered to take place during the steady penetration of the device, the total pressure recorded by each transducer corresponds to a volumetric strain $(\Delta V/V)_i$, which is defined as follows (Ladanyi and Longtin 2005):

$$\left(\frac{\Delta V}{V}\right)_i = \frac{V_i - V_o}{V_i} = \frac{{r_i}^2 - {r_o}^2}{{r_i}^2} = 1 - \left(\frac{r_o}{r_i}\right)^2 \qquad (1)$$

where V_i, V_o = expanded and initial volumes of the pilot hole. If there are n lateral pressure transducers mounted on the ISC, there will be n points on the equivalent pressuremeter expansion curve, or n points, including the origin, on the deduced stress-strain curve. The volumetric strains which correspond to the positions of the pressure transducers range between 3 and 30%. Volumetric strains are equal to shear strains generated in the horizontal plane surrounding the ISC.

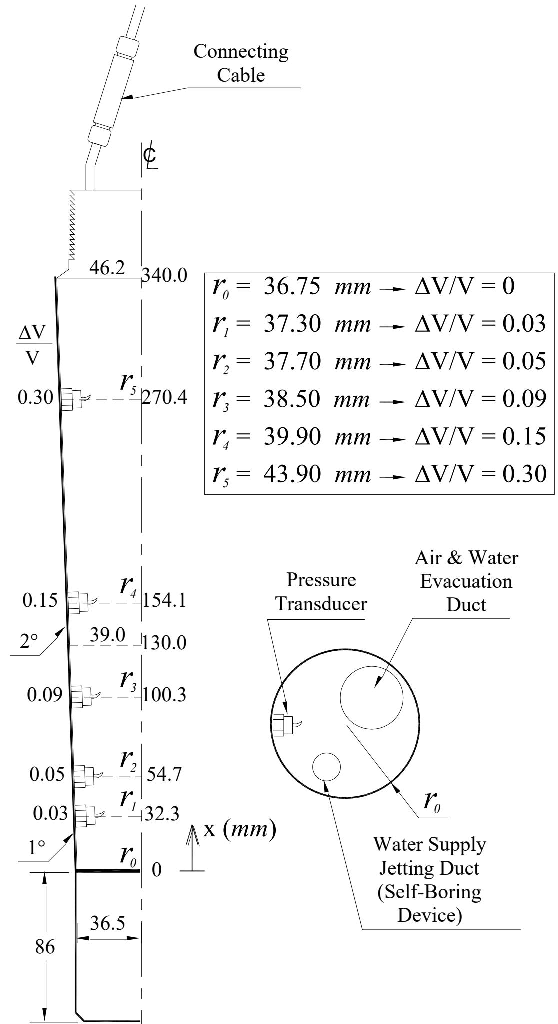

Fig. 1: Instrumented sharp cone. r_0, r_1, r_2, r_3, r_4, r_5, borehole radii; x, distance from the tip of the cone to the centre of a pressure gauge; $\Delta V/V$, current volumetric strain (Adapted from Longtin 2004).

ISCTs were carried out in 73 mm diameter vertical pilot holes which were bored by using the technique of retrieving continuous samples of clay by means of 73 mm diameter, 710 mm long Shelby tubes. ISCTs were performed immediately after completion of the pilot holes. The ISC was driven at a constant vertical penetration rate of 20 cm/min, which corresponds to the strain rate produced in a pressuremeter test in clay.

PMTs and SBPMTs

PMTs were carried out in pilot holes which were bored using the technique employed for the ISCTs. The tests were conducted with a 70 mm diameter, Texam NX probe (Felio and Briaud 1986) of length-to-diameter ratio, *L/D*, of 5.1, which was expanded at a rate of 1%/min. SBPMTs were performed using a Mark VIII, Cambridge In situ, instrument, equipped with two pore pressure gauges, and expanded at a rate of 1.09%/min.

