FIELD MEASUREMENTS OF COHESION IN CHAMPLAIN CLAYS

MESURES EN PLACE DE LA COHESION DANS LES ARGILES CHAMPLAINS

ПОЛЕВЫЕ ОПРЕДЕЛЕНИЯ СЦЕПЛЕНИЯ ГЛИНЫ В МЕСТНОСТИ ШАМПЛЕН

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SYNOPSIS. This paper presents the results of a comparative study between three *in situ* testing methods (vane, pressuremeter and penetrometer) to measure the undrained shear strength in deposits of a sensitive cemented clay on two different sites. To evaluate the effect of disturbance due to the intrusion of the vane into the clay mass, tests were made in adjacent boreholes with four different thicknesses of vane blades; the results allowed an extrapolation of shear strength corresponding to a zero blade thickness. Two techniques of preparation of the boreholes for the pressuremeter tests were used and are discussed; the results were studied by two different methods, and they are shown to be very valuable provided that a proper technique is used to prepare the cavity for the test. A method is suggested for the interpretation of the penetrometer test results based on a relationship proposed by Vesic (1972) for the expansion of a cylindrical cavity. The practical implications of this study is discussed in the conclusion.

INTRODUCTION

The sensitivity of the clays of eastern Canada constitutes one of the main problems that have to be dealt with by engineers who have to design civil engineering works in that area. The clay referred to as "Leda clay" in the literature, but more appropriately called "Champlain clay" by some authors on the basis of geological considerations (Gadd 1960, La Rochelle et al 1970), was deposited during the late Pleistocene period in the post-glacial Champlain Sea which covered the Saint-Laurent lowlands and the Saguenay and Lac Saint-Jean area some 11,000 to 8,000 years ago.

One of the main characteristics of the Champlain clay resides in its fabric which has set with "chemical bonds" at the contacts between the clayey platelets (Crawford 1963, Conlon 1966, Townsend et al 1969, La Rochelle and Lefebvre 1971a) with the result that its structure is very brittle and extremely strain-sensitive. At the low confining stresses most often met in engineering problems, the major part of the shear stresses are resisted by the cementation bonds and, therefore, any disturbance causing a partial destruction of those bonds during the engineering investigation of the mechanical properties may result in a substantial underestimate of the shear strength.

It has been shown (La Rochelle and Lefebvre 1971a) that even with the most refined tube sampling techniques presently available, it is virtually impossible to preserve all the bonds in the clay samples; sampling may in fact result in a reduction of the undrained shear strength of the order of 50% of its full value (Coates and Mc Rostie 1963, Crawford 1963). Hence, the design engineer prefers to resort to one of the available methods of *in situ* measurements of the undrained shear strength. By and large the most widely used such method is the vane test; however, during the last few years, some work (Ladanyi and Eden 1969, Ladanyi 1972) has been done to try and make operational the use of pressuremeter and penetrometer in sensitive soils.

To assess the potentialities and limitations of these three *in situ* testing methods (vane, pressuremeter and penetrometer) comparative studies were made on two different deposits of sensitive clays which have been the sites of previous extensive studies. In the present paper, the results obtained during the comparative studies are analyzed and compared together with laboratory shear strength test results previously obtained on block samples.

SITES

For the purpose of the study, the sites of Saint-Vallier and Saint-Louis were chosen. These two sites are located in the Saint-Laurent lowlands, at a distance of 210 km apart, in areas which are considered to be very unstable following the high concentration of landslides observed in these regions. The description, geographical situation and geological conditions of the sites have been described in a previous paper (La Rochelle et al 1970). During the engineering investigations of the landslides, block samples of clay were taken at the bottom of trenches dug out in the craters of the landslides; laboratory tests were then made on block and tube samples and the results were compared together with the field vane test results.

The average physical properties of the clays from the two sites are given in Table I. That previous study reported by La Rochelle and Lefebvre (1971a and 1971b) thus allows a comparison between the compression tests results obtained on good quality samples and the *in situ* measurements of the undrained shear strength made during the present study by the vane, pressuremeter and penetrometer tests.

