Comparison of undrained shear strentgh results obtained by different test methods in soft clays

Eden, W. J.; Law, K. T.

Publisher's version / Version de l'éditeur:

Canadian Geotechnical Journal, 17, 3, pp. 369-381, 1980-08

National Research Council Canada
Conseil national de recherches Canada

COMPARISON OF UNDRAINED SHEAR STRENGTH RESULTS OBTAINED BY DIFFERENT TEST METHODS IN SOFT CLAYS

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

Reprinted from
Canadian Geotechnical Journal
Vol. 17, No. 3, August 1980
p. 369–381

DBR Paper No. 913
Division of Building Research

Price $1.00 OTTAWA NRCC 18422

Publications of the Division may be obtained by mailing the appropriate remittance (a Bank, Express, or Post Office Money Order, or a cheque, made payable to the Receiver General of Canada, credit NRC) to the National Research Council of Canada, Ottawa. K1A 0R6. Stamps are not acceptable.

A list of all publications of the Division is available and may be obtained from the Publications Section, Division of Building Research, National Research Council of Canada, Ottawa. K1A 0R6.

Comparison of undrained shear strength results obtained by different test methods in soft clays[1]

W. J. EDEN AND K. T. LAW

Geotechnical Section, Division of Building Research, National Research Council of Canada, Ottawa, Ont., Canada K1A 0R6

Received August 15, 1979

Accepted April 11, 1980

The Cambridge self-boring pressuremeter and the Norwegian Geotechnical Institute field vane apparatus have been used at three sites of different subsoils. Two of the sites, South Gloucester and National Research Council of Canada grounds, are located in Leda clay in the Ottawa area. The third site features a highly plastic lacustrine clay in northwestern Quebec. At this site tests were conducted beside and under a fill that had undergone 1 m of surface settlement. At all three sites, the pressuremeter yields higher strengths than the field vane apparatus.

Conventional and special laboratory tests were conducted on large diameter tube or block samples from South Gloucester. It is found that the conventional triaxial compressive strength is intermediate between the pressuremeter and field vane strengths. In accounting for the strength discrepancy, factors such as anisotropy, stress path, and disturbance have been studied in the light of the results of the special tests.

Le pressiomètre autoforeur de Cambridge et le scissomètre de chantier NGI ont été mis en oeuvre sur trois sites dans des matériaux différents. Deux des sites, Gloucester Sud et le terrain du Conseil national des recherches du Canada sont constitués d'argile Leda, dans la région d'Ottawa. Sur le troisième site, dans le nord ouest du Québec, on trouve une argile lacustre à forte plasticité. Sur ce dernier site, des essais parallèles ont été réalisés à proximité et sous un remblai qui avait subi 1 m de tassement. Sur les trois sites, le pressiomètre donne des résistances plus élevées que le scissomètre de chantier.

Des essais de laboratoires conventionels et spéciaux ont été réalisés sur des échantillons de grand diamètre ou sur des blocs, provenant du site de Gloucester Sud. On a noté que la résistance en compression triaxiale conventionnelle est intermédiaire aux résistances mesurées au scissomètre et au pressiomètre. Compte tenu de ces différences, des facteurs tels que l'anisotropie, le cheminement des contraintes et le remaniement ont été étudiés à l'aide des résultats des essais spéciaux.

[Traduit par la revue]

Can. Geotech. J., 17, 369–381 (1980)

Introduction

The use of the pressuremeter as an *in situ* testing tool has become increasingly popular in recent years because of its potential for obtaining stress–strain relationships and *in situ* horizontal pressures. To date (1979), pressuremeter tests have been conducted over many parts of the world, both on land and offshore, and in soils ranging from soft sensitive clay to soft rock and frozen soil. For about 20 years the test has been used throughout France for foundation design (Baguelin *et al.* 1978). With the addition of the self-boring device (Baguelin *et al.* 1972; Wroth and Hughes 1973), the problem of disturbance prior to testing is greatly reduced, particularly in soft soils. Since then many tests have been carried out in the Scandinavian countries, U.K., U.S.A., and Canada.

Based on test results recorded in the literature, there is a general observation that the pressuremeter test gives a strength quite different from those of other tests. Baguelin *et al.* (1972) report that the pressuremeter strength is 50% higher than the *in situ* vane strength for the Saint-Andre-De-Cubzak clay. Results for a soft Gothenberg clay presented by Wroth and Hughes (1974) show that as the clay approaches the normally consolidated condition with depth the pressuremeter strength is increasingly higher than the vane strength. Pressuremeter tests on a glacial till in the Glasgow district (McKinlay and Anderson 1975) yield strengths about 70% higher than the triaxial strength. A study on two stiff clays by Windle and Wroth (1977) gives contrasting results. Pressuremeter strengths for Gault clay agree closely with those from triaxial and Dutch cone tests whereas for London clay the pressuremeter strength is higher than the triaxial, cone, and plate loading tests.

Thus far the explanation for the discrepancy observed was that less disturbance was involved with the pressuremeter, particularly the self-boring type, than with other tests and hence the higher strengths were understandable. It should be noted, however, that other tests, the field vane in particular, already give too high values of undrained strength when applied to well-documented embankment failures.

[1]Presented at the 32nd Canadian Geotechnical Conference, Quebec City, P.Q., Sept. 26–28, 1979.

0008-3674/80/030369-13$01.00/0

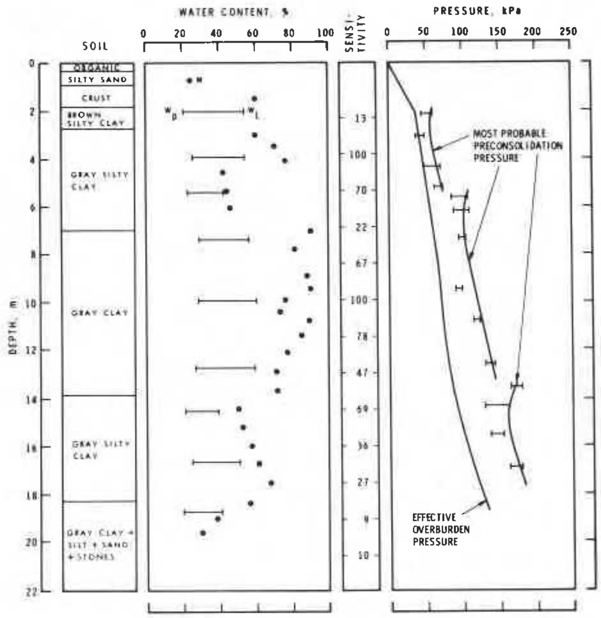

FIG. 1. Geotechnical profile at South Gloucester.

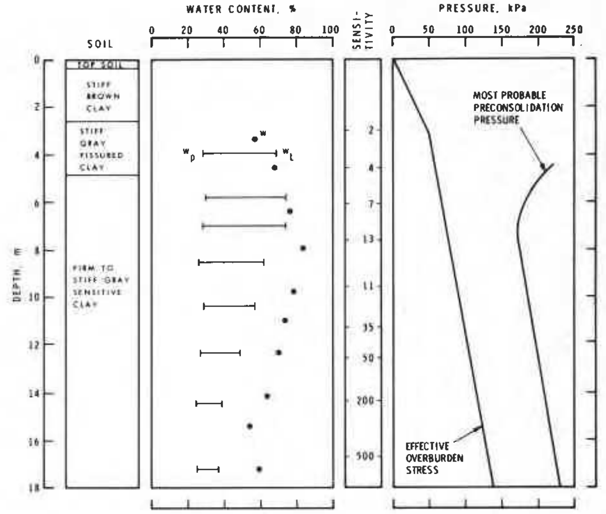

FIG. 2. Geotechnical profile at National Research Council of Canada, Ottawa.

