Application of the MCC Model for the Estimation of Undrained Geotechnical Parameters of Clays from Dilatometer Tests

Vincenzo Silvestri

Ecole Polytechnique, Montreal , Quebec, Canada. E-mail: vincenzo.silvestri@polymtl.ca

Claudette Tabib

Consultant, Montreal, Quebec, Canada. E-mail: claudette.tabib@yahoo.ca

Keywords: DMT, SBPMT, sensitive clay, cylindrical cavity, MCC model

ABSTRACT: Dilatometer test (DMT) results and correlations, obtained in a sensitive clay of Eastern Canada, are compared to data computed from self-boring pressuremeter tests (SBPMT), hydraulic fracture tests (HFT), and vane shear tests (VST). The results indicate that the original DMT corrections must be partly modified for obtaining meaningful data. A semi-analytical solution for the undrained expansion of a cylindrical cavity in Modified Cam Clay (MCC) is used for the computation of undrained shear strengths, and limit radial and pore pressures, which are compared to the values found in the DMT tests. Complementary MCC analyses are also performed on a well-known insensitive clay and the results are employed to point out the utmost importance of remoulding on measured pressures in DMT tests, following the installation of the dilatometer.

1. INTRODUCTION

The flat dilatometer test (DMT) was introduced by Marchetti (1980) as a new in-situ penetration test in soils. The equipment and test procedure are simple, and the test provides repeatable, nearly continuous data that have been correlated to a number of important soil parameters. In the original development, Marchetti (1980) combined the corrected pressures p_o and p_1 with the in-situ pore pressure u_o and effective overburden pressure σ'_{vo}, and obtained the following parameters

$$I_D = \text{Material Index} = (p_1 - p_o)/(p_o - u_o) \quad (1)$$

$$K_D = \text{Lateral stress index} = (p_o - u_o)/\sigma'_{vo} \quad (2)$$

$$E_D = \text{Dilatometer modulus} = 34.7\,(p_1 - p_o) \quad (3)$$

In clays the Lateral stress index, K_D, is of paramount importance, because it is employed (Marchetti 1980) in the estimation of a) the in-situ coefficient of lateral earth pressure, K_o, from the expression

$$K_D = (K_D/\beta_k)^{0.47} - 0.6 \quad (4)$$

with β_k =1.5, b) the overconsolidation ratio, OCR, on the basis of

$$\text{OCR} = (0.5\,K_D)^{1.56} \quad (5)$$

and c) the undrained shear strength, S_u, using the SHANSEP procedure relating the stress ratio S_u/σ'_{vo} to OCR, that is

$$S_u/\sigma'_{vo} = 0.22\,(0.5\,K_D)^{1.25} \quad (6)$$

The latter expression applies to clays that are either normally consolidated or have been rendered overconsolidated by unloading, but are neither cemented nor sensitive (Marchetti et al. 2001). Generally, investigators have found that comparisons of undrained shear strength are reasonably accurate in softer clays but appear to be less accurate in stiffer, older overconsolidated clays (Lacasse and Lunne 1988; Powell and Uglow 1986, 1988; Lutenegger and Blanchard 1990; Lutenegger 2006).

Because the problem of a rational interpretation of the DMT in clays is still not fully understood, DMT results have often been correlated to limit pressures deduced from conventional pressuremeter tests (PMT), self-boring pressuremeter tests (SBPMT), full-displacement cone pressuremeter tests (FDCPMT), and tip resistances measured in cone penetration tests (CPT), as reported, for example, by Garga and Khan (1991), Hamouche et al. (1995), Lutenegger (2006), Mayne (2006), and Robertson (2009). But, as the interpretation of pressuremeter tests in undrained clay is amenable to theoretical analysis, whereas that of CPT tests is still mostly based on empirical correlations, with the exception of a limited number of studies (Teh and Houlsby 1991; Yu 1993; Yu et al. 1993), possible DMT – SBPMT relationships were favoured in the present investigation. Preliminary calculations were then carried out using well-documented case histories with the aim of finding correction factors to apply to SBPMT data, since the strain distribution that arises around an expanding cylindrical cavity is quite different from that generated during the penetration of the flat blade of the dilatometer, as illustrated by the work of Whittle et al. (1989) which was based on the strain path method of Baligh (1985). However, the approach led to inconclusive results due to the complexity of the strain field around the flat dilatometer. It was therefore decided to directly compare DMT results with SBPMT data, following Clarke and Wroth (1988), and Lutenegger (2006).

More specifically, this paper analyses results of a field investigation carried out in a lightly overconsolidated sensitive clay deposit of Eastern Canada, by means of hydraulic fracture tests (HFT), self-boring pressuremeter tests (SBPMT), vane shear tests (VST), and Marchetti dilatometer tests (DMT). While the original tests were performed and reported by Hamouche (1995), the interpretation and analyses are solely those of the authors. It is shown that the correlations proposed by Marchetti (1980) have to be slightly modified to obtain realistic results in this sensitive clay. Additional analyses were performed using the effective stress model of Modified Cam Clay for the interpretation of the undrained expansion of a cylindrical cavity, with the aim of comparing the lateral stresses and pore pressures generated around the self-boring pressuremeter with those arising in the soil surrounding the flat blade of the dilatometer.

