A Study of the Anisotropy of Champlain Sea Clay in the South-East Ottawa Area using Field Vane Tests

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ABSTRACT
In the 1970's, a site investigation was carried out to determine geotechnical design criteria and parameters for the planning and construction of a proposed new community which was being considered for a portion of the Township of Gloucester in the south-east section of the Regional Municipality of Ottawa-Carleton, Ontario, Canada. Previous limited investigations, along with published and unpublished data, indicated that the approximately 2,500-hectare site was underlain by an extensive deposit of Leda (Champlain Sea) clay and that the subsurface conditions would significantly influence the planning for the new community, also known as 'South-East Ottawa City'. As part of the investigation, special field vane tests were performed to determine the possible anisotropy of the Champlain Sea clay.

This paper describes the investigation procedures, subsurface conditions and properties of the Champlain Sea deposits underlying the site, the method of conducting the special vane tests to determine the anisotropy, and the calculated horizontal to vertical undrained shear strength ratios.

RÉSUMÉ
Dans les années 1970, une étude du site a été effectuée pour déterminer les critères et les paramètres de conception géotechnique pour la planification et la construction d'une nouvelle communauté proposée qui était envisagée pour une partie du canton de Gloucester dans la section sud-est de la municipalité régionale de Ottawa-Carleton, Ontario, Canada. Des enquêtes limitées antérieures, ainsi que des données publiées et non publiées, ont indiqué que le site reposait sur un vaste dépôt d'argiles de Leda (mer de Champlain) et que les conditions souterraines influenceraient considérablement la planification de la nouvelle communauté, également connue sous le nom de « Sud-Est Ville d'Ottawa'. Dans le cadre de l'enquête, des essais spéciaux à la girouette ont été effectués sur le terrain pour déterminer l'anisotropie possible des argiles de la mer de Champlain.

Cet article décrit les procédures d'investigation, les conditions souterraines et les propriétés des dépôts de la mer de Champlain sous-jacents au site, la méthode d'exécution des essais spéciaux à la girouette pour déterminer l'anisotropie et les rapports calculés de résistance au cisaillement horizontal à vertical non drainé.

1 INTRODUCTION

In the early 1970's, the Ontario Government assessed the feasibility of the development of a planned community in parts of Concessions VII, VIII and IX of Gloucester Township, Carleton County. The proposed development, known as the 'South-East Ottawa City', was loosely bordered by Bar Line Road on the west, Highway 417 and Boundary Road to the north and east, Thunder Road and Mitch Owens Road to the south, with Bear Brook running through the property (Figure 1).

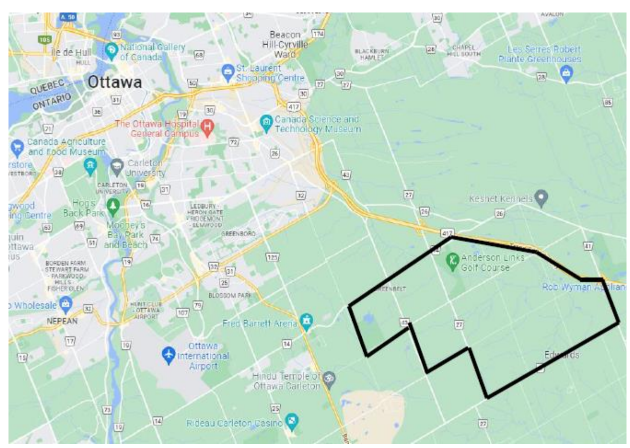

Figure 1 – Site Location

A preliminary site investigation carried out in 1972 revealed that much of the site was underlain by a deposit of weak, slightly over-consolidated Leda (Champlain Sea) clay, extending to a depth of at least 30 m. In view of the rather unfavourable conditions, a further investigation was performed during the winter of 1973 and the first half of 1974. Several deep boreholes, with extensive sampling and laboratory testing was performed by one geotechnical firm. More frequent and shallower (typically 8.5 m deep) boreholes were advanced by another geotechnical firm, along with the delineation of the subsurface conditions by means of geophysical methods.