Bjerrum (1972) has suggested a "corrected vane shear strength" depending on the plastic index of the soil. Justification for the correction has been attributed to: (a) effects of rate of loading; (b) anisotropic effects; and (c) progressive failure considerations.

At the three sites noted in this paper, the undrained strength obtained with the Cambridge self-boring pressuremeter was found to be consistently higher than that obtained by the Norwegian Geotechnical Institute (NGI) field vane test. To gain some insight into this discrepancy a systematic study using both field and laboratory methods has been conducted.

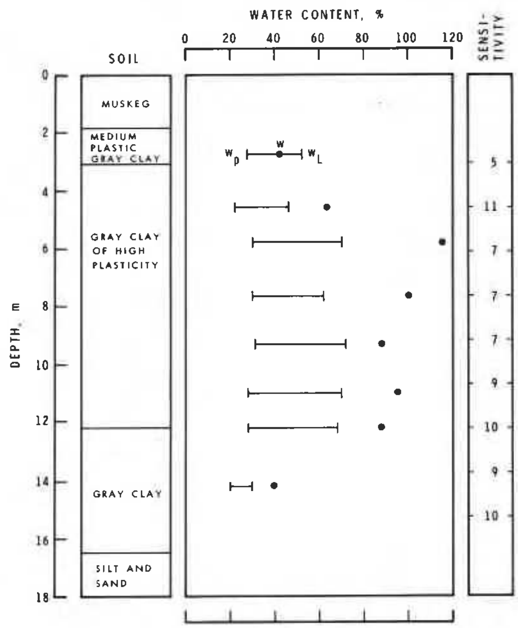

FIG. 3. Soil profile at Matagami, Quebec. (Data from Dascal *et al.* 1972.)

Soil Conditions

The three sites considered herein are: South Gloucester, an open field within the National Research Council of Canada (NRCC) grounds, and Matagami, Quebec. The first two sites are of Leda clay in the Ottawa area; their geotechnical profiles are shown in Figs. 1 and 2. The subsoil at the South Gloucester site is composed mainly of a highly sensitive soft marine deposit formed in various stages of deposition, erosion, and redeposition cycles (Bozozuk 1972). It is lightly overconsolidated with an overconsolidation ratio of about 1.5. The subsoil at the NRCC site is of similar origin but with a higher strength because of the higher preconsolidation pressure (Eden and Hamilton 1956). The clay there is fissured to a depth of about 7 m, below which the pressuremeter tests described in this paper have been carried out. The third site features a highly plastic lacustrine deposit of postglacial origin. A test embankment was constructed at this site 6 years before the present test series was conducted at 24 m from the toe and under the centre of the embankment, which had undergone 1 m of settlement. The soil profile shown in Fig. 3 is based on the work of Dascal *et al.* (1972) who investigated an embankment failure at a nearby site.

Test Description

Norwegian Geotechnical Institute (NGI) Vane Test

The NGI vane apparatus is fully described by Andresen and Bjerrum (1956). The outstanding feature of the apparatus is that the torque rod is completely isolated from the soil so that a soil–rod friction correction is eliminated. Tests at all three sites were conducted at $\frac{1}{2}$ m intervals.

The conventional method for interpreting the vane test was used. It assumes a cylindrical failure surface and a uniform distribution of shear strength yielding the following equation for strength, S_u:

$$[1] \quad S_u = 6T/7\pi D^3$$

where T is the measured torque; and D is the vane diameter, which is half the vane height.

Cambridge Self-boring Pressuremeter Test

The basic principle and operation of the Cambridge self-boring pressuremeter are described by Wroth and Hughes (1974). From the manufacturer, Cambridge *Insitu*, an 80 mm diameter pressure expansion cell with the lead-in cable and supply tube was obtained. The pressure supply panel and electronic readout equipment for the strain gauge systems were also acquired. Initially two X–Y recorders were used to obtain continuous records of total and effective pressure versus strain.

The original cutting shoe on the pressuremeter was replaced with removable hardened tool steel shoes so that the cutting shoe diameter would match the probe diameter with the membrane in place. Law and Eden (1980) have shown that a slight difference in the diameter of membrane and that of the cutting shoe can significantly affect the results obtained.

The equipment was adapted to a hydraulic-powered truck-mounted drill rig by the Geotechnical Section of the Division of Building Research. Figures 4 and 5 illustrate the probe adapted to the drill rig. Beginning at the top of the apparatus, Part A is a lightweight water swivel to supply water, by a small piston pump, to the inner rods, which rotate the cutter head. Part B is an auxiliary drive to rotate the inner rods. It consists of a hydraulic motor powering a roller chain drive and is mounted on top of the outer EWX casing. Oil supply for the hydraulic motor is supplied from the main hydraulic pump on the drill rig and is metered through a flow divider and on–off control valve. A constant speed of rotation of about 70 rpm was used. Part C is a chuck that holds the EWX casing. The downward advance of the pressuremeter was controlled by the 1.5 m long hydraulic feed cylinders on the drill rig. The rate of advance was maintained constant by restricting the flow of hydraulic fluid to the cylinders through a needle valve.

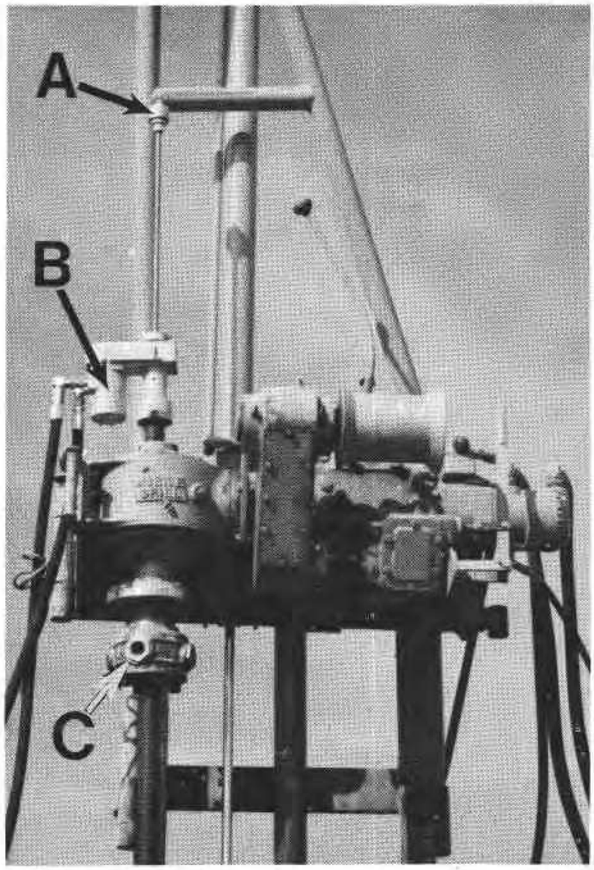

FIG. 4. Details for adapting the probe to the drill rig: A = water swivel; B = auxiliary drive; and C = chuck for the EWX casing.

Part D (Fig. 5) is a T-connection to which the drainage hose is connected to drain the cuttings and wash water away from the drill hole. When extension rods are added the top section remains in place in the chuck and is raised. The extension sections are added below the T-connection.