2. FIELD TEST RESULTS

To resolve some of the inconsistencies reported in the literature concerning DMT-deduced correlations, it was deemed necessary to have at one's disposal well-executed and complementary field tests. In addition, if the geotechnical properties of the clay deposit at hand have been studied by means of laboratory and field tests, then comparisons can be made, correlations verified, and inconsistencies removed.

The investigation performed by Hamouche (1995) still constitutes, in the authors opinion, one of the most complete field study of the sensitive clay deposits of Quebec. Several in-situ tests were carried out on the experimental site of Louiseville, a town located 125 km northeast of Montreal. The site has been studied over the past 35 years by research teams from Laval University of Quebec and Ecole Polytechnique of Montreal. For example, Hamouche et al. (1995) analysed the causes of the unusual high K_o values deduced from SBPMT and HFT tests, and Silvestri (2003) compared the undrained shear strength S_u derived from vane tests with values predicted from SBPMT tests, using an elastic-plastic total stress approach. More recently, Silvestri and Tabib (2013) interpreted SBPMT results using an improved solution for the undrained expansion of a cylindrical cavity by modelling the soil as Modified Cam Clay (MCC).

Fig. 1 presents the soil stratigraphy. A 1.8 [*m*] thick crust of oxidized and fissured clay overlies the

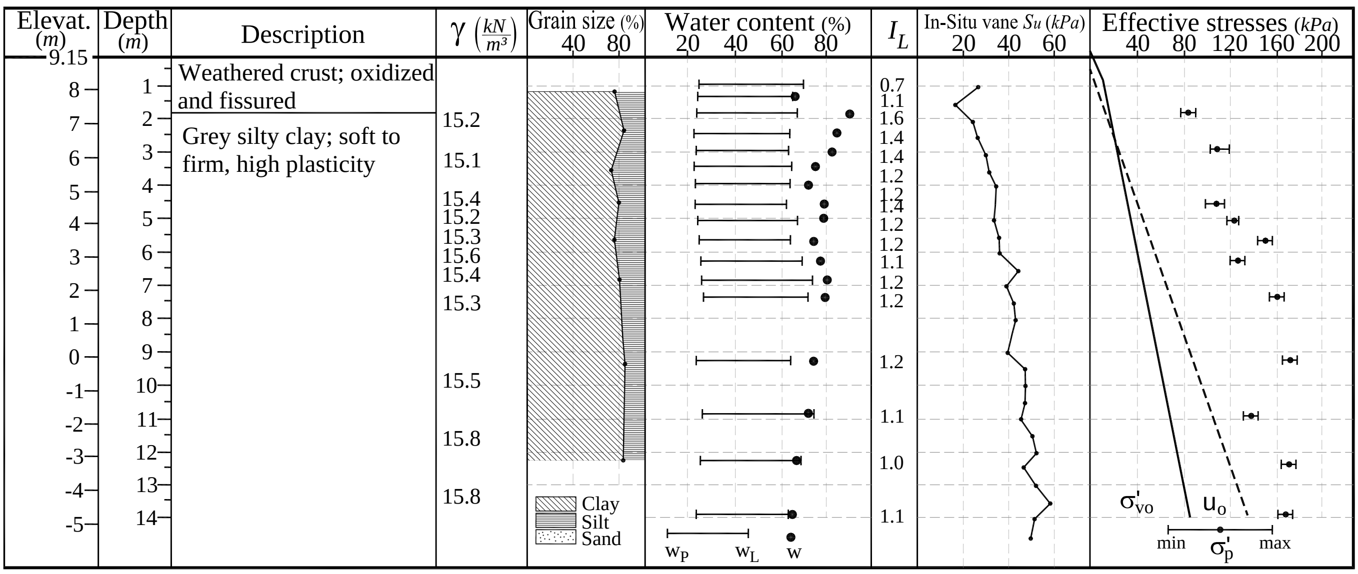

Fig. 1. Soil stratigraphy

main deposit of sensitive clay whose total thickness is about 60 [*m*] at the site. In the depth interval of interest, from 1.8 to 14 [*m*], the natural moisture content decreases from 90 to 65%, the liquidity index varies from 1.6 to 1.1, and the plasticity index stays constant at 45%. The field undrained shear strength S_u, measured with a Nilcon vane, increases linearly with depth, from 20 [*kPa*] at 1.8 [*m*] to 55 [*kPa*] at 14 [*m*]. The overconsolidation ratio (OCR) deduced from oedometer tests decreases from 5.6 at 1.8 [*m*] to 2.4 at 14 [*m*].

DMT pressures p_o and p_1 are compared with in-situ effective overburden pressure σ'_{vo} and pore pressure u_o in Fig. 2. Resulting Material and Lateral stress indices, I_D and K_D respectively, are also reported in Fig. 2.

Fig. 3 presents a comparison between K_o values inferred from both SBPMT lift-off pressures and HFT closure pressures, and those determined from Eq. (4) for β_k =1.5 and 2. It appears that Eq. (4) with β_k =1.5 provides a better correlation. This is contrary to what was reported by Hamouche et al. (1995), for whom β_k =2 resulted in better agreement. Fig. 4 compares the overconsolidation ratios determined from oedometer tests with corresponding values obtained from Eq. (5). A better agreement is found by using the relationship

$$\mathrm{OCR} = \delta_k \, (K_D)^{1.56} \qquad (7)$$

with δ_k = 0.35 – 0.45, which was proposed by Lunne et al. (1989) for young clays. Fig. 5 compares the undrained shear strength predicted from Eq. (6) with values deduced from SBPMTs and VSTs. It is evident that while S_u values determined from field vane tests agree well with predictions from Eq. (6), SBPMT deduced values are overestimated considerably.