3. INTERPRETATION METHODS

PMTs, SBPMT, and ISCT results were first analysed by assuming that the clay obeyed an ideally elastic perfectly plastic (Tresca) soil model (Gibson and Anderson 1961). For small strains, the constitutive relations are:

$$\tau = \begin{cases} \gamma G & , \quad \gamma \le \dfrac{S_u}{G} \\ S_u & , \quad \gamma > \dfrac{S_u}{G} \end{cases} \qquad (2)$$

where τ = shear stress, γ = shear strain, G = shear modulus, and S_u = undrained shear strength. The corresponding radial stress-shear strain relationships are the following:

$$\sigma_r = p = \begin{cases} p_o + \gamma G & , \quad \gamma \le \dfrac{S_u}{G} \\ p_o + S_u\left(1 + \ln\left(\dfrac{G\gamma}{S_u}\right)\right) & , \quad \gamma > \dfrac{S_u}{G} \end{cases} \qquad (3)$$

where $\sigma_r = p$ = applied or generated radial pressure, $p_o = \sigma_{ho}$ = initial horizontal geostatic stress, and G/S_u = rigidity index. Eq. 3 indicates that if the pressure p is drawn as a function of $\ln\gamma$, the undrained shear strength S_u equals the constant slope of the resulting curve in the plastic phase of the expansion (See, also, Whittle and Wroth 1977). In addition, the shear modulus G may be found from the value of the shear strain γ recorded at the end of the elastic phase of the expansion, because $G = (p - p_o)/\gamma$.

PMTs and SBPMT data were also interpreted using Palmer's approach (Palmer 1972), which allows the determination of the shear stress τ from the pressure-expansion curve, by means of the following expression:

$$\tau = \gamma \frac{dp}{d\gamma} \tag{4}$$

where $dp/d\gamma$ represents the slope of the experimental curve. Eq. 4 is only valid for small strains.

4. SOIL DEPOSIT

PMTs, ISCTs, and VSTs were completed at an experimental site located 25 km northeast of Montreal in Quebec. The soil stratigraphy is shown in Fig. 2. It consists of a 2.2 m thick crust of oxidized clay which overlies a deposit of firm to medium stiff sensitive clay of high plasticity. Below the crust, the moisture content is about 70% and the plasticity index averages 40%. The overconsolidation ratio, *OCR*, ranges between 4 and 7. The undrained shear strength, S_u, measured with a Nilcon vane, increases from 50 kPa at 2 m to just over 100 kPa at 6 m.

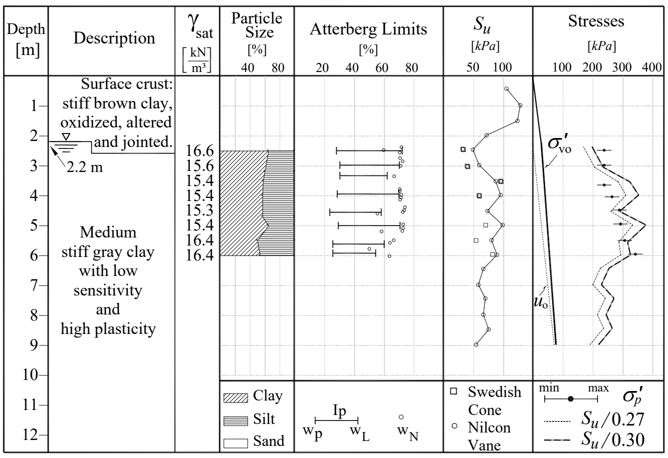

Fig. 2: Soil stratigraphy at Mascouche. S_u, undrained shear strength; w_L, liquid limit; w_N, natural water content; w_P, plastic limit; γ_{sat}, saturated unit weight; σ'_p, vertical preconsolidation pressure.

While most ISCTs were carried out in the depth interval from 0.5 to 6 m, PMTs were completed at three distinct depths of 2.5, 4.5, and 6.0 m. Typical PMT and ISCT results are shown in Figs. 3 and 4. The ISCT data reported in the latter figure clearly indicate the existence of a softer clay layer between 2.1 and 2.7 m depth. Palmer's approach was not used to interpret ISCT results because of the small number of experimental points on the expansion curve.

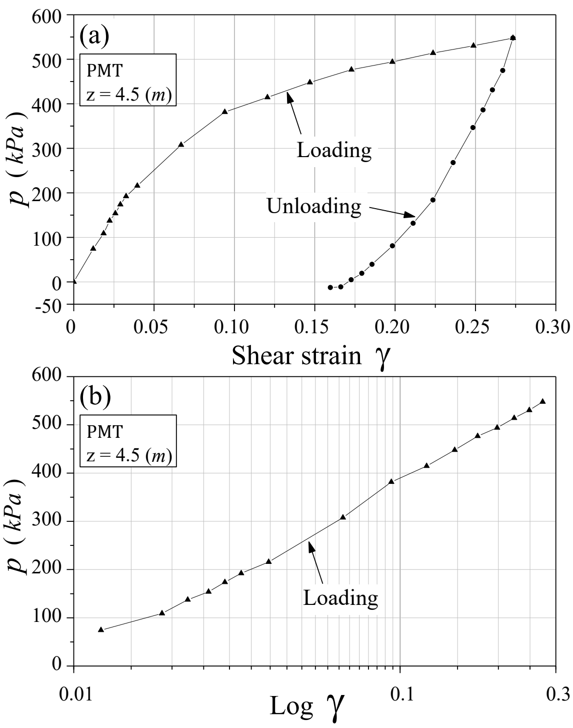

Fig. 3: a) Applied pressure versus shear strain, b) Applied pressure (loading) versus natural logarithm of shear strain.