The drilling sequence is as follows. A 0.15 m hole is augered to 1.5 m above the test location. The hole is filled with water. The probe is positioned over the hole and the porous stones on the pore pressure sensors are saturated using a hypodermic needle. The probe is immediately lowered in the hole until the pore pressure sensors are submerged and a set of zero readings is taken. The probe is then lowered to the bottom of the augered hole and coupled to the chuck. The inner rods are then positioned in the upper chuck so that the cutter blades are set from 6 to 13 mm above the cutting edge of the shoe. Water is

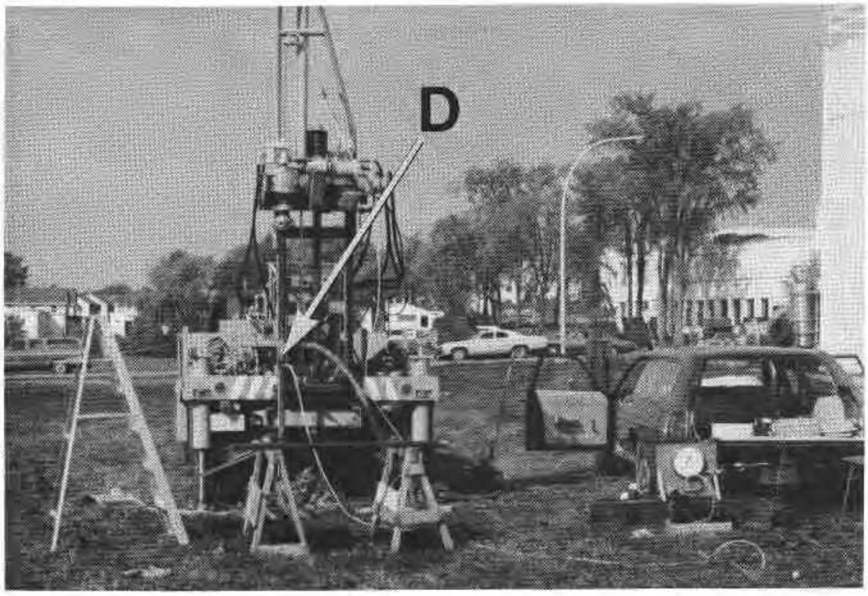

FIG. 5. Probe mounted on the drill rig. D = T-connection for the drainage hose.

supplied to the inner rods and, after flow is established, rotation of the inner rods begins. The probe is then advanced at about 0.1 m/min. Observations are maintained on the return wash water to make sure no restriction develops. The effective pressure and strain sensors are monitored to check against a malfunction. Efforts are made to maintain a steady rate of advance avoiding a stoppage, which would allow the clay to grip the membrane. To date no serious difficulties have been encountered in advancing the probe.

During the first few field trials a great deal of difficulty was encountered in maintaining balance on the *X–Y* recorders under field conditions. Because the recorders are so sensitive to external vibrations of the service vehicle and to small variations of power supply from a portable generator the records obtained were of questionable value for quantitative purposes, but they did provide a continuous record of the progress of the test.

The replacement method was to monitor the three circuits with the digital voltmeter, then calculate the stress–strain characteristics directly from the readings using a programmable hand calculator as the test progressed. Subsequently a Kaye System 8000 data logger was used to record the three measurements every minute just prior to the application of pressure increments of 9.8 kPa. This rate of pressure application was also used by Wroth and Hughes (1974) for a number of soft clays. The data logger yielded a printed record of good accuracy, which could be used in subsequent calculations. Figure 6 illustrates the monitoring equipment as used in a covered service truck.

The procedure used with the programmable calculator for obtaining the shear stress from the pressuremeter test results follows Ladanyi's (1972) method, which is in principle identical to those of Baguelin *et al.* (1972) and of Palmer (1972).

Laboratory Tests

The laboratory tests on South Gloucester clay were conducted with two aims: (1) to obtain conventional triaxial undrained shear strength for comparison with the *in situ* test strengths; and (2) to study factors influencing the measured shear strengths. The different test series and sample types are listed in Table 1. To achieve the first aim, conventional triaxial undrained compression tests (Series 1) were done. The samples were consolidated to pressures approximately equal to the *in situ* stress conditions before being subjected to undrained shear at about 2% axial strain per hour. To achieve the second aim, special tests were carried out on the soft gray silty clay samples from depths between 3 and 5 m.

There were two groups of special tests. The first was designed to study undrained strength anisotropy and the second to examine the effect of different stress paths followed in the laboratory and *in situ* tests.

Three methods have been employed here in measuring undrained strength anisotropy. The first two made use of the triaxial machine and the third the

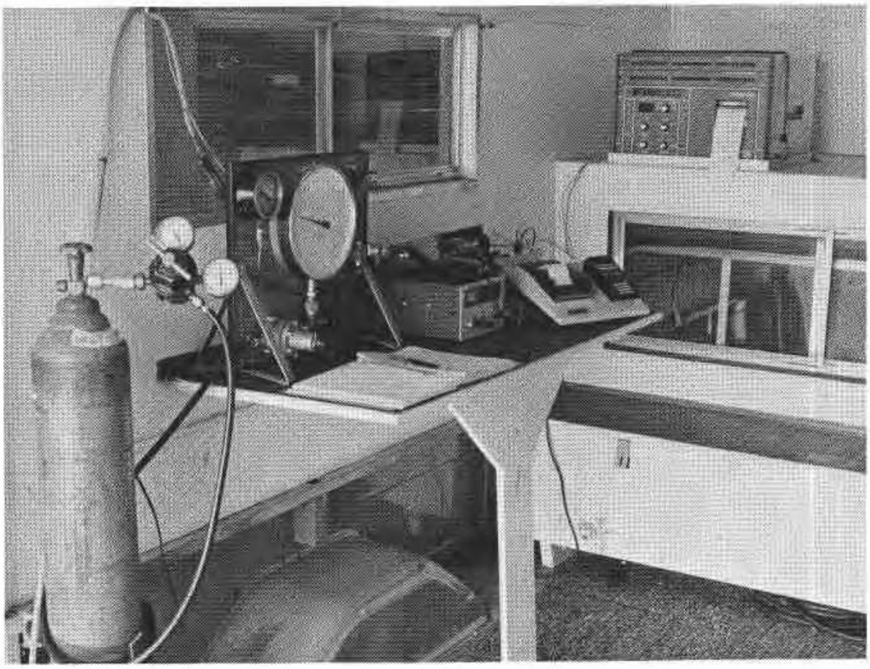

FIG. 6. Monitoring equipment for the probe.

TABLE 1. Summary of laboratory test series

Test series	Description	Sample type
1	Conventional triaxial test on vertical specimens from various depths	A
2	Triaxial compression tests on specimens trimmed at various angles	B
3	Triaxial compression and extension tests on vertical specimens	A
4	Triaxial-vane tests	A
5	Conventional triaxial tests along with controlled stress path plane strain tests	A

NOTES: A = 124 mm diameter Osterberg samples; B = block samples.

triaxial-vane apparatus. The first method was to test specimens (Series 2) trimmed at various angles from undisturbed block samples. The specimens were first consolidated to the *in situ* pressure before the onset of undrained shear. Throughout this test series, an isotropic consolidation pressure equal to the effective overburden pressure was applied as the coefficient of earth pressure at rest, K_0, was found close to 1.0 using the hydraulic fracture technique (Bozozuk 1974). By comparing strengths of the specimens at various angles, a measure of anisotropy was obtained. This method has been used by a number of researchers (e.g., Duncan and Seed 1966; Lo and Milligan 1967; DeLory and Lai 1971).

The second method compares strengths from triaxial compression and extension tests on vertical specimens (Series 3). The specimens were again consolidated to the effective overburden pressure followed by undrained shear, one via compression and the other extension. The compression test was a standard test of compressing the specimen and the extension test of pulling the specimen. It should be noted that this manner of conducting the extension test is equivalent to maintaining constant vertical pressure and increasing the horizontal pressure (Law and Holtz 1978). This approach has been advocated by Ladd (1965) and Bjerrum (1972).