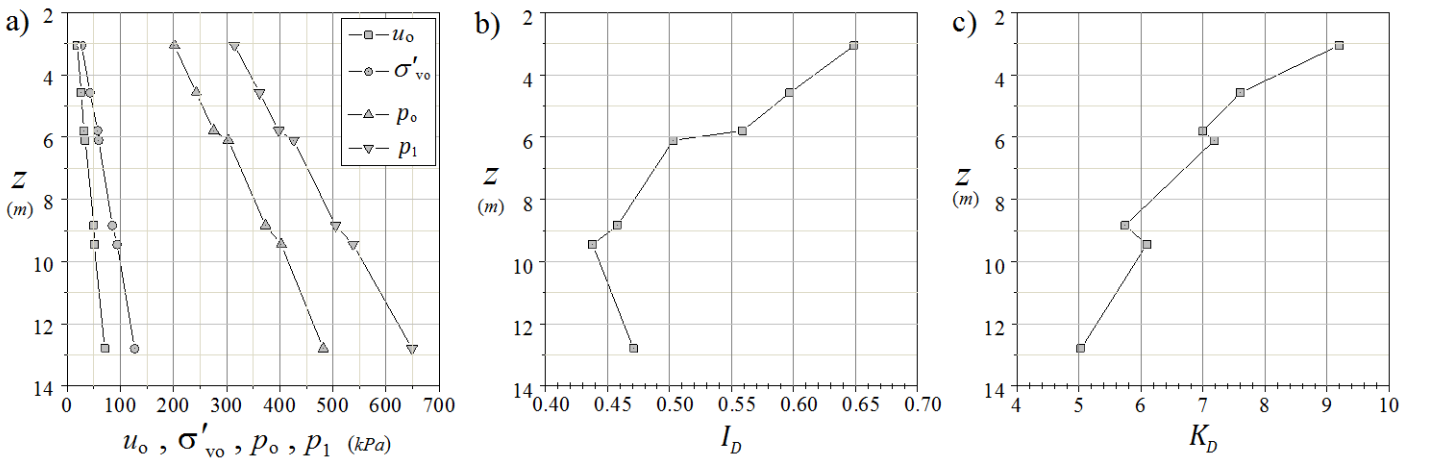

Fig. 2. DMT results.

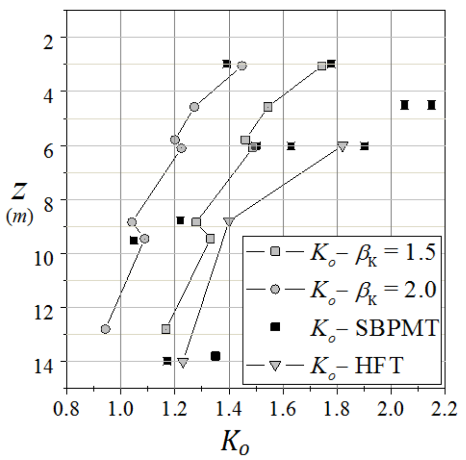

Fig. 3. K_o correlations.

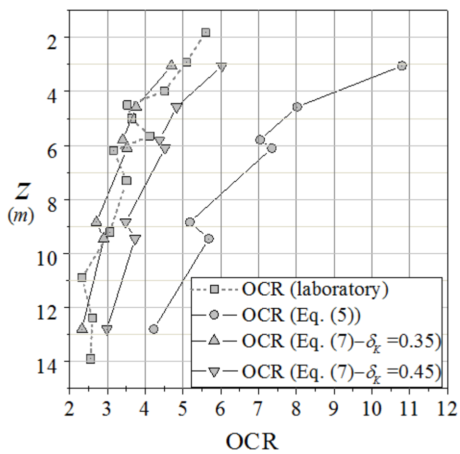

Fig. 4. OCR correlations.

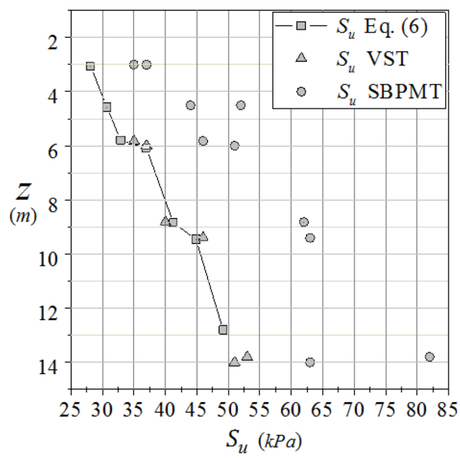

Fig. 5. S_u correlations.