Soil Properties	St-Vallier	St-Louis
Water Content (%)	59	69
Plastic Limit (%)	23	27
Liquid Limit (%)	60	50
Plasticity Index (%)	37	23
Liquidity Index (%)	0.97	1.83
Clay Content (%)	65	79
Silt Content (%)	28	20
Sand Content (%)	7	1
Activity	0.57	0.29
Sensitivity		
by Field Vane	7	12
by Laboratory vane	20	50
Salt Content of the pore water (gram/liter)	4	0.40

Table I Physical Properties of the Clays Studied

FIELD VANE APPARATUS

It has been, and still is for many engineers, the usual practice to consider the profile of the undrained shear strength of sensitive clays as determined by the field vane tests to be the standard of reference for other shear strength tests. Indeed, when the results of laboratory compression tests made on tube samples did agree well with the field vane test results this agreement was interpreted as an evidence of the good quality of the samples. However, there are many reasons why the shear strength determined by the field vane should be appreciably lower than the values obtained by laboratory compression tests made on good quality samples; three of these reasons are discussed hereafter, i.e., the anisotropy of the clay deposit, the disturbance caused by the intrusion of the vane into the clay mass, and the progressive failure which may affect the strength measurements in a brittle material.

Anisotropy of the clay

It is not within the scope of the present paper to discuss at length the interpretation of the vane test in relation with the anisotropy of the soil. It may be sufficient to state that in the case of a vane apparatus, as discussed by many authors (see for example Lo 1965), where the blades have a ratio of height to diameter of 2.0, most of the applied torque is resisted by the shear strength mobilized on a vertical cylindrical surface along which the shear strain is in the horizontal direction. Consequently, taking into account the possible anisotropy of the clay, the vane tests should logically be compared with compression tests made on horizontal samples, i.e, samples where the applied major principal stress is in a horizontal plane relatively to the position of the soil element in the deposit.

To allow such a comparison, unconfined compression tests were made on horizontal specimens trimmed from the block samples taken on the two sites. The clay deposits on the two sites were found to be anisotropic as illustrated by the ratio of horizontal to vertical compression tests which was 0.73 in Saint-Vallier and 0.92 in Saint-Louis. However, when comparing compression test results on horizontal samples with the field vane results obtained at the level and close to the locations of the block samples, it was found (La Rochelle and Lefebvre 1971b) that the vane strength was only of the order of 70% of the strength measured by horizontal compression tests. Hence, even taking into account the anisotropy of the clay, the field vane tests give results which are appreciably lower than the compression tests on good quality samples of the sensitive Champlain clays.

Disturbance caused by the intrusion of the vane

One possible explanation for that difference may be the disturbance caused by the intrusion of the vane into the clay mass. La Rochelle and Lefebvre (1971a) have shown that a lateral strain of only 0.5 mm applied in an undrained state to Champlain clay samples of 3.7 cm diameter was sufficient to break the bonds in the clay and reduce the undrained strength to 70% of its undisturbed value; this may be attributed to the fact that cementation bonds of the Champlain clays are highly sensitive to strains. That observation did explain well the reason for the disturbance caused by the intrusion into the clay mass of a thin wall sampling tube which causes still much larger strains.

The same reasoning may apply to the intrusion into the clay mass of the vane blades normally used in sensitive clays, which have a thickness of the order of 1.9 mm. It would then seem reasonable to say that the thicker the blades are, the more disturbance they would produce. Moreover, the decrease of strength resulting from the intrusion of the vane may also be partly attributed to the induced increase in pore pressure in the clay surrounding the vane and which has not dissipated when the torque is applied shortly after the intrusion of the vane (Flaate 1966); this effect would similarly be influenced by the thickness of the blades.

In order to check this contention, field vane tests were made in adjacent boreholes with four vanes having the same over-all dimensions of 9.5 × 4.75 cm but with different blade thicknesses "e" of 1.6, 1.95, 3.1 and 4.7 mm. The N.G I. vane apparatus was used for the study and the testing procedure was kept very similar for all the boreholes. The torque was applied at a constant rate of strain of 0.1 degree per second. The boreholes were made in the slopes adjacent to the slide craters where the block samples were taken in order to allow a comparison to be made with the results obtained from laboratory tests on block samples.

The results of the vane tests made in Saint-Louis with four different blade thicknesses are given in figure 1. It may be seen that there is a decrease of the undrained shear strength with an increase in the blade thickness. The results of Saint-Vallier have shown the same tendency, although not as marked. In order to try and extrapolate the results for a zero blade thickness, the schema suggested by Cadling and Odenstad (1950) for the disturbance produced by the intrusion of the vane into the soil mass was considered. From that schema illustrated in figure 2, it was reasoned that the strength along the potential cylindrical failure surface would be mainly affected by the disturbed zones at the edges of the four blades and that the thicknesses of these zones would be a function of the thickness "e" of the blades. Therefore, a perimeter ratio α was