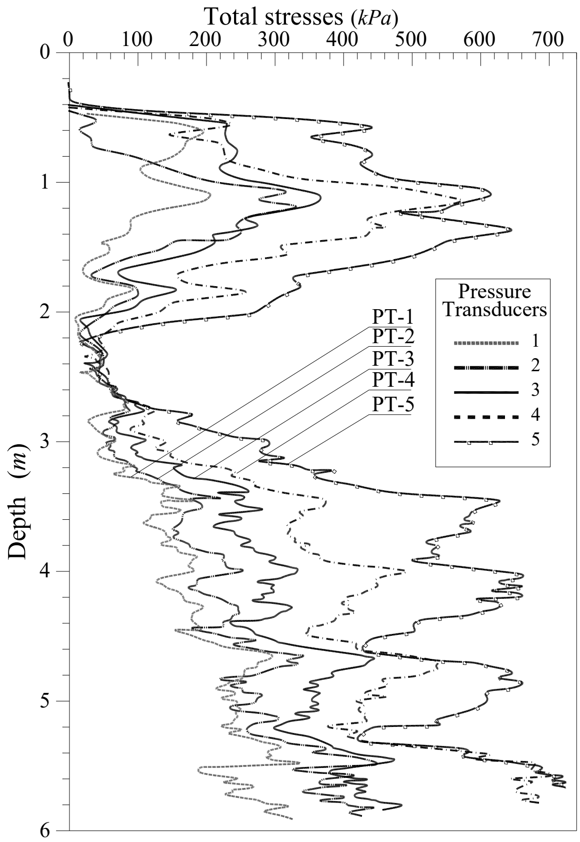

Fig. 4: ISCT profile at Mascouche (adapted from Longtin 2004)

SBPMTs, CPTus, and VSTs were performed at a nearby site by a team from Laval University of

Quebec City. Typical SBPMT and CPTu data are reported in Figs. 5 and 6. Comparison between the CPTu and VST results showed that the cone factor N_{kt} varies between 14.3 and 12.8 in the depth interval from 4.5 to 7.9 m. The cone factor $N_{kt} = (q_t - \sigma_{vo})/S_{uVST}$, where q_t = measured cone tip resistance, σ_{vo}= total vertical stress, and S_{uVST}= undrained shear strength by vane shear test.

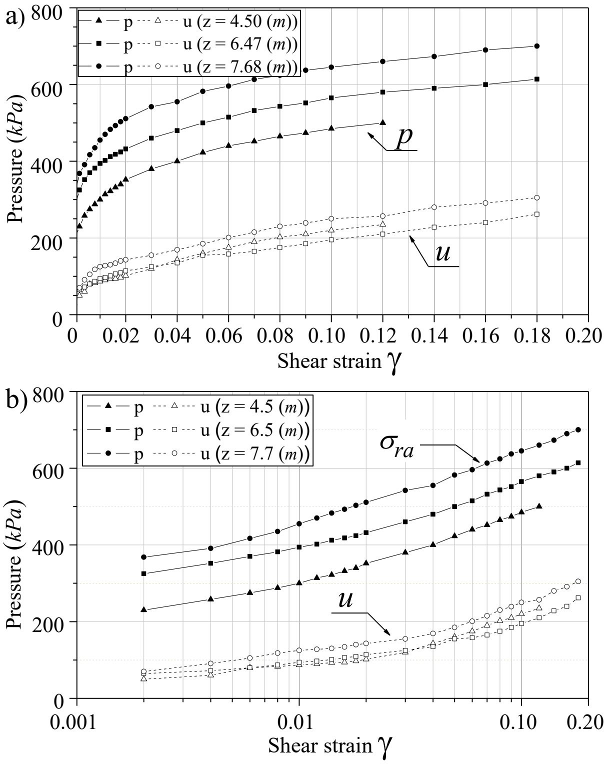

Fig. 5: SBPMT expansion tests. p, applied pressure; u, pore pressure; z, depth.