The third method using the triaxial-vane apparatus was described in detail by Law (1979). Briefly, vane tests were conducted in triaxial specimens (Series 4) consolidated to the desired pressure. Undrained strength anisotropy can be studied by measuring the vane strengths on the vertical and the horizontal planes at the *in situ* effective pressures. These strengths, denoted by S_{v0} and S_{h0}, respectively, can be deduced by conducting three triaxial-vane tests: (1) under an isotropic consolidation pressure equal to the *in situ* effective horizontal pressure, σ_{h0}'; (2) under an anisotropic consolidation pressure system with σ_{h0}' in the horizontal direction and σ_{vf}' in the vertical direction with σ_{vf}' being a pressure greater than the preconsolidation pressure; and (3) under an isotropic consolidation pressure equal to σ_{vf}'. Denoting the torques obtained in these three tests by T_a, T_b, and T_c, respectively, it could be shown that

$$[2] \quad S_{v0} = \frac{2(T_b - 3T_c/10\alpha)}{\pi D^3\left(\alpha - \frac{3}{10}\right)}$$

and

$$[3] \quad S_{h0} = \frac{2}{\pi D^3}\left[\frac{10}{3}(T_a - T_b) + T_c/\alpha\right]$$

where

$$[4] \quad \alpha = (10H + 3D)/10D$$

and D and H are the diameter and height of the vane.

It is appropriate to point out here that the strength anisotropy measured in the different methods will not be the same, as there are many operating factors. The important ones are soil structure, geological stress history, and stress path leading to failure. These factors are closely interrelated and exert varying influences in different tests or in different soils. Soil structure that relates to particle orientation or fabric may have a dominant effect in the tests on inclined specimens. The stress paths employed in the compression and extension tests generate different pore pressures at failure and hence produce unequal undrained strengths (Law and Holtz 1978) even for an isotropic material. The triaxial vane measures strengths on the horizontal and vertical planes that are generally different from the triaxial tests. The effect of geological stress history and soil structure may therefore play an important role. The purpose for conducting these tests was not to study anisotropy per se but to obtain an indication of its effect in understanding the strength difference measured in the various tests.

The second group of special tests (Series 5) was conducted on soil specimens using a plane strain cell (Bozozuk 1972) and a triaxial cell. The plane strain cell is similar in principle to the triaxial cell except that side blocks are mounted to maintain zero strain and to measure stress in the intermediate principal stress direction. In order to minimize sample variability, four specimens, each 80 mm high, were trimmed from a single section of the Osterberg sample shown in Fig. 7. Two were horizontal (36.5 mm by 36.5 mm cross section) for the plane strain tests and the other two (36.5 mm diameter) were vertical for the triaxial tests.

The triaxial test was a consolidated undrained test

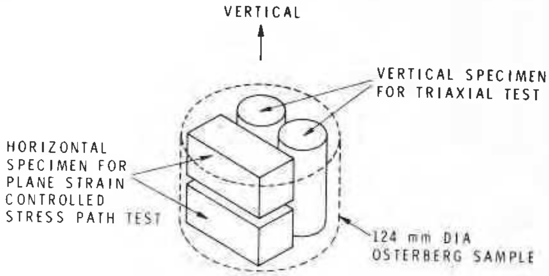

FIG. 7. Specimen arrangement for triaxial and plane strain tests.

at a strain rate and consolidation pressure equal to those in the plane strain test. The soil specimen was mounted in the plane strain apparatus in such a way that the *in situ* vertical direction was subject to zero strain (intermediate principal stress direction) during shearing. This enables the laboratory simulation of a plane strain condition along the vertical direction during the *in situ* pressuremeter tests. Furthermore, the effective stress path during the pressuremeter test was estimated based on the field test results and reproduced in the laboratory plane strain test. This requires a control of the stress path during shearing in the laboratory test. Such a control has been possible with the use of a sophisticated system, which comprises, besides the plane strain cell, an electric-pneumatic transducer and a mini-computer. The pressure supplied to the plane strain cell is regulated by the electric–pneumatic transducer, which is in turn controlled by the computer. By installing the appropriate software in the computer, any desired stress path can be prescribed. Further details of this system will be reported elsewhere (Law 1980).

By examining results from the controlled stress path plane strain test and triaxial test, factors contributing to the strength discrepancy between the pressuremeter and conventional triaxial tests can be studied.

Test Results and Discussion

Strength Comparison

The results of pressuremeter and vane tests at the NRCC site are shown in Fig. 8. For this firm to stiff Leda clay, the pressuremeter strength exceeds the field vane by about 20–40% (Table 2).

A typical pressure–strain plot from the pressuremeter test is shown in Fig. 9*a*.

Figures 10 and 11 show the *in situ* test results beside and under the embankment at Matagami site. The pressuremeter strengths are again higher than the vane strength with the former exceeding the latter by about 60–110% beside the embankment, and

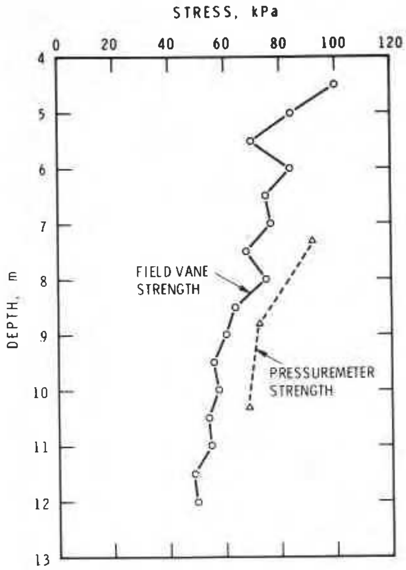

FIG. 8. *In situ* strength profile at National Research Council of Canada site.

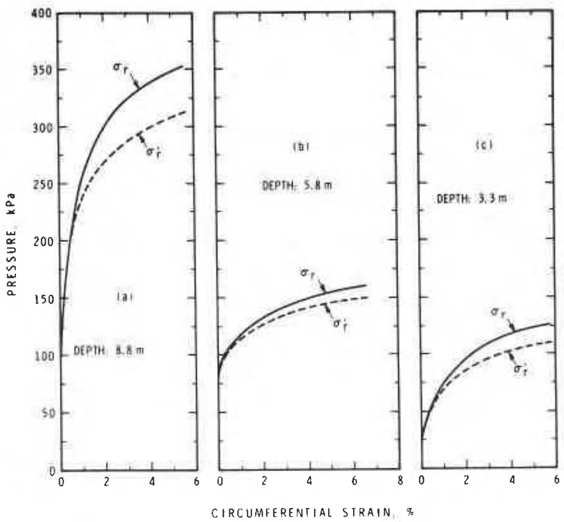

FIG. 9. Typical pressure–strain curves from pressuremeter tests: (*a*) National Research Council of Canada site; (*b*) Matagami site; (*c*) South Gloucester site.

about 25–110% under it (Table 3). Figure 9*b* shows a typical pressure–strain curve from a pressuremeter test at the site.