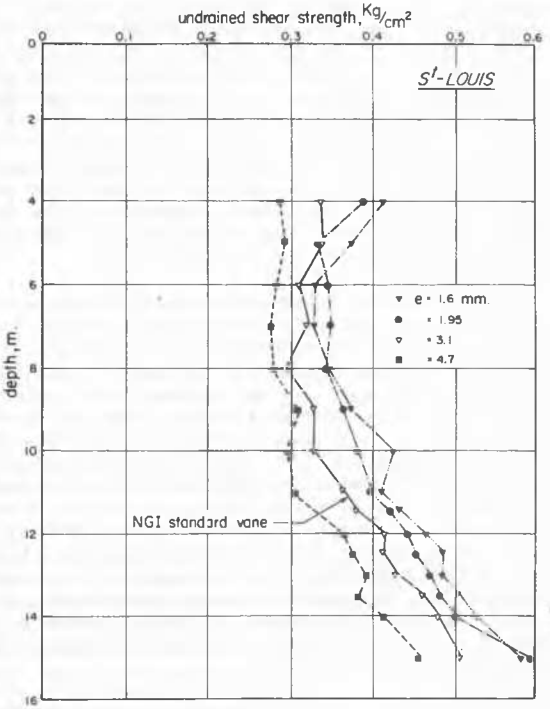

Figure 1- Results of Field Vane Tests with Different Blade Thicknesses

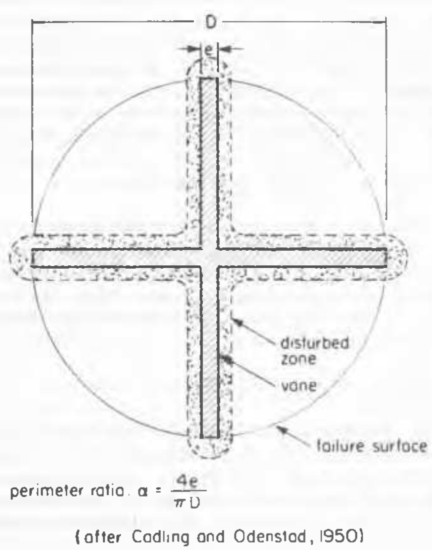

Figure 2- Disturbance Around the Vane Blades

defined as $\alpha = 4e/\pi D$, where D is the diameter of the vane. The shear strengths measured with the four different blades were then plotted against the perimeter ratio of the corresponding blade for all the elevations in the profile where measurements were taken; such plots obtained for three different depths in Saint-Louis are given in figure 3. It may be seen that there is a definite trend for the shear strength to increase with a decrease in the perimeter ratio. In very few cases, the scattering of the results made difficult the interpretation of the plots, especially for Saint-Vallier. However, in the great majority of the plots, a straight line representing the average variation of shear strength with perimeter ratio could easily be drawn although most of the results indicated that for blade thicknesses lower than the standard thickness (e = 1.95 mm), there was a tendency for an upward curvature in the relationship (see depths 4 m and 12.5m in figure 3). Therefore, it may be considered that the undrained shear strength c_{uo} extrapolated for a zero blade thickness is probably a minimum.

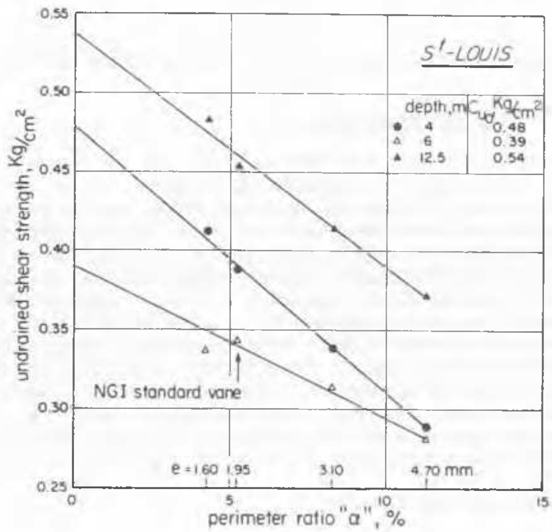

Figure 3- Extrapolation of Vane Strength for Zero Blade Thicknesses (c_{uo})

The profiles of the corrected undrained shear strength c_{uo} as a function of depth were obtained and compared with the results of the standard vane. The resulting average increase of strength was 15% in Saint-Louis (Fig.4) and 11% in Saint-Vallier. The fact that the disturbance effect was on the average larger in Saint-Louis than in Saint-Vallier may be due to the higher sensitivity of the clay in Saint-Louis (Table I). However, the corrected undrained shear strengths for both sites are still lower than the results of unconfined compression tests (U) made on horizontal specimens trimmed from the block samples. The remaining difference may be explained partly by the fact that the correction of the undrained shear strength obtained by the plots of figure 3 should be considered as minimum, but also by the possibility of progressive failure.