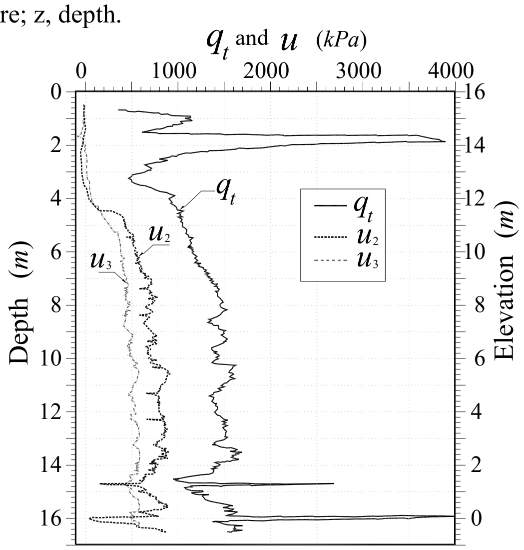

Fig. 6: CPTU data at Mascouche B. q_t, cone resistance; u, pore pressure; u_2, pore pressure above the cone tip; u_3, pore pressure at five diameters above the shoulder of the piezocone shaft (adapted from D. Demers, personal communication, 2001)

5. DISCUSSION OF TEST RESULTS

In situ Coefficient of Lateral Earth Pressure at Rest

Table 1 presents the initial stress states in the clay deposit, as deduced from SBPMTs. The horizontal geostatic stress, p_o or σ_{ho}, was found from the average lift-off pressure registered by the three feeler arms of the pressuremeter. The initial pore water pressure, u_o, was measured in standard Geonor piezometers. The in situ coefficient of lateral earth at rest, K_o, was deduced from the ratio $\sigma'_{ho}/\sigma'_{vo}$ or $(\sigma_{ho} - u_o)/(\sigma_{vo} - u_o)$, where σ'_{ho}, σ'_{vo} = effective lateral and vertical pressures, and σ_{vo} = calculated total vertical overburden pressure. K_o values which vary between 3.9 at 4.50 m and 3.6 at 6.5 m, are much higher than the corresponding ones obtained from application of the well-known relationship:

$$K_o = (1 - \sin\phi'_{nc})\, OCR^{\sin\phi'_{nc}} \quad (5)$$

where ϕ'_{nc} = friction angle of destructured clay. For the sensitive clay at study, ϕ'_{nc} = 33°. High lateral pressures similar to those deduced from the SBPMTs were also obtained from hydraulic fracture tests and flat dilatometers tests carried out at the same site, as reported by Hamouche at al. (1995). According to these investigators, the causes of the very high values of K_o are related primarily to the cemented nature of the sensitive clay and unloading due to past erosion, and to a lesser extent to delayed consolidation.

Table 1 Initial stress states of clay deposit

Depth (m)	u_o (kPa)	σ'_{ho} (kPa)	σ'_{vo} (kPa)	OCR	K_o $\sigma'_{ho}/\sigma'_{vo}$	K_o Eq.5
4.5	30	165	42	6.4	3.9	1.25
6.5	44	232	60	5.1	3.9	1.11
7.7	55	255	70	5.9	3.6	1.20

Undrained Shear Strength and Shear Modulus

PMTs were carried out first by loading the soil to a maximum predetermined radial strain of 10%, and then by unloading to zero radial pressure. However, because only loading phases were completed in the SBPMTs and necessarily so in the ISCTs, the discussion will be restricted to soil strength parameters deduced from the loading test results. Table 2 presents a comparison between the undrained shear strengths and shear moduli which were determined based on the Tresca model. Examination of the data immediately shows that both the PMTs and ISCTs greatly overestimate the undrained shear strength compared to SBPMTs and VSTs. In addition, SBPMTs overestimate by 10 to 25% the undrained shear strength deduced from VSTs. The existence of such differences between PMT, SBPMT, and VST data has been known for several decades (See, for example, Roy et al. 1975 and Lacasse et al. 1981). The cause has been attributed to a) the existence of a remoulded annulus of soil at the wall of the pilot holes due to the boring technique, b) unloading of the soil resulting from drilling of the pilot holes, and c) the possibility that the PMT tests were carried out under partially drained conditions. While factors a) and b) are considered to lead to overestimation of the undrained shear strength (Baguelin at al. 1978; Prévost 1979; Eden and Law 1980; Law and Eden 1982; Benoît and Clough 1986; Prapaharan et al. 1990), the last factor c) is less important because the tests were carried out quite rapidly. There is, however an additional factor which is believed to have considerable impact on the experimental data obtained from the ISCTs. Indeed, after completion of these tests, the boreholes were examined for possible damage resulting from the sharp tip of the probe. It was discovered that the interior surface of 20 to 40% of the pilot holes was not as smooth as at the beginning of the tests. The steady penetration of ISC caused spalling of the clay ahead of the advancing conical probe. There were fragments and chunks of soil on the bottom of the pilot holes. Such a phenomenon which is thought to especially occur in fragile materials like the sensitive clays of eastern Canada, may be eliminated by conducting ISCTs using a self-boring device.