The investigations showed that about half of the site was covered with a surficial sand deposit, especially in a band of land bordering Highway 417, where the maximum depth of the sand deposit reached 5 m. Elsewhere, the sand was sporadic in occurrence and its thickness was generally less than 2.5 m. The significant soil stratum underlying the site, and known to occur extensively throughout the region, was Champlain Sea clay. The thickness of the clay in the south-east corner area was about 6 m, while in the north-west corner it was inferred to be about 58 m. During this investigation, extensive field vane testing was performed in the shallow boreholes to determine the undrained in situ shear strength of the clay deposit using a standard field vane. Special field vane tests were also conducted to assess any possible anisotropy of the clay.

This paper presents the findings of the special field vane tests.

2 THE PROJECT

A large development (in this case approximately 2,500-hectare satellite community to the City of Ottawa) requires the planning for single family dwellings and heavier structures such as multi-residential units, schools, office buildings, fire and city halls, libraries, commercial and light industrial structures, along with underground services, etc.

While much of the Ottawa area to the south-east is underlain by the Champlain Sea clay, the clay at this development site appeared to possess less favourable characteristics, especially for the construction of foundations. The groundwater level was found at or near the ground surface. A permanent lowering of the groundwater table was inevitable (and to a certain extent, desirable) due to the development (e.g., underground installations, basements, etc.). In view of the anticipated considerable land subsidence due to inevitable groundwater lowering, the services and subdrains would need to be installed at least two years in advance of construction.

From the available data, relatively more favourable subsurface conditions were anticipated near Highway 417 (straddling the north-east and eastern boundaries of the site) which was the logical choice for the town centre where heavier structures, deeper underground services and major roads would be concentrated.

As a result of the preliminary subsurface investigation, the scale of the development was reduced, leaving out the least favourable areas. The detailed investigation (1973-1974) concentrated on the further details of the design, as well as the delineation of the surficial sand deposit which covered about half of the site, the thickness of the crust of the clay deposit and the depth to a competent bearing stratum to glacial till or bedrock, suitable to support driven pile or cast-in-place concrete pile foundations for heavier structures.

3 FIELD VANE TESTING PROCEDURES

The shallow boreholes were advanced with power auger drill rigs equipped for soil sampling and testing. Because of caving ground conditions, hollow-stem augers were employed (hollow-stem augers were relatively new in the early 1970’s). The primary purpose of these boreholes was to explore the variations in the upper zones of the subsoil. The boreholes were taken to a depth of 28 ft or 8.5 m and a uniform sampling and testing pattern was employed. The soil samples were obtained at 2.5 ft (0.76 m) intervals of depth, mostly by driving a 2-inch (5.08 cm) diameter split-spoon sampler by the Standard Penetration Test (SPT) method.

As the SPT method does not yield reliable values for determining the consistency of weak cohesive soils, the undrained shear strength was measured in situ by means of vane shear tests performed at 2.5 ft (0.76 m) intervals of depth where the boreholes were in clay. In these tests, a four-bladed vane was attached to an A-sized drill rod and pushed through the hollow-stem augers about 12 inches (0.305 m) into the undisturbed soil after the removal of each split-spoon sample. The drill rods were attached to a ball-bearing collar at the top of the augers to hold the weight of the vane testing equipment and to minimize side friction. The torque required to shear the clay was measured with a calibrated torque wrench equipped with a follow-up pointer to record the maximum reading. After measuring the undisturbed shear strength, in which the failure was reached in about one minute, the vane was rotated rapidly six times and the remolded shear strength was measured. To assess the amount of friction in the apparatus, several ‘dummy tests’ were performed using a plain rod in place of the vane. It was found that the torque required to turn this rod was negligible and therefore, no correction had to be applied to the actual test reading. The undrained shear strength of the clay was calculated from the torque readings using a vane constant 10 per cent less than the theoretical. This compensated, at least in part, for the effect of the relatively rapid rate at which these tests were performed.