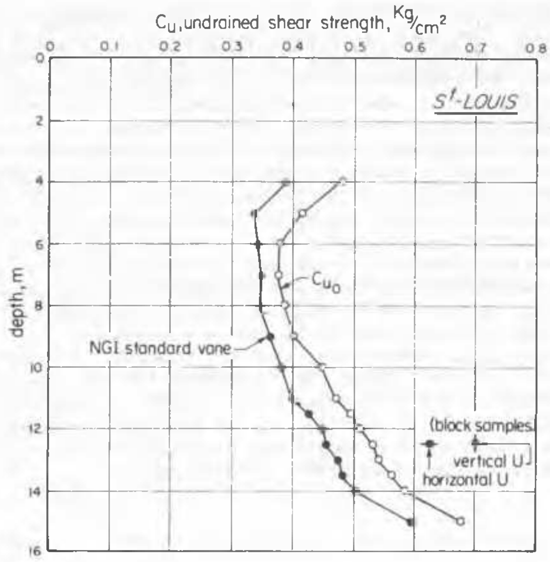

Figure 4- Comparison of NGI Standard Vane Strength with c_{u_0}

Progressive failure

Very little work has been made on this aspect of the failure around the cylindrical failure surface of the vane test. Cadling and Odenstad (1950) have attempted to analyse this phenomenon and have found no evidence of progressive failure around the vane failure surface in the Swedish clays. However, when dealing with cemented clays which are extremely brittle, there is little doubt that a progressive failure may be initiated at the edges of the blades where the clay is more intensely strained than in the rest of the shear zone. The strains required for the cementation bonds to be broken are so small that they could hardly be observed by the visual method used by Cadling and Odenstad (1950). More work will be required to evaluate this effect.

PRESSUREMETER TESTS

The pressuremeter test is one other method which allows the determination *in situ* of the undrained shear strength and gives also some data on the stress strain characteristics of the soils.

The apparatus used in the present study is a "pressiomètre Ménard" which has been described in details by Ménard (1957); it consists essentially in a probe lowered into a borehole and connected by plastic tubings to a pressure-volumeter instrument at the surface. The three-cell cylindrical probe is inflated by means of a gaz pressure and the change of volume of the central cell only is recorded in order to eliminate end effects. The pressure was increased by small steps lasting two minutes each and the volume changes were recorded at 30 sec., 1 min., and 2 min. The pressure increments were chosen small enough that at least eight points were obtained to trace the pressure-volume graph. The tests were made on the two sites in boreholes adjacent to the vane tests discussed and a type E probe was used.

Preparation of the cavity

The results obtained with the pressuremeter test will be greatly influenced by the quality of the soil at the surface of the cylindrical cavity in which the probe is inserted since the pressure-volume curve must include a linear part allowing the determination of the modulus of elasticity of the soil which is used in the calculation of the undrained shear strength. In the sensitive clays, if an appreciable thickness of the soil around the cavity is disturbed, no linear pressure-volume relationship will be obtained and the resulting calculations of the undrained shear strength will be very erratic. Hence, two different methods were used to prepare the cavity for the measurements.

In the first method, the borehole was carried out by continuous sampling by means of a fixed piston thin wall NGI sampler. The diameter of the cutting edge was 54 mm, the outside diameter of the tube 57 mm, and the thickness of the wall 1.3 mm. Although this technique may be expected to produce a fairly clean and straight hole, it involves two causes of disturbance. The first one is consequent to the change of volume produced by the intrusion of the sampler tube into the soil mass; since the angle of the cutting edge is on the outside of the tube, the change of volume will in part take place on the outside of the cutting edge and will produce a distorsion of the soil elements at the surface of the cavity; the second cause of disturbance consists in the stress release around the cavity, which is aggravated by the suction produced during the extraction of the sampler from the hole.

In an attempt to eliminate these two causes of disturbance, the second borehole was prepared by means of a specially designed motorized auger incorporating a system of bentonite injection. The auger consisted of two sharp cutting edges, 50 cm long and forming a cylinder of 50 mm diameter when rotated: at the lower end of the auger on the inside of the cutting edges, small rods were fixed to remould the clay which was then washed by the bentonite injected at that level through the center of the auger. The auger was motorized in such a way that its rate of progression down the borehole was kept constant at one centimeter per four revolutions. It is thought that this technique eliminated the two above-mentioned causes of disturbance; however, it is possible that some disturbance was produced at the immediate surface of the cavity by the rotation of the auger cutting edges.