3. EXPANSION OF CYLINDRICAL CAVITIES

As mentioned above, interpretation of the self-boring pressuremeter tests was approached by considering that the tests simulate the expansion of cylindrical cavities in Modified Cam Clay under plane strain and undrained conditions. Soil parameters needed to describe the MCC model were determined from triaxial tests carried out on undisturbed clay specimens which were recovered at a depth of 6 [*m*]. The derived Cam Clay parameters are: υ=3.07, λ=0.65, κ=0.03, Λ= $((\lambda-\kappa))/\lambda$=0.954, and M=1.2. The specific volume υ equals (1+*e*), with *e* = void ratio; λ is the slope of υ : ln p' line in loading, with p' = mean effective stress; κ is the slope of the υ: lnp' line in unloading; and $M=6\sin\phi'/(3-\sin\phi')$, is the gradient of the critical state line, with ϕ' =30°. The effective stress path followed by the clay in the undrained cylindrical expansion is shown in Fig. 6 on a p': q plane, where q = deviator stress. The path comprises two sections: in the first vertical section AB, the soil is overconsolidated and elastic, and in the last section BF, the soil becomes plastic and reaches the critical state at point F. The coordinates of the in-situ stress state at point A are p'_i =40 [*kPa*] , q_i =0 , K_o=1. The coordinates of the critical state at point F are $p'_f = 2^{-\Lambda}\, p'_o$ = 77.5 [*kPa*], with p'_o = 150.1 [*kPa*], and q_f =M p'_f = 93.0 [*kPa*]. The latter parameter allows the computation of the undrained shear strength S_u which is equal to q_f /$(3^{0.5})$ or 53.7 [*kPa*]. The parameter p'_o represents the isotropic effective stress on the plastic effective stress path, as shown in Fig. 6.

Fig. 7 compares the total radial stress σ_{ra} and pore pressure u measured at the wall of the cavity in the SBPM tests with the theoretical values predicted using the semi-analytical solution of Silvestri and Tabib (2013). The horizontal axis in this figure represents the finite Almansi tangential strain $\alpha_o = ((a^2 - a_o^{\,2}))/(2a^2)$, where a_o and a refer to the initial and current radii of the cavity, respectively. The clay reached failure at a tangential strain of about 10% in all SBPM tests.

Fig. 8 compares DMT pressures p_o and p_1 with SBPMT lift-off pressures p_{oh} , experimental limit pressures p_{lim}, and theoretical limit pressures p_L computed by extending the expansion process to the ultimate Almansi tangential strain α_o of 0.5

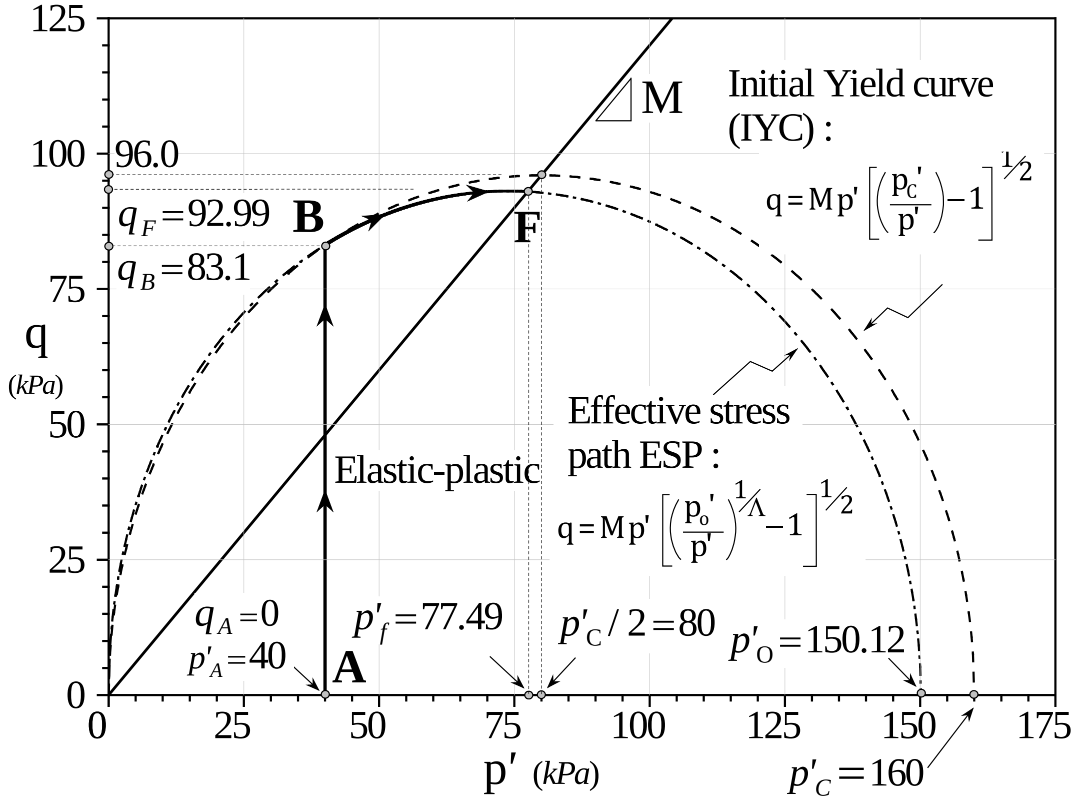

Fig. 6. Effective stress path for Louiseville clay at z=6.0 [*m*] in undrained expansion.