4 THE CLAY

Clay was encountered in all the boreholes advanced during the investigation. Its thickness varied considerably within the project area. In the south-east corner, the least thickness (about 6 m) was found, while in the north-west corner area the thickness was inferred to be about 58 m.

The deposit, which occurs extensively throughout the region, is known as the Champlain Sea clay or the Leda clay, as it was known in the early 1970’s (see Bélanger 1998). Although generally referred to as one soil deposit, it is in fact probably composed of three or four strata of different geological origin. For the purposes of the following discussion, the clay described within about 8.5 m of the ground surface will be treated as a single soil deposit.

Generally, the clay, as encountered, was grey in colour, with distinct bands of red-brown coloration in places. The presence of occasional black mottles was observed. In places, the upper part of the clay was brown as a result of oxidation and banded with red-brown colour. The grey part of the clay had a blocky structure in that, when squeezed, it failed along these pre-determined planes of weakness. The upper parts of the stratum, particularly where oxidation occurred, were highly fissured.

Typical borehole logs are given in Figures 2 and 3.

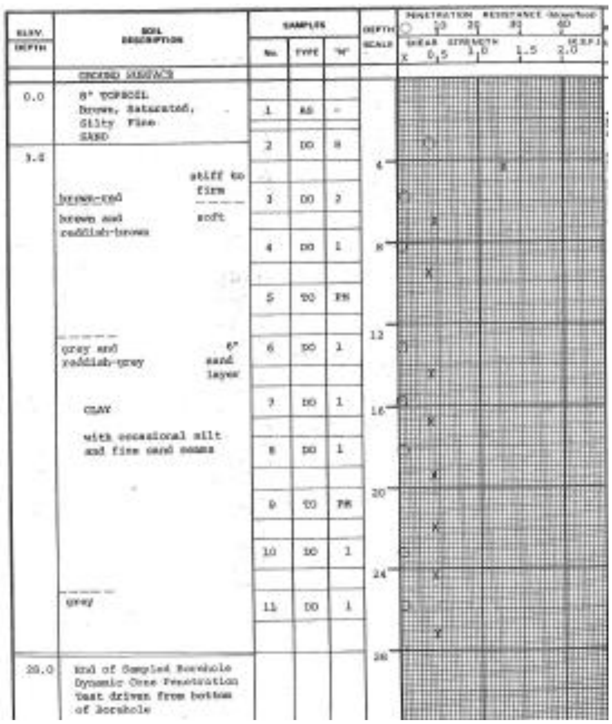

Figure 2 – Typical borehole log (BH 149)

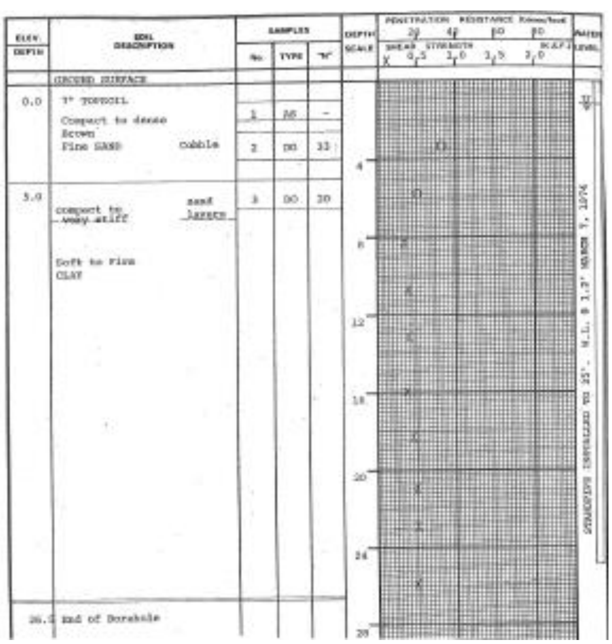

Figure 3 – Typical borehole log (BH 159)

Typical grain size curves for the clay, as plotted in Figure 4, show that the percentage of the clay size particles (i.e., finer than 0.002 mm size) is high, at between 50% and 70%.