Concerning the shear modulus G deduced from the data, Table 2 indicates that while the highest values are obtained from the SBPMTs, the lowest values are deduced from the ISCTs. Borehole disturbance and unloading are thought to be responsible for the poor performance of the PMTs and ISCTs, as also discussed by Silvestri and Abou-Samra (2008).

Table 2 Soil strength parameters

Depth	PMT		ISCT		SBPMT		VST
	S_u	G	S_u	G	S_u	G	S_u
(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)
2.5	100	1000	184	1050	91	--	50
4.5	180	4540	189	1550	91	8190	72
6.0	260	5100	--	--	91	8190	80

Stress Paths

Total and effective stress paths, followed by the clay during the expansion process, could only be deduced from SBPMT data, because only this test allowed determination of the initial stress state. For discussion purposes, only the test completed at the depth of 4.5 m will be analysed in the present study. Total stress paths were obtained from both the Tresca model and Palmer's approach. The stress parameters, s and t are given by $s = (\sigma_r + \sigma_t)/2$ and $t = (\sigma_r - \sigma_t)/2$, where σ_t = tangential stress, and become $s = \sigma_r - \tau$ and $t = \tau$, because $\sigma_t = \sigma_r - 2\tau$. The total stress paths are shown in Fig. 7. For the Tresca model, the initial phase of the expansion is elastic for $t \leq S_u$ and, therefore, s is constant, equal to p_o or σ_{ho}. Once the shear stress reaches the shear strength S_u, the soil becomes plastic, and the stress path turns to the right and remains horizontal until the radial pressure σ_r attains its ultimate value, $p_{ult} = p_o + S_u\ [1+\ln(G/S_u)]$, as shown by Gibson and Anderson (1961).

For Palmer's approach, the resulting stress path is curved, as shown in Fig. 7. At large strain, the stress path converges with that deduced from the Tresca model, because the shear strength S_u is the same in both cases. The effective stress path, which is also shown in Fig. 7, was obtained by subtracting the pore water pressure measured during the expansion test from the curved Palmer's total stress path.

Because the effective stress path is almost vertical throughout the deformation process, the increase in total radial pressure during the plastic phase of the expansion is practically equivalent to the excess pore water pressure generated in the clay. Such behaviour is clearly indicated in Fig. 5 where the curves of radial pressure are almost parallel to those of pore water pressure.

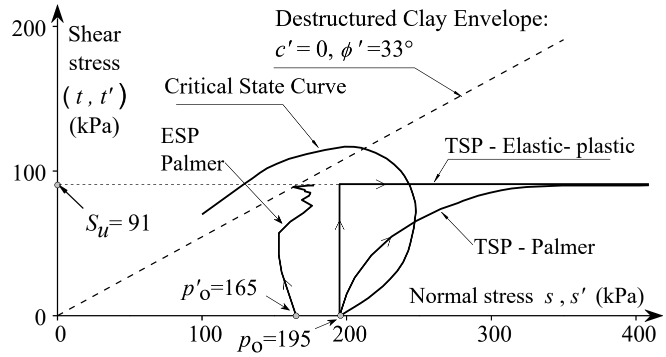

Fig. 7: Stress paths from SBPTM. c', effective cohesion; ESP, effective stress path; p', effective pressure; p_o, initial horizontal total pressure; p'_o, initial horizontal effective pressure; S_u, undrained shear strength; TSP, total stress path; ϕ', effective friction angle.