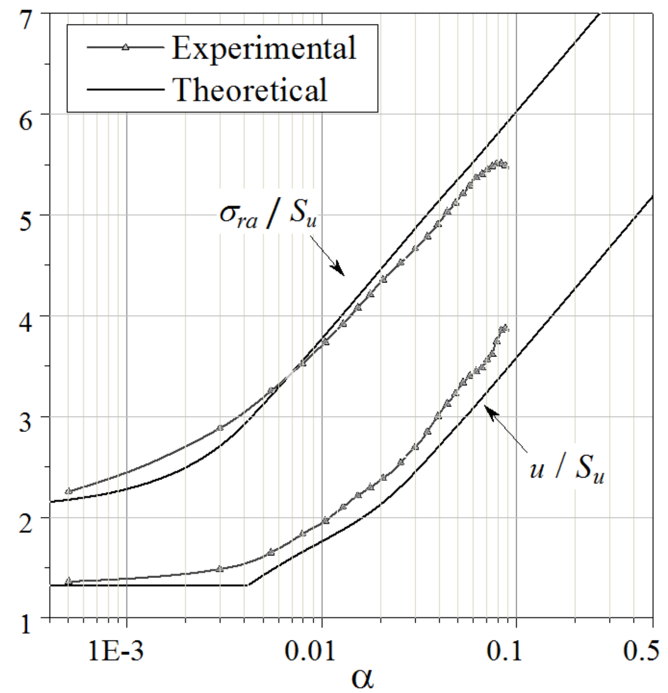

Fig. 7. Total stresses and pore pressures for clay at z=6.0 [m].

Examination of the data reported in Fig. 8 indicates that, on the one hand, DMT pressures p_o are practically equivalent to SBPMT experimental limit pressures p_{lim} , and, on the other hand, DMT pressures p_1 are also quite similar to SBPMT theoretical limit pressures p_L. It thus appears that since theoretical limit pressures p_o are approximately equal to the experimental radial pressures measured at failure in the SBPMT tests, the penetration of the dilatometer allowed failure to also occur in the sensitive clay surrounding the flat blade. As for the theoretical limit pressures, p_L, which were never reached in the real SBPMT tests, they were nonetheless attained in the expansion phase of the DMT tests (i.e., $p_L \approx p_1$), thus indicating that very large strains are generated around the dilatometer blade, as also discussed by Whittle et al. (1989).

To gain further insight into the mechanisms which govern the generation of lateral radial stresses and pore pressures around an expanding cylindrical cavity in Modified Cam Clay, the solution reported in Silvestri and Tabib (2013) was applied to a well-known case-history simulation involving the insensitive Boston Blue Clay (Carter et al. 1979).

The MCC soil parameters are: υ=2.16, λ=0.15, κ=0.03 , $\Lambda=((\lambda-\kappa))/\lambda=0.8$, and M=1.2. Due to lack of space, only the cases of OCR=1 and OCR=32 will be discussed in some detail herein. The initial stress states and properties are reported in Table 1. The total stresses and pore pressures at critical state are summarized in Table 2.

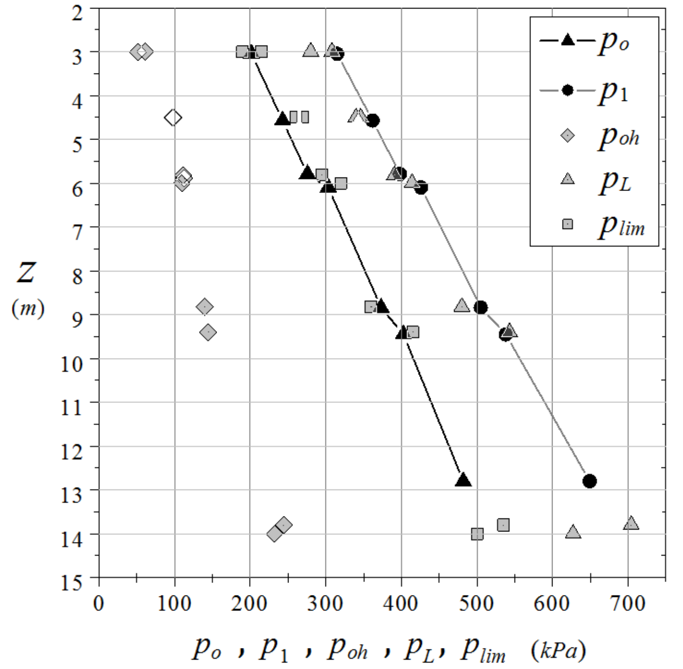

Fig. 8. Comparison of DMT p_o and p_1 with SBPMT p_{oh}, p_{lim} and p_L.

If the OCR values in Table 1 are employed for the calculation of the Lateral stress index, K_D, by means of Eq. (5) proposed by Marchetti (1980), then it becomes possible to compute K_o and S_u /(σ'_{vo}) from Eqs. (4) and (6), respectively. The results which are reported in Table 3 together with the true values based upon the entries in Tables 1 and 2 show that Marchetti's correlations lead to realistic results for OCR ≤ 4 to 8.