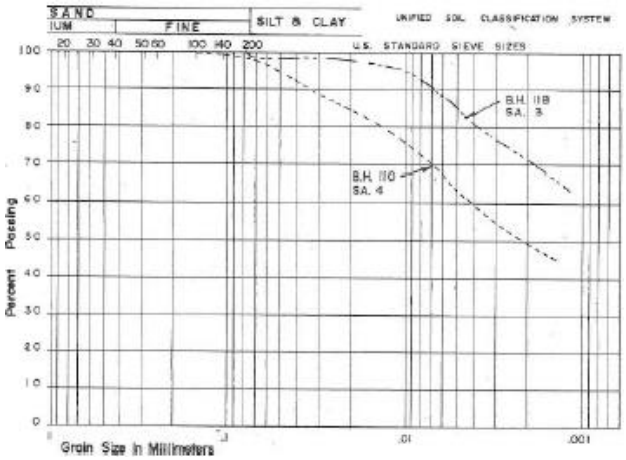

Figure 4 – Typical grain size curves for the clay

The results of typical Atterberg Limits tests are plotted on the Plasticity Chart in Figure 5. This shows that the clay is of medium to high plasticity (Liquid Limit 28% to 86% and Plasticity Index 14 to 63). The results fall close to a straight line which is approximately parallel to the 'A' Line. This indicates that the samples represent soil of the same origin.

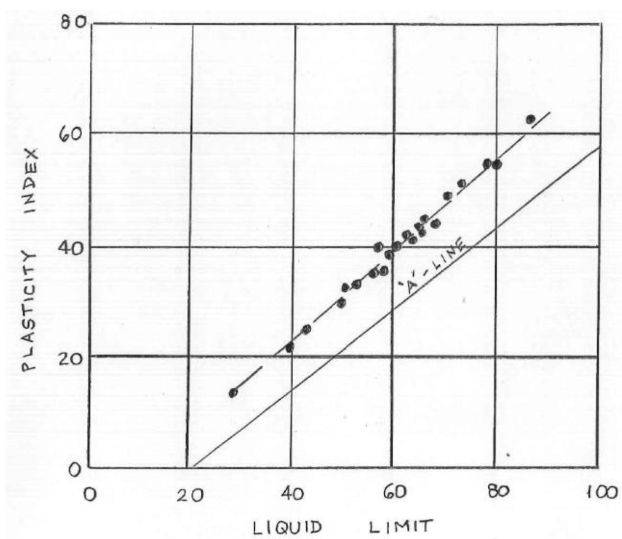

Figure 5 – Typical Atterberg Limits test results

The measured natural moisture content of the clay ranged between 28% and 102%, with the lowest values being obtained in the upper part of the stratum, denoting desiccation. With depth, the natural water content is generally of the order of 60% ± 20% with occasional values of 100%. Generally, the clay has a natural water content of at or above its liquid limit except in the desiccated crust where the liquidity index drops to about 0.3. These findings indicate that the clay has a low shear strength and that it has received little, if any, precompression in its history.

Values of undrained shear strength of the clay, as determined by in situ vane tests, are plotted on the summary of plots of undrained shear strength versus depth in Figure 6 and of undrained shear strength versus elevation in Figure 7. A combined plot of the test results from all the boreholes was made. This showed considerable scatter of values, possibly due in part, to the effect of sand and silt seams. However, it was felt that there were sufficient number of consistent results for the following conclusions to be drawn. In places, there is a stiff upper crust on the clay where it has been desiccated, as already noted. In the crust, the shear strength, as measured by means of field vane tests, ranges up to 120 kPa and occasionally higher. At the same time, the Standard Penetration Tests gave 'N' values of 6 to 29 blows per 0.3 m in the crust, indicating a firm to very stiff consistency.