The pressuremeter, field vane, and conventional triaxial (Series 1) strengths for South Gloucester clay are compared in Fig. 12. Throughout the depth studied, the pressuremeter yields the highest strength, the triaxial ranks second, and the vane gives the lowest strength. The average relative strength values

TABLE 2. Results of *in situ* tests at National Research Council of Canada site

Depth (m) (1)	Strength (kPa) Pressuremeter (2)	Vane (3)	Strength ratio (4) = (2)/(3)
7.3	92.2	73.5	1.25
8.8	72.6	61.8	1.17
10.3	78.5	55.0	1.43

TABLE 3. Results of *in situ* tests at Matagami site

Depth (m) (1)	Strength (kPa) Pressuremeter (2)	Vane (3)	Strength ratio (4) = (2)/(3)
	Under the embankment		
3.1	32.4	—	—
4.7	29.4	23.5	1.25
6.4	38.2	25.5	1.50
8.0	61.8	29.4	2.10
	24 m from embankment toe		
4.3	23.5	14.2	1.66
5.8	25.5	16.0	1.59
7.3	36.3	21.6	1.68
8.9	52.0	25.0	2.08

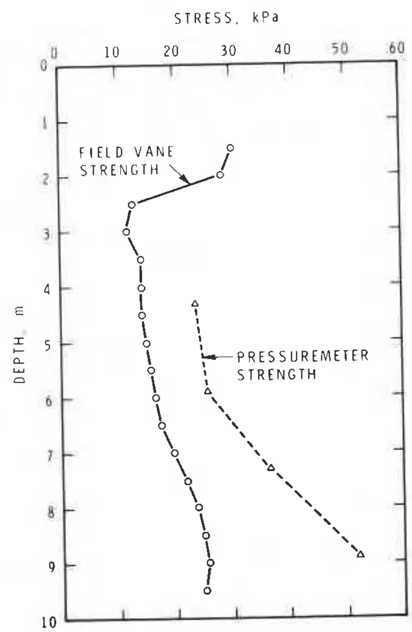

FIG. 10. *In situ* strength profiles beside the embankment at Matagami.

are compared in Table 4. The pressuremeter strength is about 17% higher and the field vane strength is about 35% lower than the triaxial strength and the

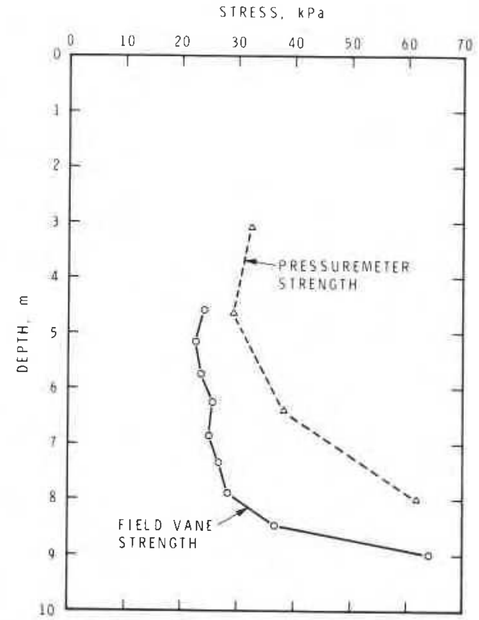

FIG. 11. *In situ* strength profile under the embankment at Matagami.

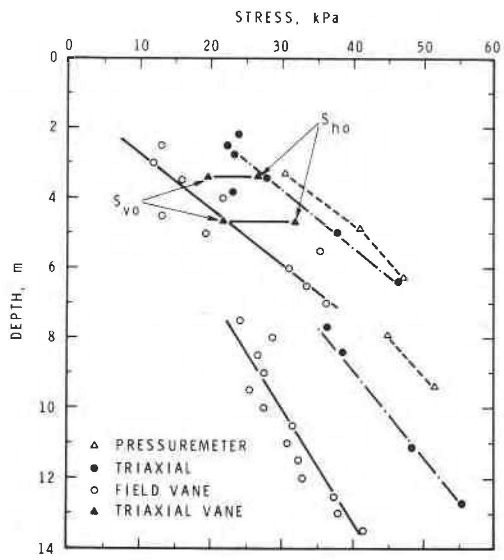

FIG. 12. Strength profiles at South Gloucester site.

pressuremeter strength is 85% in excess of the vane strength.

Figure 9*c* shows a typical pressure–strain curve from the pressuremeter test at South Gloucester. The shear stress – strain curves from the pressuremeter and the triaxial tests for two depths are shown in Fig. 13. The maximum shear strain, γ, is used for both tests for convenience of comparison. This strain

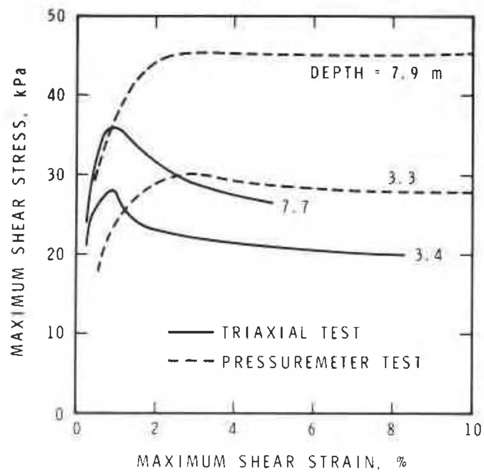

FIG. 13. Stress–strain curves from pressuremeter and triaxial tests.

TABLE 4. Strength comparison at South Gloucester site

Depth (m)	Strength ratio		
	Pressuremeter/ triaxial	Vane/ triaxial	Pressuremeter/ vane
2.0–6.5	1.12	0.60	1.88
6.5–12	1.22	0.67	1.82

TABLE 5. Results of triaxial compression tests on specimens trimmed from various angles *i*, from block samples (Series 2)

i	Natural moisture content (%)	Failure strain (%)	A_f	Peak strength (kPa)	Strength ratio
0	70.3	1.9	0.45	22.8	1.00
30	71.6	1.9	0.58	22.5	0.99
45	67.3	1.8	0.68	19.3	0.85
60	72.3	1.6	0.71	17.7	0.78
90	70.3	2.2	0.77	18.3	0.80

γ is 1.5 times the axial strain obtained from the undrained triaxial tests and two times the circumferential strain as measured in the pressuremeter. The pressuremeter test, besides yielding a higher peak strength, gives larger strain to failure and higher post-peak strength than those of the triaxial test. These aspects will be discussed in a later section.

Strength Anisotropy

The results of tests on inclined specimens are shown in Fig. 14 and summarized in Table 5, which indicates a significant strength anisotropy. The horizontal and 60° specimens yield the minimum strength, amount to about 80% of the vertical strength. Part of

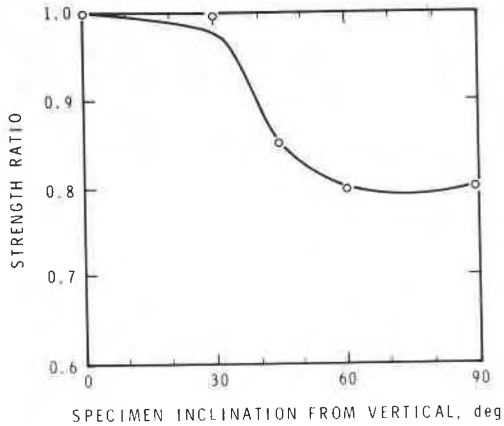

FIG. 14. Strength variation vs. specimen inclination.

TABLE 6. Results of triaxial compression and extension tests on vertical specimens (Series 3)

Test type	Natural moisture content (%)	Failure strain (%)	A_f	Peak strength (kPa)
Compression	64.9	1.8	0.38	26.1
Compression	65.3	2.1	0.43	27.5
Extension	65.3	1.0	1.10	15.5
Extension	65.4	1.3	1.00	15.0

NOTES: ratio of average extension to compression strength = 0.57; ratio of average A_f for extension to A_f for compression = 2.6.

this anisotropy is caused by the pore pressure effect as a higher pore pressure parameter, A_f, has been measured in the inclined specimens.