Fig. 9 presents the stress paths followed by all soil specimens. For the K_o–normally consolidated soil of OCR=1, the initial stress state, represented by point A, is already on the plastic effective stress path. The critical state is reached at point C, whose coordinates are $p'_f = 2^{-\Lambda}\, p'_o = 2^{-0.8}\, 257$ [kPa] =147.6 [kPa] , q_f =M p'_f =177.1 [kPa], from which $S_u = q_f /(3^{0.5})$ =102.3 [kPa]. For the K_o–rebounded overconsolidated clays, the initial stress states is also represented by point A in each case. However, the initial segment AB of each stress path is now vertical, because the clay is elastic. Once the stress path reaches the yield surface, the clay becomes plastic and the path follows the curved segment BC. The effective radial, vertical, and tangential stresses at critical state are unique; they are given by (Carter at al. 1979):

$$\sigma'_{rf} = p'_f \ [(\mathrm{M} / 3^{0.5}) + 1] \qquad (8a)$$

$$\sigma'_{vf} = p'_f \ [\mathrm{M} / 3^{0.5}] \qquad (8b)$$

$$\sigma'_{\theta f} = p'_f \; [(M / 3^{0.5}) - 1] \quad (8c)$$

Table 1 Initial soil properties

OCR	K_o	σ'_{vo} [kPa]	$\sigma'_{ro}=\sigma'_{\theta o}$ [kPa]	G [kPa]
1	0.55	300	165	7570
2	0.70	169.8	118.9	8488
4	1.00	92.2	92.2	9307
8	1.35	50.9	68.7	10227
32	2.75	15.0	41.3	12679

Table 2 Stresses at critical state

OCR	σ_{rf} [kPa]	σ_{vf} [kPa]	$\sigma_{\theta f}$ [kPa]	u_f [kPa]	S_u [kPa]
1	685.5	583.0	480.7	435.5	102.3
2	685.2	582.9	480.7	435.4	102.3
4	656.4	554.1	451.8	405.7	102.3
8	642.1	539.8	437.5	392.3	102.3
32	619.5	517.2	414.9	369.5	102.3

Table 3 Deduced DMT parameters and correlations

OCR	K_D	K_o		$S_u/(\sigma'_{vo})$	
	Eq. (5)	Table 1	Eq.(4)	Tables 1 and 2	Eq. (6)*
1	2	0.55	0.54	0.34	0.34
2	3.12	0.70	0.81	0.60	0.59
4	4.86	1.00	1.14	1.11	1.03
8	7.58	1.35	1.54	2.10	1.80
32	18.44	2.75	2.65	6.82	5.46

* Eq. (6) was modified to read

$$S_u/(\sigma'_{vo}) = 0.34\,(0.5\,K_D)^{1.25}$$

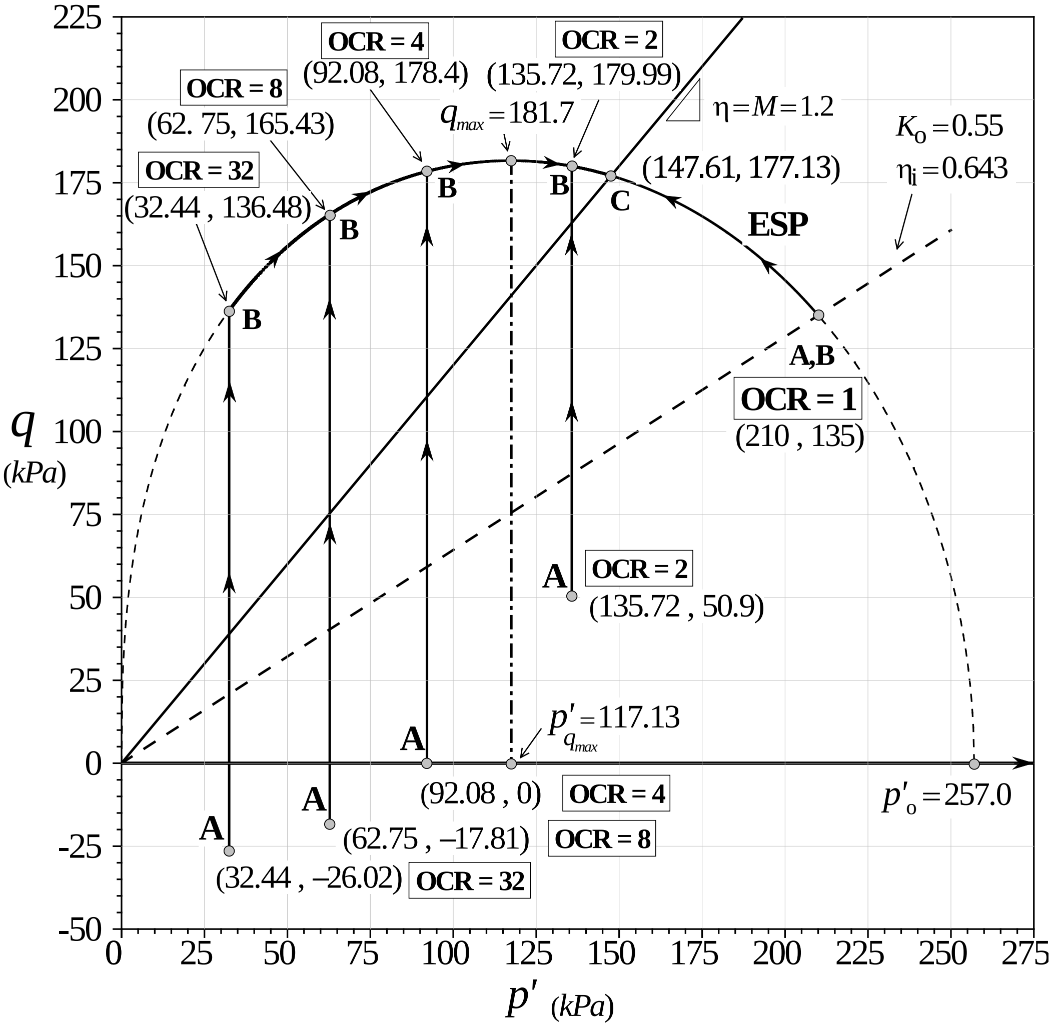

Fig. 9. Effective stress paths for Boston Blue Clay in undrained expansion.