As a matter of fact the results obtained by the first technique were found to be unusable for the determination of a proper value of the modulus of elasticity from the pressure-volume curves, while the results of tests obtained with the second technique are satisfactory as discussed below.

Analysis of test results

A typical pressure-volume curve obtained in an augered borehole in Saint-Louis is illustrated in figure 5. This loading diagram allows the calculation of the pressuremeter modulus E and of the undrained shear strength c_u. The undrained pressuremeter modulus E as defined by the theory of elasticity is given by the following expression:

$$E = \frac{2\ (1 + \nu)\ \Delta P}{\Delta V/\ (V_o + \Delta V)} \quad (1)$$

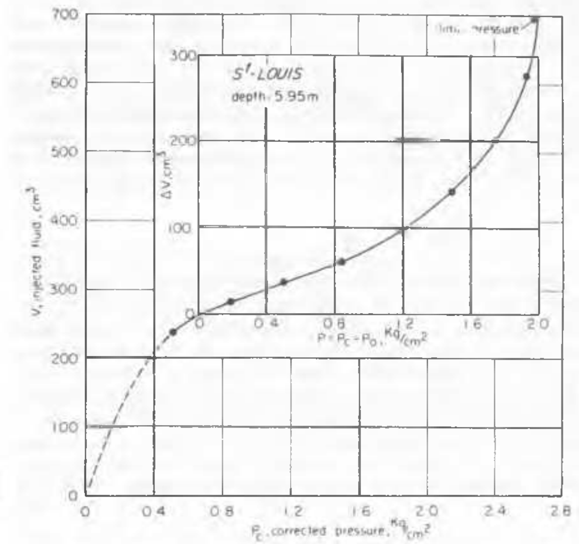

Figure 5- Pressure-volume Curve of Pressuremeter Test

where the Poisson's ratio ν is taken equal to 0.5, the changes of volume ΔV and of pressure ΔP are obtained during the pseudo-elastic phase of the test and computed on the linear portion of the curve (Fig.5). V_o is the volume of the probe determined at the point of the curve where the pressure has reached the value of the initial horizontal ground pressure P_o. The value of P_o is determined on the graph of figure 5 and corresponds to the pressure at the start of the linear portion of the curve.

Two methods were used to evaluate the undrained shear strength from pressuremeter tests. The first method proposed by Ménard (1957) is based on the limit pressure $P\ell$ which is the pressure acting in the cavity when the soil is in a state of plastic equilibrium. The value of $P\ell$ is given by the position of a vertical asymptote which may be drawn to the right of the curve in the graph of figure 5. Once the value of the limit pressure is known, the following equation suggested by Bishop, Hill and Mott (1945) may be solved by iteration to calculate the undrained shear strength:

$$P\ell - P_o = c_u \left[1 + \ell n \frac{E}{2(1+\nu)\, c_u}\right] \qquad (2)$$

The second method used was proposed by Ladanyi (1972) and was derived by combining the original theory of Gibson and Anderson (1961) with a strain-displacement relationship. This extended theory allows a direct determination of the stress-strain curve of the clay from a conventional pressuremeter test.

The results by both methods on the sites of Saint-Louis are plotted on figure 6 together with the standard field vane test results. It may be seen that both methods used to interpret the pressuremeter tests give strengths which are higher than the standard vane test results. In the case of Saint-Louis (Fig.6), the pressuremeter strengths are even higher than the unconfined compression test results obtained on vertical specimens trimmed from block samples. However, following a series of quick triaxial tests with varying