The triaxial compression and extension tests measured a greater strength anisotropy (Table 6) than the last test series. This is caused by a low strength in the extension test. By comparing A_f values, it is clear that such a low strength is largely due to the pore pressure effect: the A_f for extension is 2.6 times that for compression. This high A_f is in turn caused by the fact that the intermediate principal stress, σ_2, is equal to the major principal stress, σ_1, as compared with $\sigma_3 = \sigma_1$ in the compression test, where σ_3 is the minor principal stress. In the pressuremeter test where a plane strain condition prevails (Wroth and Hughes 1973), the pore pressure response is in general lower than in the extension test. The strength anisotropy component caused by the stress path to failure in the pressuremeter test therefore will not be as pronounced as in the extension test.

A significant strength anisotropy has also been measured in the triaxial-vane tests, the results of which are shown in Table 7. The strength on the vertical plane, S_{v0}, is smaller than that on the horizontal plane, S_{h0}, with S_{v0}/S_{h0} equal to 0.64 and 0.71 at the two elevations, respectively. These values are intermediate between those measured in the inclined specimens test series and compression and extension test series, showing the varying influence, in the different test types, of factors contributing to anisotropy.

TABLE 7. Results of triaxial-vane tests (Series 4)

Depth (m)	Natural moisture content (%)	Consolidation pressure σ_h' (kPa)	Consolidation pressure σ_v' (kPa)	Measured torque ($\times 10^{-3}$ Nm)	S_u* (kPa)
3.4	88.4	34.3	34.3	73.4	20.0
3.4	89.7	34.3	73.6	75.7	20.7
3.4	87.6	73.6	73.6	115.9	31.6
S_{v0} = 19.3*		S_{h0} = 27.2*		S_{v0}/S_{h0} = 0.71	
4.7	71.7	37.3	37.3	87.0	23.7
4.7	71.5	37.3	83.4	88.1	24.0
4.7	71.9	83.4	83.4	135.6	37.0
S_{v0} = 22.4*		S_{h0} = 35.2*		S_{v0}/S_{h0} = 0.64	

*S_u, S_{v0}, and S_{h0} are evaluated using [1], [2], and [3], respectively.

Also shown in Table 7 is the value of S_u, the vane strength computed using [1]. S_u is remarkably close to S_{v0}.

Based upon the foregoing laboratory results, it is clear that, for soils similar to that from South Gloucester, the influence of strength anisotropy should be important. With reference to the maximum strength from vertical compression test or on the horizontal plane, anisotropy may lead to a strength decrease from 20 to 40% depending on the type of shear. In the case of the NGI vane and triaxial-vane tests, the soil resistance mainly comes from the vertical failure surface. The conventional interpretation of the vane test assuming isotropy should therefore give results very close to the strength on the vertical plane S_{v0} as is shown in Table 7. Furthermore, when S_{v0} and S_{h0} are plotted in Fig. 12 they are quite close to the *in situ* vane strength and the triaxial compression strength, respectively. This implies that the low field vane strength compared with the triaxial compression strength (Series 1) can be largely explained by the effect of anisotropy.

In the case of the pressuremeter test, the major principal stress acts in the horizontal direction. If anisotropy is the only factor affecting strength, the pressuremeter should yield a lower strength than the triaxial compression tests (Series 1). This view, however, is in contrast to the strength comparison presented earlier. To explain this apparent contradiction, other factors of equal or greater importance have to be considered. They are to be discussed in the

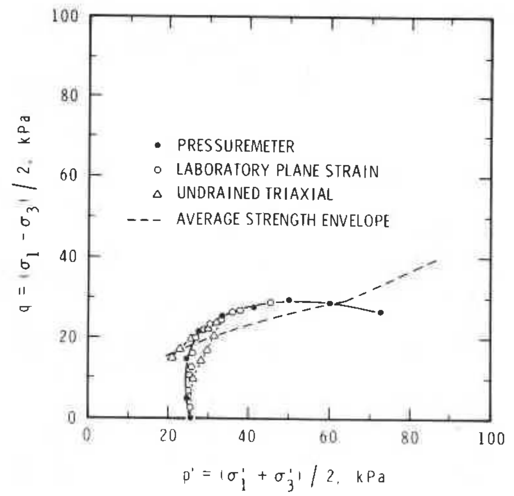

FIG. 15. Effective stress paths for various tests.

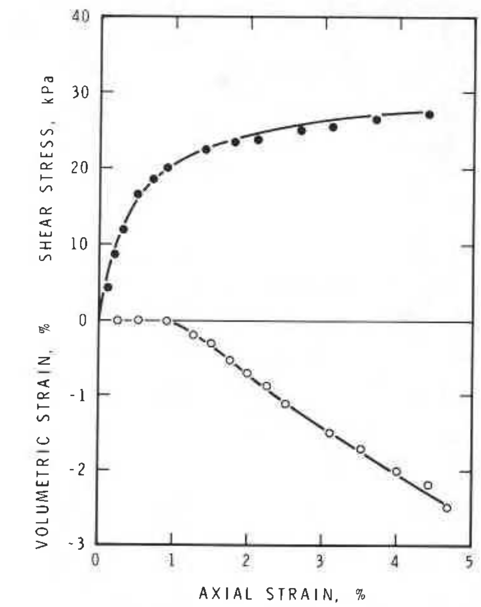

FIG. 16. Shear stress – axial strain and volumetric strain – axial strain relationships during the controlled stress path plane strain test.

ensuing sections. It is sufficient here to say that pressuremeter strength interpretation is subject to factors that can either overestimate or underestimate the undrained strength.

Stress Path Effect

The computed stress path from the pressuremeter test at South Gloucester from depth 3.3 m is shown in Fig. 15. Also shown are the effective stress paths of the plane strain tests on the horizontal specimen and the triaxial test on the vertical specimen (Series 5). In this figure, q is plotted against p' where $q = (\sigma_1 - \sigma_3)/2$ and $p' = (\sigma_1' + \sigma_3')/2$. σ_1' and σ_3' are the effective major and minor principal stresses and in the pressuremeter test they are equal to the effective radial and circumferential stresses, σ_r' and σ_θ', respectively. The effective stress path observed in the field is basically reproduced in the laboratory plane strain test.

There is a striking difference between the effective stress paths in the triaxial and the plane strain tests. After reaching the maximum q value, the triaxial stress path bends towards the origin, resulting in a reduction of both p' and q values. On the other hand, the plane strain stress path, after rising almost vertically, bends to the right. It then describes a shape roughly conforming to the average peak strength envelope derived from the conventional triaxial compression test. Finally, it dips off the envelope after reaching a certain point. The fact that a section of this path is higher than the strength envelope may be partly because of sample variability at that particular depth and partly because of the plane strain mode of shearing.

One important consequence arises from the observed plane strain stress path: as p' continues to increase along the strength envelope, q increases accordingly and a higher strength will be attained. In addition, there is another phenomenon associated with this increasing p', which is shown in Fig. 16 on which the shear stress and volumetric strain versus axial strain are plotted. Along the initial part of shearing, hardly any volume change can be detected, implying the undrained condition assumed in the pressuremeter test is valid at this stage. Upon further straining to higher q values (and hence higher p' values) there is a gradual decrease in the specimen volume. Significant consolidation is therefore taking place at this stage and the undrained condition is no longer satisfied. As a result, the pressuremeter test, conducted along the observed stress path, corresponds to a partially drained test, yielding a strength substantially larger than the undrained value. For South Gloucester clay, this partially drained strength exceeds the triaxial undrained strength by 19% (Table 8). Furthermore, this partial drainage leading to shear at higher p' values also results in a greater strain to failure and gives rise to higher strength at large strain (postpeak), both being mentioned in an earlier section.