Fig. 10 presents the total radial stress σ_{ra} and pore pressure u generated at the wall of the cavity as function of the finite Almansi strain α_o. Their values at critical stress state are given approximately by:

$$\sigma_{rf} = \sigma_{ro} + S_u [1 + \ln (G / S_u)] \quad (9a)$$

and

$$u_f = u_o + \sigma_{rf} - p'_f \ [(M / 3^{0.5}) + 1] \quad (9b)$$

where σ_{ro} , u_o are the corresponding initial values, and G is the shear modulus. Note that Eq. (9a) is the same as that derived from a linearly elastic perfectly plastic analysis. For example, for OCR=1, Eqs. (9) give σ_{rf} =707.6 [*kPa*] and u_f =457.7 [*kPa*], compared to σ_{rf} = 685.5 [*kPa*] and u_f =435.5 [*kPa*] from Table 2. As for OCR=32, Eqs. (9) yield σ_{rf} = 636.7 [*kPa*] and u_f = 386.8 [*kPa*], compared to σ_{rf} = 619.5 [*kPa*] and u_f = 369.5 [*kPa*] from Table 2.

It should be noted that the validity of the MCC model was considered to apply even to the heavily overconsolidated cases reported in Tables 1 and 2, although it is well known that the MCC model overestimates considerably the strength of such clays.

The result shown in Fig. 10 indicate quite clearly that the stresses at critical state are essentially function of the initial undrained shear strength S_u and rigidity index G/S_u. It is thus plausible to assume that if the clay is remoulded either prior to the performance of a SBPMT test or during penetration of the dilatometer blade, there would then follow partial loss of strength and rigidity with a consequent decrease of the lateral pressures, as suggested by Lutenegger and Timian (1986). Such phenomenon will undoubtedly be more pronounced in sensitive cemented clays than in insensitive plastic clays. Because the original correlations (i.e., Eqs. (4), (5), and (6)) proposed by Marchetti (1980) give reasonable results in the latter clays, then it is quite probable that such soils are not unduly remoulded following the penetration of the dilatometer blade. As for sensitive clays, remoulding of their particular structure, acquired through unloading and cementation, is responsible for the observed deviations from Marchetti's corrections.

As for old heavily overconsolidated clays, penetration of the flat dilatometer blade induces very large strains in the surrounding soil. These are incompatible with the strains, which are generated from self-boring pressuremeter tests, for which failure is reached too early. As a result, even in the stiff clays, DMT-deduced pressures p_1 are usually higher than experimentally-inferred SBPMT limit radial pressures, as reported, for example, by Powell and Uglow (1986), and Clarke and Wroth (1988).

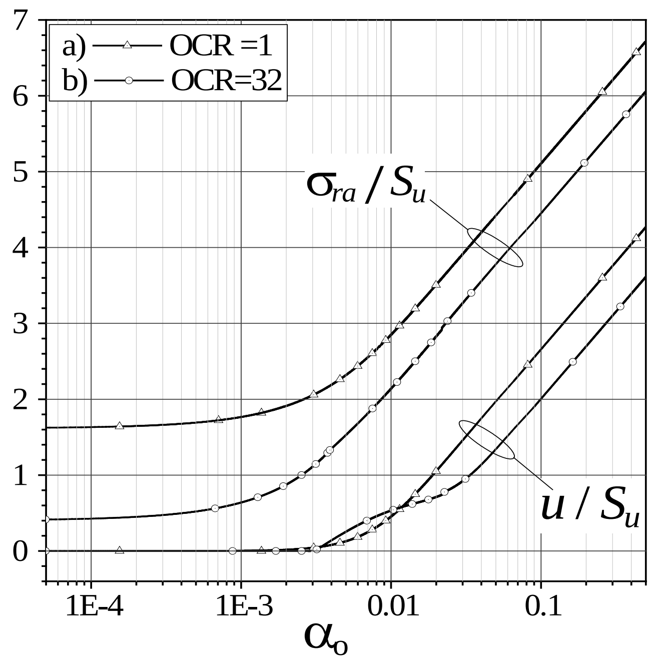

Fig. 10. Variation of total lateral stress and pore pressure in Boston Blue Clay.

4. CONCLUSIONS

The following conclusions are drawn on the basis of the results obtained in this paper:

a) The Marchetti's correlation involving the overconsolidation ratio, OCR, was modified for obtaining realistic results in the Louiseville clay.
b) DMT pressures, p_o, were found to be equivalent to SBPMT experimentally-measured limit pressures, p_{lim}.
c) DMT pressures, p_1, were found to be similar to SBPMT theoretically predicted limit pressures, p_L.
d) The particular response of the Louiseville clay is considered to stem from the nature of the overconsolidation which is due to both unloading and cementation bonds.
e) Complementary analyses involving Boston Blue Clay pointed out the paramount importance of the undrained shear strength and the rigidity index in the generation of the lateral stresses and pore pressures. Partial remoulding of the clay could then lead to lower values.