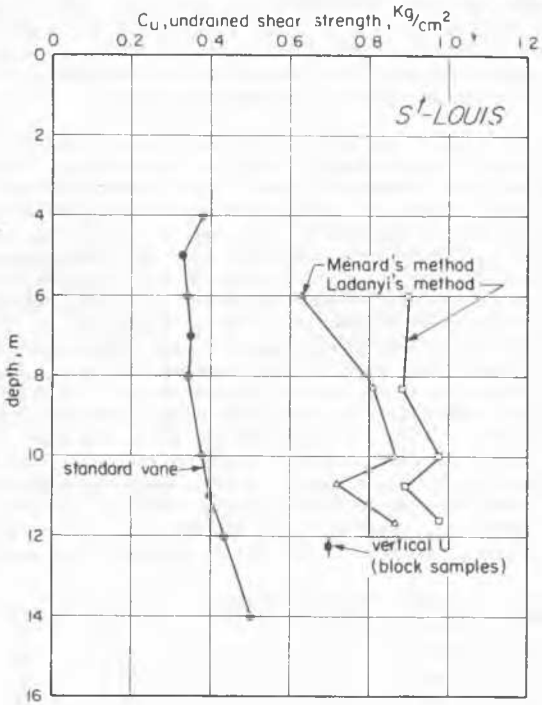

Figure 6- Comparison of Pressuremeter and Vane Strengths

confinement pressures made on Saint-Louis clay samples by Lefebvre (1970), it was observed that, at confinement pressures corresponding to the total overburden pressure, the undrained shear strengths obtained were 10% to 20% higher than the values given by unconfined compression tests. This fact could be explained by the tendency for the unconfined samples of very brittle clay to fail by vertical splitting, and also by the opening up and propagation of fissures. Referring again to figure 6, it may be observed that an increase of 10% to 20% of the vertical U strength would result in a very good agreement between the values obtained by the pressuremeter tests and those given by the confined vertical compression tests on good quality samples.

In Saint-Vallier, the pressuremeter results were lower than the vertical compression test results but closer to these than to the standard vane results. The fact that the pressuremeter results are lower than the vertical compression tests may be attributed to the difficulties which were encountered on the field during the preparation of the cavities for the pressuremeter tests. These difficulties resulted in some disturbance along the face of the cavity as evidenced by the shape of the pressure-volume curves obtained.

On both sites, Ladanyi's method gave values of undrained strengths which are higher than the values obtained by Ménard's method. This may be due to the fact that in the case of Ladanyi's method, the limit pressure is not used to calculate the peak strength of the soil but only to determine the residual strength; in this very brittle and sensitive clay, which flows when failed, it is possible that the value of the limit pressure is underestimated due to the progressive failure and plastic flow which may take place as the cylindrical cavity expands. As a matter of fact, the limit pressures measured in the cavities prepared by pre-sampling were on the average 19% lower than the values obtained in augered holes, thus indicating the possible influence of plastic flow on the limit pressure.

Table II gives the details of the results obtained during the pressuremeter tests in Saint-Louis. The values of c_u listed in Table II are those obtained by Ménard's method. It is immediately evident that the ratios E/c_u obtained are very low, being on the average 40 in Saint-Louis and 35 in Saint-Vallier, as compared to 370 for Saint-Louis and 430 for Saint-Vallier as determined by compression tests on block samples (La Rochelle and Lefebvre 1971a). Since the c_u values determined by the pressuremeter tests agree reasonably well with those given by the compression tests, it would seem that the moduli as determined by the pressuremeter tests are at fault. The moduli computed by Ladanyi's method are very similar since the average value of the E/c_u ratio is 31 in Saint-Louis. The low values of the pressuremeter moduli might be explained by some disturbance of the clay on the face of the borehole wall since no other explanation is available at present; more work will be required on this problem.

Depth (m)	(kg/cm^2) P_o	(kg/cm^2) P_ℓ	(kg/cm^2) E	(kg/cm^2) c_u	E / c_u
5.95	0.93	3.25	27.4	0.63	43.5
8.25	1.28	4.05	27.6	0.81	34.1
9.90	1.46	4.38	27.7	0.87	31.8
10.65	1.98	4.84	39.1	0.72	54.4
11.6	1.80	4.84	31.0	0.87	35.6
				average	40.0

Table II Pressuremeter Tests Results in Saint-Louis

STATIC PENETROMETER

Originally developed for the investigation of cohesionless deposits, the static penetrometer test is increasingly used to determine the undrained shear strength of sensitive clays. The main advantage of the test is to give a continuous recording of the penetration resistance and therefore to allow an analysis of the stratigraphy of the deposit. However, the main problem related to its use in clays has consisted in computing the undrained shear strength of the clay from the measured point resistance.

By extending the usual concepts of bearing capacity analysis to the static penetrometer it was first proposed to express the point resistance R_p as:

$$R_p = c_u N_c + \gamma D \qquad (3)$$

where γ is the total unit weight of the overburden soil of thickness D and N_c is the bearing capacity factor which depends only on the friction angle of the soil and is equal to 5.14 or 6.28 for $\phi = 0°$. Following the work of many investigators who attempted to verify this formula, values of N_c varying from 4 to 30 were suggested. Such variations could not be explained by the usual bearing capacity theory, and were indicative that the point resistance measured by the penetrometer test was a function not only of the undrained shear strength of the clay but also of other independent soils parameters.