Typical effective stress paths during pressuremeter tests at NRCC and Matagami sites are shown in Fig. 17. They are essentially of the same shape as that of the South Gloucester clay, again indicating

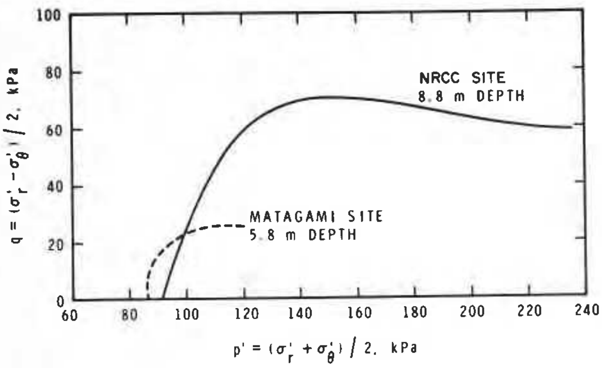

FIG. 17. Effective stress paths as measured from pressuremeter tests.

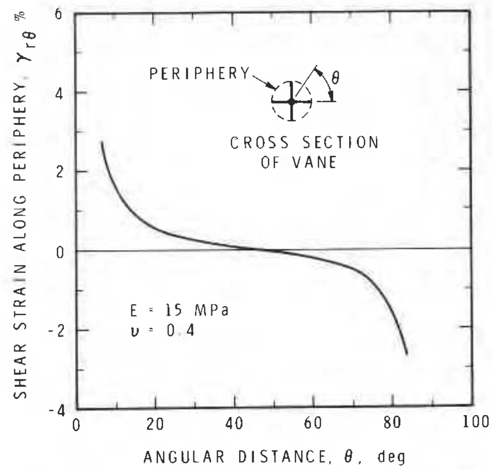

FIG. 18. Shear strain induced along periphery due to vane insertion.

TABLE 8. Results of plane strain and triaxial tests (Series 5)

Test type	Natural moisture content (%)	Failure strain (%)	Peak strength (kPa)
Controlled stress path plane strain test on horizontal specimen	67.8 70.6	4.5 4.7	29.0 28.0
Undrained triaxial compression test on vertical specimen	70.2 69.2	1.41 1.51	23.5 24.6

NOTE: ratio of average plane strain to triaxial strength = 1.19.

that the measured pressuremeter strengths occur in a pressure range higher than that in an undrained test.

Disturbance

Disturbance may be caused by physical disruption of the soil structure and by stress change. It cannot be completely eliminated in any existing laboratory or field test.

In laboratory tests, disturbance is introduced through sampling and trimming of specimens. In this study, physical disruption of the soil structure has been reduced as large diameter or block samples have been used. Stress changes due to sampling are removed by reconsolidating the specimens to *in situ* pressures prior to shearing. It is believed therefore that the laboratory tests measured realistic strengths corresponding to the stress path and strain rate used in the tests.

Inserting the vane into soil will also create disturbance. The vane blades tend to split the soil at the tips, but this tendency is restrained partly or wholly by the confinement of the surrounding soil, hence imposing a stress change. This process is difficult to analyze precisely. It can be studied, at least qualitatively, by means of the finite element method assuming elastic soil behaviour. When applied to the field vane, this method will indicate the induced stress and strain due to vane insertion. Of particular interest is the shear strain along the periphery of the prospective failure surface circumscribed by the vane. The distribution of this strain along a quadrant of the periphery is shown in Fig. 18. It is asymmetrical about and changes sign at the 45° line. The magnitude and direction of the induced shear there will also follow the same pattern. During shearing, however, the vane is turned in only one direction, which is the same as the induced shear on some part of the failure surface but opposite on the other part. This will then give rise to failure under both the passive and the active modes, yielding a strength intermediate between values corresponding to the two different modes of failure. This strength is therefore smaller than the triaxial compression strength that is associated with the active failure mode. How much smaller depends, of course, on the characteristics of the soil.

Soil disturbance plays a different role in the interpretation of the pressuremeter tests. Instead of reducing the measured strength, it increases the strength. Based on the theories of Baguelin *et al.* (1975) and Prévost (1979), the apparent undrained pressuremeter strength may be double the undisturbed strength. This peculiarity is derived from the method of interpretation, which assumes uniform soil surrounding the pressuremeter without regard to the soil softening process due to disturbance.

For the present test series, it is possible to evaluate the effect of such disturbance using the method of Baguelin *et al.* (1975). Briefly, this method yields the apparent stress–strain relationship when the size of the disturbed zone and the stress–strain character-

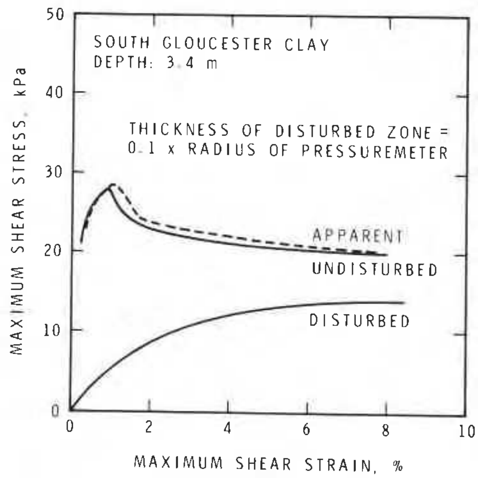

FIG. 19. Theoretical apparent stress–strain curve from a pressuremeter test conducted in an annulus of disturbed soil.

istics in the disturbed and the undisturbed zones are known. With the pressuremeter test setup described earlier, it is unlikely that the thickness of the disturbed zone will exceed 10% of the radius of the pressuremeter. The undisturbed stress–strain characteristics may be obtained from the triaxial tests. The disturbed stress–strain relationship is an unknown, but different probable assumptions can be made. Based on these considerations, the apparent stress–strain relationship for South Gloucester clay at 3.4 m depth is shown in Fig. 19 along with the triaxial undisturbed and the assumed disturbed stress–strain characteristics. It is clear that disturbance in this case leads to an insignificant strength overestimation of only about 2%.

Other disturbed stress–strain functions have also been assumed with the disturbed strength values ranging from one quarter of the undisturbed peak strength to that of the undisturbed postpeak strength. The results show little influence in the apparent strength at this size (10% of the pressuremeter radius) of disturbed zone. Hence, it can be remarked that the present setup of conducting pressuremeter tests in soft clays will incur negligible effect from soil structural disruption.

Rate Effect

The time to reach failure in the pressuremeter tests and the conventional triaxial test was nearly the same, about half an hour. Rate effect should not be important therefore in comparing strengths derived from these tests. In the case of the field vane test, however, the soil failed in a few minutes, which is about one order of magnitude less than the time in the other two tests. This higher rate of shearing may increase the soil resistance by roughly 10% based on existing experience (Bjerrum 1972; Law 1974). This increase is insufficient to compensate for the decrease caused by anisotropy and disturbance, hence resulting in a vane strength being smaller than the triaxial strength.

Summary and Conclusions

The Cambridge self-boring pressuremeter and the NGI field vane were used to measure the *in situ* stength at three sites of different subsoils. Two of the sites are located in Leda clay in the Ottawa area. The one at South Gloucester comprises a soft clay and the other on the National Research Council of Canada grounds consists of a firm clay. The third site involves a highly plastic lacustrine clay at Matagami in northwestern Quebec. At all the sites, the pressuremeter strength exceeds the vane strength by an amount ranging from about 20% for the firm Leda clay to about 100% for the two soft clays.

Conventional triaxial compression tests were performed on South Gloucester clay. The results show that the triaxial undrained strength is intermediate between the two measured *in situ* strengths.