5. ACKNOWLEDGEMENTS

The authors express their gratitude to the Natural Sciences and Engineering Research Council of Canada for the financial support received in the course of the present study.

6. REFERENCES

Baligh, M. 1985. "Strain path method." *J. Geotech. Eng.*, 111(9): 1103-1136.

Carter, J.P., Randolph, M.F., and Wroth, C.D. (1979). "Stress and pore pressure changes in clay during and after the expansion of a cylindrical cavity." *Int. J. Numer. Anal. Meth. Geomech.*, 3(4),305-323.

Clarke, B.G., and Wroth, C.P. (1988). "Comparison between results from flat dilatometer and self-boring pressuremeter tests." *Penetration Testing in the UK, Proc. Geotechn. Conf.*, University of Birmingham, July 6-8, ICE, Thomas Telford, London, pp. 141-144.

Garga, V.K., and Khan, M.A. (1991). "Laboratory evaluation of K_o for overconsolidated clays." *Can. Geotech. J.*, 28(5),650-659.

Hamouche, K.K. (1995). "Comportment des argiles Champlain sollicitées horizontalement." *Ph. D. Thesis.* Département de Génie Civil, Université Laval, Québec, Que., Canada.

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

Lacasse, S., and Lunne, T. (1988). "Calibration of dilatometers correlations." *Proc. Ist. Int. Symp. Penetr. Test.* (ISOPT-1), Orlando, Balkema, Rotterdam, Vol. 1, pp. 539-548.

Lunne, T., Lacasse, S., and Rad, N.S. (1989). "State of the art report on in situ testing of soils." *Proc. 12th Int. Conf. Soil Mech. Found. Engng.*, Rio de Janeiro, August 13-18, Balkema, Rotterdam, Vol. 4 , pp. 2339-2403.

Lutenegger, A.J., and Timian, D.A. (1986). "Flat-plate penetrometer tests in marine clays." *Proc. 38th Can. Geotech. Conf.*, Ottawa, August 27-30, pp.301-308.

Lutenegger, A.J. 2006. "Cavity expansion model to estimate undrained shear strength in soft clay from dilatometer." *Proc. 2nd Int. Flat Dilat. Conf.*, Washington, April 2-5, In-situ Soil Testing, pp. 319-326.

Lutenegger, A.J., and Blanchard, J. D. (1990). "A comparison between full displacement pressuremeter and dilatometer tests in clay." *Proc. 3rd. Int. Symp. Press.* (Pressuremeters-ISP3),Oxford University, April 2-6, Thomas Telford, London, pp. 309-320.

Marchetti, S. (1980). "In situ tests by flat dilatometer." *J. Geotech. Eng. Div.*, ASCE, 106(GT3):229-231.

Marchetti, S., Monaco, P., Tatani, G., and Calabrese, M. (2001). "The flat dilatometer test (DMT) in soil investigations." *A report by the ISSMGE Committee TC 16. In Proc. Int. Conf. In Situ Meas. Soil Prop. and Case Hist.*, Bali, May 21-24, Graduate Program, Parahyangan Catholic University,41p.

Mayne, P.W., (2006). "Interrelationships of DMT and CPT readings in soft clays." *Proc. 2nd Int. Flat Dilat. Conf.*, Washington, April 2-5, In-Situ Soil Testing, pp. 220-225.

Powell, J.J.M., and Uglow, I.M. (1988). "Marchetti dilatometer testing in U.K." soils. *Proc. Ist Int. Symp. Penetr. Test.* (ISOPT-1), Orlando, Balkema, Rotterdam, Vol.1, pp. 552-562.

Powell, J.J.M., and Uglow, I.M.(1986). "Dilatometer testing in stiff overconsolidated clays." *Proc. 39th Can. Geotech. Conf.*, Ottawa, August 27-30, pp 317-326.

Robertson, P.K. (2009). The James K. Mitchell Lecture: "Interpretation of in-situ tests. Some insights. " *Proc. Geotech. & Geophys.*, Site Charac. 4, CRC Press, Boca Raton , Vol. 1 , pp. 2-24.

Silvestri, V. (2003). "Assessment of self-boring pressuremeter tests in sensitive clay." *Can. Geotech. J.* , 40(2):362-387.

Silvestri, V., and Tabib, C. (2013). "Analysis of self-boring pressuremeter tests in a sensitive clay of Quebec." *Proc. 18th Int. Conf. Soil Mech. Found. Eng.*, Paris, September 2-6, Pressio 2013 (ISP6), pp. 1-4.

Teh, C.I., and Houslby, G.T. (1991). "An analytical study of the cone penetration test." *Géotechnique*, 41(1):17-34.

Whittle, A.J., Aubeny, C.P., Rafalovich, A., Ladd, C.C., and Baligh, M.M. (1989). "Interpretation of in situ testing of cohesive soils using rational methods." *Annual Report, Air Force Office of Scientific Research*, Washington, 238p.

Yu, H.S. (1993). "A new procedure for obtaining design parameters from pressuremeter tests." *Aust. Trans. Civil Engng.*, CE35(4) pp:353-359.

Yu, H.S., Carter, J.P., and Booker, J.R. (1993). "Analyses of the dilatometer test in undrained clay." In *Predictive Soil Mechanics: Proc. Wroth Memorial Symposium*, Thomas Telford, London, pp. 785-795.