It was first suggested by Ladanyi (1967) that the N_c coefficient was a function of the undrained compressibility and of the sensitivity of the clay; he then proposed an expression based on the expansion of a spherical cavity as being representative of the penetration phenomenon. Schmertmann (1969) has also suggested that R_p is a function of the undrained compressibility, but he correlated the penetration phenomenon with the expansion of a cylindrical cavity, i.e. the static penetration test with the pressuremeter test. Finally, Vesic (1967-1972) also shows that the correlations between the point resistance and the undrained shear strength is a function of the compressibility of the soil expressed by a rigidity index I_r equal to:

$$I_r = \frac{E}{2\,(1+\nu)\,c_u} \qquad (4)$$

In order to verify the similarity between the penetration problem and the expansion of a spherical or cylindrical cavity, full scale laboratory tests were made on a synthetic material reproducing the mechanical properties of the Champlain clays (Tavenas & Chapeau, 1972). A semi-cylindrical penetrometer, 1½" in diameter, was pushed along a glass wall in that material and the failure pattern observed. A typical result is presented on figure 7. The displacements field is entirely different of any of the pattern corresponding to the usual

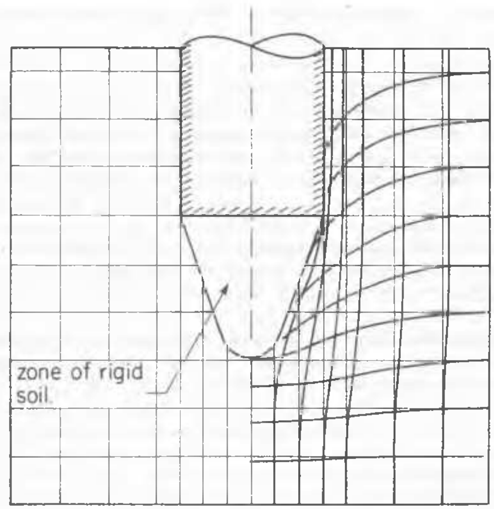

Figure 7- Displacements Field Around Penetrometer Point

bearing capacity theories; however, it is very close to the failure pattern suggested by Vesic (1967) for a penetrometer driven into a friction material; this would suggest that, even though the test is quick and should be considered as undrained, the effective parameters of the soil may have an influence. It is also remarkable that the general deformation pattern is finally somewhat intermediate between that produced by the expansion of a cylindrical and of a spherical cavity. In any case, the methods of interpreting the penetrometer test on the basis of the cavity expansion theory should be the most satisfactory.

Two series of static penetrometer tests were made at Saint-Louis and Saint-Vallier. The equipment used was built at Laval University, but is in principle very similar to the Fugro electrical penetrometer (De Ruiter

1970). Fig.8 gives the typical point resistance profile for Saint-Louis. The R_p/c_u ratios obtained on both sites are very similar and of the order of 9 with the standard vane shear strength and 5.5 with the vertical unconfined compression strength on block samples. However, the ratio with the pressuremeter shear strength is 4.1 in Saint-Louis and 6.4 in Saint-Vallier; this difference is a further indication that the pressuremeter results obtained in Saint-Vallier were not satisfactory.

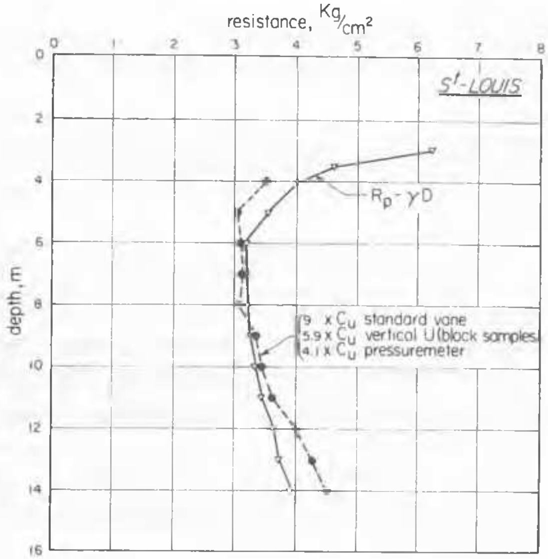

Figure 8- Typical Point Resistance Profile Obtained at Saint-Louis

As discussed above, the behaviour of the soil mass appears to be similar in both the static penetration test and the pressuremeter test. In such case the point resistance R_p and the limit pressure $P\ell$ should be related to the undrained shear strength c_u by the same factor. The theoretical expression of $P\ell/c_u$ is given as a cylindrical cavity expansion factor F_c' by Vesic (1972). F_c' is a function of the rigidity index I_r of the soil as shown on figure 9. Also plotted on the same figure are the values of R_p/c_u as computed for both sites from the E/c_u and c_u values determined by pressuremeter tests and by unconfined compression tests.