Special tests were conducted also on South Gloucester clay to study factors contributing to the strength discrepancy observed between the measured *in situ* and laboratory triaxial strengths. The analysis of results led to the following:

1. Field Vane Strength

Strength anisotropy and disturbance are two factors that cause strength reduction measured by the vane. The clay is anisotropic with strength on a vertical plane or from a horizontal specimen being 20–35% lower than on a horizontal plane or from a vertical specimen. This anisotropy substantially accounts for the low strength compared with the triaxial strength. Vane insertion into the soil partially disrupts the soil structure and creates a stress change, both tending to reduce the available strength.

Rate effect, on the other hand, increases the vane strength but such an increase is outweighed by the strength decrease due to anisotropy and disturbance.

2. Pressuremeter Strength

As the maximum shear stress acts on a vertical plane in the pressuremeter test, strength anisotropy in this clay is therefore expected to lead to a strength decrease of 20–30% compared with the triaxial strength. To the contrary, the pressuremeter strength was found to be about 20% higher instead. This anomaly has been explained in two ways. Firstly, soil disturbance around the pressuremeter gives rise to a higher apparent strength, but this effect is minimal in the present test series. Secondly, the effective stress path during a pressuremeter test is markedly different from that of an undrained triaxial test. It swings upward along the strength envelope instead of downward in the triaxial test. As a result, higher resistance associated with higher effective confining pressure is reached. This upward swing of the effective stress constitutes the main reason for the observed high strength. A laboratory simulation using sophisticated apparatus confirmed the possibility of such a stress path. Furthermore, the simulation yields evidence of pore pressure dissipation in the pressuremeter test. The use of this test for obtaining so-called undrained strength at the present rate (9.8 kPa/min) is therefore questionable.

The results of the various tests reported emphasize the point made by Kenney (1968) that "undrained strength" is a measure of behaviour and not a soil property. The behaviour is influenced by test method, including the associated disturbance, the stress path followed, the rate of stress application, and the anisotropic conditions prevailing in the clay deposit. It is evident that no one test method can be expected to completely describe undrained strength behaviour.

Acknowledgements

The tests at Matagami were made possible through the cooperation and assistance of The Société d'Énergie de la Baie James. The conscientious efforts of A. Laberge and T. J. Hoogeveen, Technical Officers, in conducting the tests under sometimes difficult circumstances are gratefully acknowledged. This paper is a contribution from the Division of Building Research, National Research Council of Canada, and is published with the approval of the Director of the Division.

ANDRESEN, A., and BJERRUM, L. 1956. Vane testing in Norway. Symposium on Vane Shear Testing of Soils, ASTM (Americain Society for Testing and Materials) Special Technical Publication No. 193, pp. 54–60.

BAGUELIN, F., FRANK, R., and JÉZÉQUEL, J. F. 1975. Quelques résultants theoriques sur l'essai d'expansion dans les sols et sur le frottement latéral des pieux. Bulletin de Liaison de Laboratoires des Ponts et Chaussées, **78**(juillet–aôut), pp. 131–136.

BAGUELIN, F., JÉZÉQUEL, J. F., LEMÉE, E., and LE MÉHAUTÉ, A. 1972. Expansion of cylindrical probes in cohesive soil. ASCE Journal of the Soil Mechanics and Foundations Division, **98**(SM11), pp. 1129–1142.

BAGUELIN, F., JÉZÉQUEL, J. F., and SHIELDS, D. H. 1978. The pressuremeter and foundation engineering. Trans. Tech Publications, Causthal, Germany.

BJERRUM, L. 1972. Embankments on soft ground. Proceedings, ASCE Specialty Conference on Performance of Earth and Earth-supported Structures, Purdue University, Lafayette, IN, Vol. 2, pp. 1–54.

BOZOZUK, M. 1972. The Gloucester test fill. Ph.D. thesis, Purdue University, Lafayette, IN.

——— 1974. Minor principal stress measurements in marine clay with hydraulic fracture tests. Proceedings, ASCE Specialty Conference on Subsurface Exploration for Underground Excavation and Heavy Construction, Henniker, NH, pp. 333–349.

DASCAL, P., TOURNIER, J. P., TAVENAS, F., and LA ROCHELLE, P. 1972. Failure of a test embankment on sensitive clay. Proceedings, ASCE Specialty Conference on Performance of Earth and Earth-supported Structures, Lafayette, IN, Vol. 1, Part 1, pp. 129–158.

DELORY, F. A., and LAI, H. W. 1971. Variation in undrained shearing strength by semiconfined tests. Canadian Geotechnical Journal, **8**, pp. 538–545.

DUNCAN, J. M., and SEED, H. B. 1966. Strength variation along failure surfaces in clay. ASCE Journal of the Soil Mechanics and Foundations Division, **92**(SM6), pp. 81–104.

EDEN, W. J., and HAMILTON, J. J. 1956. The use of a field vane apparatus in sensitive clay. Symposium on Vane Shear Testing of Soils, ASTM (American Society for Testing and Materials) Special Technical Publication No. 193, pp. 41–53.

KENNEY, T. C. 1968. A review of recent research on strength and consolidation of soft sensitive clays. Canadian Geotechnical Journal, **5**, pp. 97–119.

LADANYI, B. 1972. *In situ* determination of undrained stress–strain behavior of sensitive clays with the pressuremeter. Canadian Geotechnical Journal, 9, pp. 313–319.

LADD, C. C. 1965. Stress–strain behaviour of anisotropically consolidated clays during undrained shear. Proceedings, 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, P.Q., Vol. 1, pp. 282–286.

LAW, K. T. 1974. Analysis of embankments on sensitive clays. Ph.D. thesis, The University of Western Ontario, London, Ont.

——— 1979. Triaxial-vane tests on a soft marine clay. Canadian Geotechnical Journal, **16**, pp. 11–18.

——— 1980. A servo system for controlled stress path tests. ASTM Symposium on Shear Strength of Soil, Chicago, IL, June.

LAW, K. T., and EDEN, W. J. 1980. Influence of cutting shoe size in self-boring pressuremeter tests in sensitive clays. Canadian Geotechnical Journal, **17**, pp. 165–173.

LAW, K. T., and HOLTZ, R. D. 1978. A note on Skempton's *A* parameter with rotation of principal stresses. Géotechnique, **28**(1), pp. 57–64.

LO, K. Y., and MILLIGAN, V. 1967. Shear strength properties of two stratified clays. ASCE Journal of the Soil Mechanics and Foundations Division, **93**(SM1), pp. 1–15.

MCKINLAY, D. G., and ANDERSON, W. F. 1975. Determination of the modulus of deformation of a till using a pressuremeter. Ground Engineering, **8**(6), pp. 51–54.

PALMER, A. C. 1972. Undrained plane-strain expansion of a cylindrical cavity in clay: a simple interpretation of the pressuremeter test. Géotechnique, **22**(3), pp. 451–457.

PRÉVOST, J. H. 1979. Undrained shear test on clays. ASCE Journal of the Geotechnical Engineering Division, **105**(GT1), pp. 49–64.

WINDLE, D., and WROTH, C. P. 1977. *In situ* measurement of the properties of stiff clays. Proceedings, 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 1, pp. 347–352.

WROTH, C. P., and HUGHES, J. M. O. 1973. An instrument for the *in situ* measurement of the properties of soft clays. Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol. 1, Part 2, pp. 487–494.

——— 1974. The development of a special instrument for the *in situ* measurement of the strength and stiffness of soils. Proceedings, ASCE Specialty Conference on Subsurface Exploration for Underground Excavation and Heavy Construction, Henniker, NH, pp. 295–311.