6. CONCLUSIONS

The following conclusions ensue from the results reported in the paper:

a) Both the PMTs and ISCTs considerably overestimate the undrained shear strength of the clay, as compared to VSTs. The cause is attributed to borehole disturbance and unloading.
b) SBPT-deduced S_u values are from 10 to 25% higher than VST-derived parameters.
c) Shear moduli derived from PMTs and ISCTs are much lower than values deduced from SBPMTs. Again, the cause is probably related to borehole disturbance and unloading.
d) Initial horizontal geostatic stresses were found to be much higher than expected using a well-known correlation.
e) Because of the spalling phenomenon of the pilot holes ahead of the advancing ISC, it is recommended that ISCTs be performed using a self-boring device. This will also avoid unloading and will minimize remoulding of the soil around the boreholes.

7. ACKNOWLEDGEMENTS

The senior author expresses his gratitude to the late Professor Marius Roy of Laval University for making available the SBPMT test data. The authors are grateful to the Natural Sciences and Engineering Research Council of Canada for the financial support received in the course of the present investigation.

8. REFERENCES

Baguelin, F., Jézéquel, J.F., and Shields, D.H. 1978. "The pressuremeter and foundation engineering". *Trans Tech Publications*, Clausthal, Germany.

Benoît, J., and Clough, G.W.1986. "Self-boring pressuremeter tests in clay". *Journal of the Geotechnical Engineering Division, ASCE*, 112:60-78.

Demers, D. 2001. "Personal communication". *Department of Transport of Quebec*, Quebec City, Quebec, Canada.

Eden, W.J., and Law, K.T. 1980. "Comparison of undrained shear strength results obtained by different test methods". *Canadian Geotechnical Journal*, 17:369-381.doi:10.1139/t80-044.

Felio, G.Y., and Briaud, J.L. 1986. "Conventional parameters from pressuremeter tes data: review of existing methods". Proceedings of the 2nd International Symposium on the Pressuremeter and its Marine Applications, College Station, TX., 2-3 May 1986, ASTM STP 950, pp. 265-282.

Gibson, R.E., and Anderson, W.F. 1961. "In situ measurements of soil properties with the pressuremeter". *Civil Engineering and Public Works Review*, 56:615-618.

Hamouche, K.K., Leroueil, S., Roy, M., and Lutenegger, A.J. 1995. "In situ evaluation of K_0 in eastern Canada clays." *Canadian Geotechnical Journal*. 32(4):677-688.

Lacasse, S., Jamiolkowski, M. Lancellotta, R., and Lunne, T. 1981. "In situ characteristics of two Norwegian clays". *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering*, 15-19 June 1981, Stockholm, Vol. 2, pp.507-511.

Ladanyi, B., and Longtin, H. 2005. "Short- and long-term sharp cone tests in clay". Canadian Geotechnical Journal, 42:136-146.

Law, K.T., and Eden, W.J. 1982. "Effects of soil disturbance in pressuremeter test". *Proceedings of the Conference on Updating Subsurface Sampling of Soils and Rocks and their In-Situ Testing.* Santa Barbara, Calif., 3-8 January 1982, pp. 291-303.

Longtin, H. 2004. "Characteristics of clayey soils by instrumented sharp cone" (In French). *Master's Thesis, Department of Civil, Geological, and Mining Engineering, Ecole Polytechnique of Montreal*, Montreal, Quebec, Canada.

Palmer, A. C. 1972. "Undrained plane strain expansion of a cylindrical cavity in clays: a simple interpretation of the pressuremeter test". Geotechnique, 22:451-457.

Prapaharan, S., Chameau, J.L., Altschaeffl, A.G., and Holtz, R.D. 1990. "Effect of disturbance on pressuremeter results in clays". *Journal of Geotechnical Engineering, ASCE*, 116:35-53.

Prévost, J.H. 1979. "Undrained shear tests on clay". *Journal of the Geotechnical Engineering Division, ASCE*, 105(GT1):49-64.

Roy, M. Juneau, R. La Rochelle, P., and Tavenas, F.A. 1975. "In situ measurements of the properties of sensitive clays by pressuremeter tests". *Proceedings of the Conference on In-situ Measurement of Soil Properties*, Raleigh, N.C., 1-4 June 1975, Geotechnical Engineering Division, ASCE, New York. Vol. 1, pp. 350-372.

Silvestri, V., and Abou-Samra, G. 2008. "Analysis of instrumented sharp cone and pressuremeter tests in stiff sensitive clay". *Canadian Geotechnical Journal*, 45:957-972.

Whittle, D., and Wroth, C.P. 1977. "Use of self-boring pressuremeter to determine the undrained properties of clays". *Ground Engineering*, 10:37-46.