Below the crust (where it exists) or immediately below the ground surface at other locations, the shear strength ranges from about 10 to 34 kPa within the depths of 8.5 m explored. In many of the boreholes, an approximately linear increase of undrained shear strength with depth was observed. This usually indicates normally consolidated to lightly over-consolidated clays. This pattern can be observed in Figure 6, which is a plot of envelopes of undrained shear strength versus depth for all the boreholes. Several of the boreholes gave results consistently falling close to the lowest values. The ratio of the undrained shear strength to effective overburden pressure for this line is 0.5 but nearly all the observed values fall within the shaded area, which represents a comparatively narrow range of values bounded by the lowest. In a few of the boreholes (e.g., BH 7 and BH 17), the undrained shear strength versus depth relationship is significantly higher than the norm. The higher values in these boreholes can be explained by the fact that all these boreholes were located close to watercourses and have been affected by local drawdown. A profile of the lowest undrained shear strength having a c/p ratio of 0.30 was interpreted with two distinct zones of higher undrained shear strength around 3 m and 8 m depth. As the shear strength versus depth profiles, when projected upwards, pass close to the origin, it was concluded that the clay has little, if any, pre-consolidation.

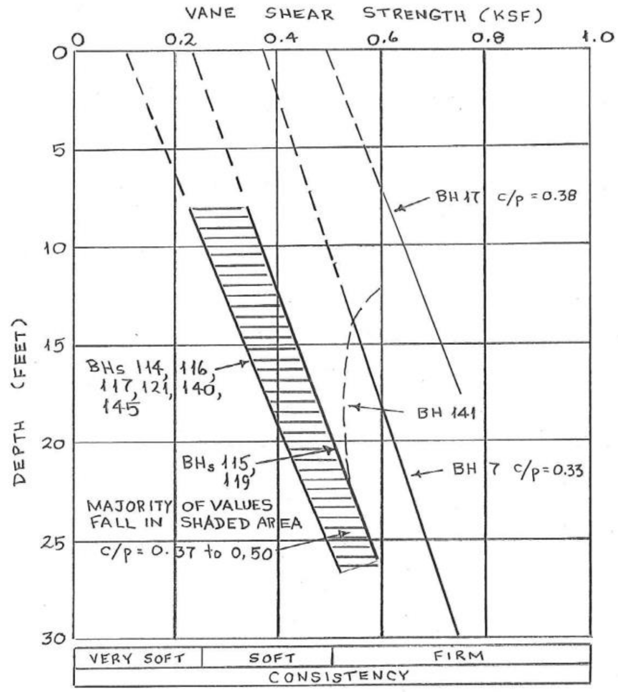

Figure 6 – Undrained shear strength of clay versus depth

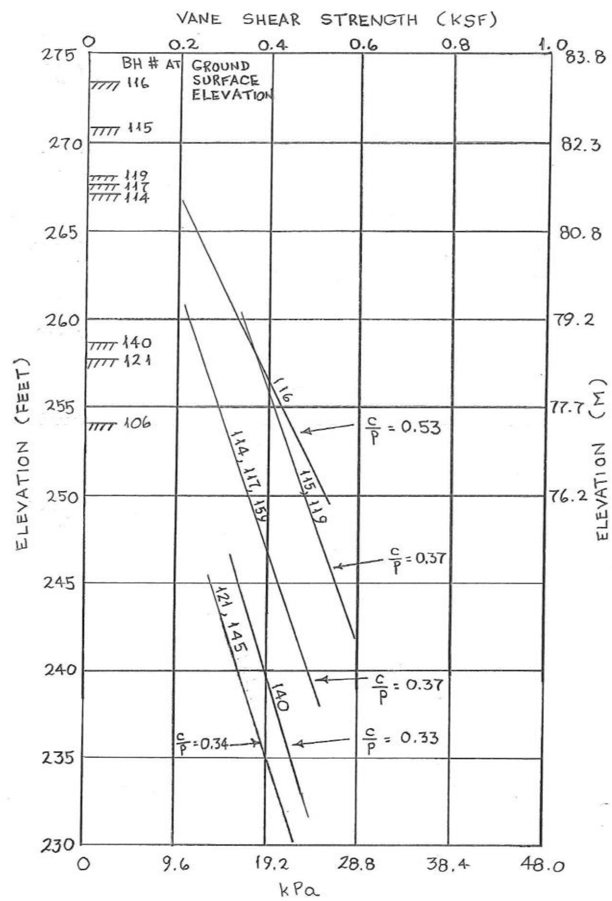

Figure 7 – Undrained shear strength of clay vs elevation

The results of two of the many one-dimensional consolidation (oedometer) tests are given in Figures 8 and 9.

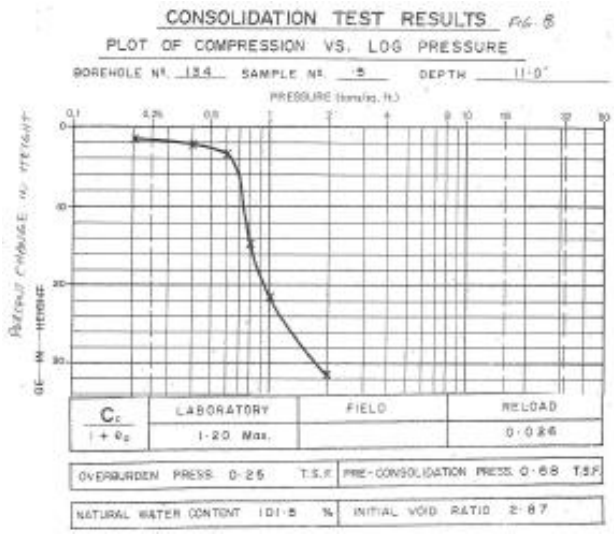

Figure 8 – One-dimensional consolidation (oedometer) test

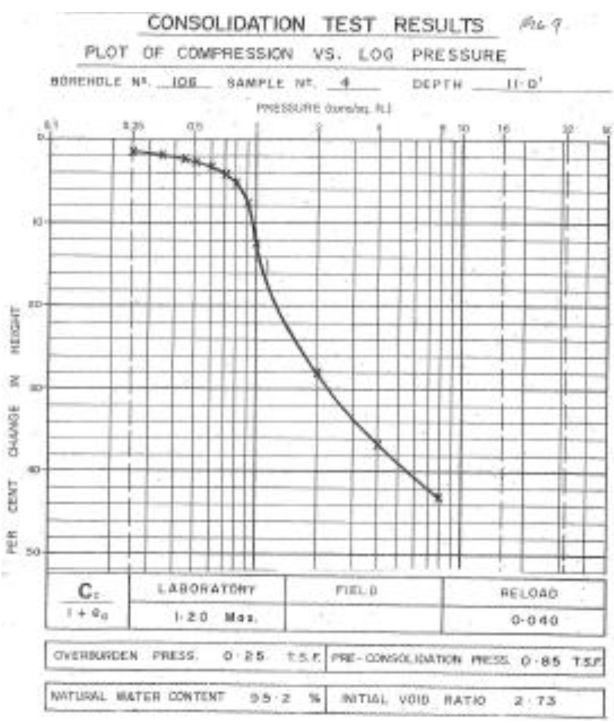

Figure 9 – One-dimensional consolidation (oedometer) test

5 METHOD OF INVESTIGATION OF ANISOTROPY

A series of in situ vane tests were performed to determine if the clay displayed anisotropic properties with respect to horizontal and vertical planes. Field vane tests were performed in boreholes, which were advanced 1.8 to 2.0 m apart, using vane blades of different lengths and widths, with square ends, as detailed below in Table 1 and in Figure 10:

Table 1 – Vane blade dimensions

Length (L) cm (in)	Diameter (D) cm (in)	D/L	Vane Constant
12.70 (5.0)	5.08 (2.0)	0.40	48
7.62 (3.0)	6.35 (2.5)	0.83	48
3.81 (1.5)	7.62 (3.0)	2.00	46
15.24 (6.0)	5.71 (2 5/8)	0.44	22*

* Standard vane used for project – diamond shaped.