The pressuremeter tests leads to R_p/c_u vs I_r correlations different from the theory. Since it was shown before that the undrained shear strength obtained by the pressuremeter has the right order of magnitude, it may be concluded again that the rigidity index, and particularly the modulus E measured in that test is too low by a factor of about 10, which confirms the previous remark.

The R_p/c_u ratio on rigidity indexes determined in unconfined compression tests on block samples are in perfect agreement with the theoretical values proposed by Vesic (1972). Since the block samples may be considered as representative of the *in situ* properties of the Champlain clay, it must be concluded that the penetrometer test can be interpreted as a cylindrical cavity expansion test, or that the R_p/c_u ratio can be deduced from Vesic (1972).

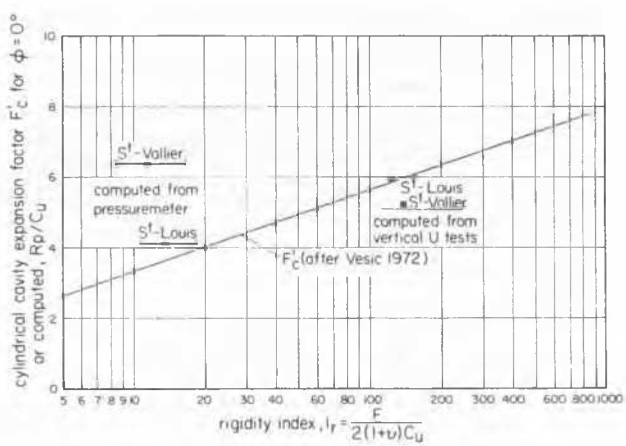

Figure 9- Relation Between N_c and the Rigidity Index

To compute the undrained shear strength of a clay from the point resistance measured by a static penetrometer it is therefore necessary but sufficient to know the rigidity index of that clay. As shown above, I_r can be best determined by unconfined compression tests on block samples; this could be a major problem since such samples are very difficutt to get. However, La Rochelle and Lefebvre (1971a) have shown that the E/c_u ratio is practically independant of the disturbance during normal sampling. Therefore the rididity index may be determined by unconfined compression tests on thin-wall tube samples of usual quality.

CONCLUSION

From the comparative study of the field strength test methods (vane, pressuremeter and penetrometer) together with the laboratory tests made on good quality samples, the following conclusions may be suggested.

The pressuremeter seems to be the only field test method allowing the proper determination of the undrained shear strength of the sensitive clays without the help of other tests. The undrained strengths obtained by that method are in full agreement with the confined compression tests made in the laboratory on vertical specimens trimmed from block samples provided that sufficient care is taken in the preparation of the cavity in which the pressuremeter probe is to be inflated. However, the undrained moduli of deformation measured by pressuremeter tests are much too low and are comparable to the drained moduli.

The penetrometer may also be used with some advantages to determine the undrained shear strengths of sensitive clays if the proper values of the ratio E/c_u are available. This ratio may be determined in the laboratory by unconfined compression tests made on clay specimens sampled by the usual techniques recommended for sensitive clays, as for example the fixed piston thin wall sampling apparatus. Once the E/c_u ratio is available, the relationship proposed by Vesic (1972) for the expansion of a cylindrical cavity may be used to compute the values of c_u from the point resistance data of the penetrometer. This apparatus is then an ideal method to complement an investigation of a site where minimum sampling has been made.

Of the three proposed field strength test methods, the vane test is the only one which seems to grossly underestimate the *in situ* shear strength of clays when compared to compression tests on good quality samples; this underestimate is due mainly to the influence of the anisotropy of the clay deposits, to the disturbance resulting from the intrusion of the vane into the clay mass, and also possibly to the progressive failure which may affect the vane results.

However, the authors do not deny that the vane test remains an extremely useful tool in the engineering practice on clay deposits; nevertheless it should be realized that the agreement found to exist between vane strength and observed full scale failures of slopes or fills, is no proof of the validity of the vane measurements but it only shows that our approach to stability analysis overlooks important factors. Once more, the soil engineer, when using vane tests, is right for the wrong reason. Obviously even if the pressuremeter and penetrometer tests yield undrained shear strength values which may be considered to be representative of the *in situ* strength of the clay, these values could not be used in combination with the presently available methods of stability analysis without risks of mishaps. Until our approach to stability analysis is better adapted to reality, the engineer will have in many cases to remain satisfied with the vane tests.

ACKNOWLEDGMENTS

The present study was made at the Geotechnical Section of Laval University under research grants from the National Research Council of Canada and the Quebec Ministry of Education. Some of the experimental results were extracted from the M.Sc. thesis of Messrs Jean-Pierre Debaire and Serge Leroueil.

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