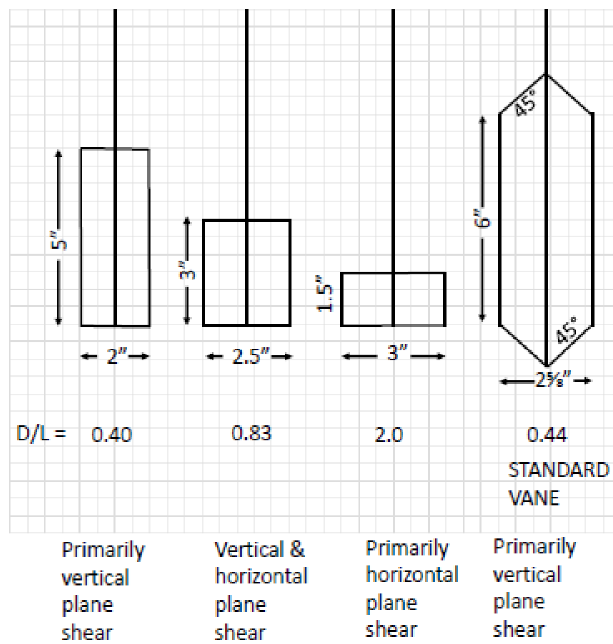

Figure 10 – Different field vane blades used

The results of the tests are presented on the plots of undrained shear strength versus depth in Figures 11 to 14:

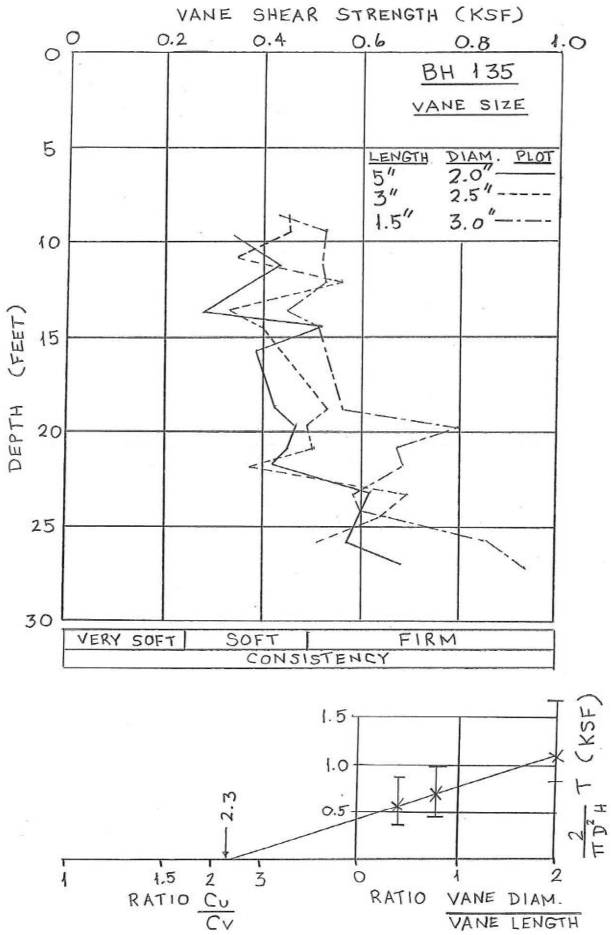

Figure 11 – Undrained shear strength versus depth (BH 135)

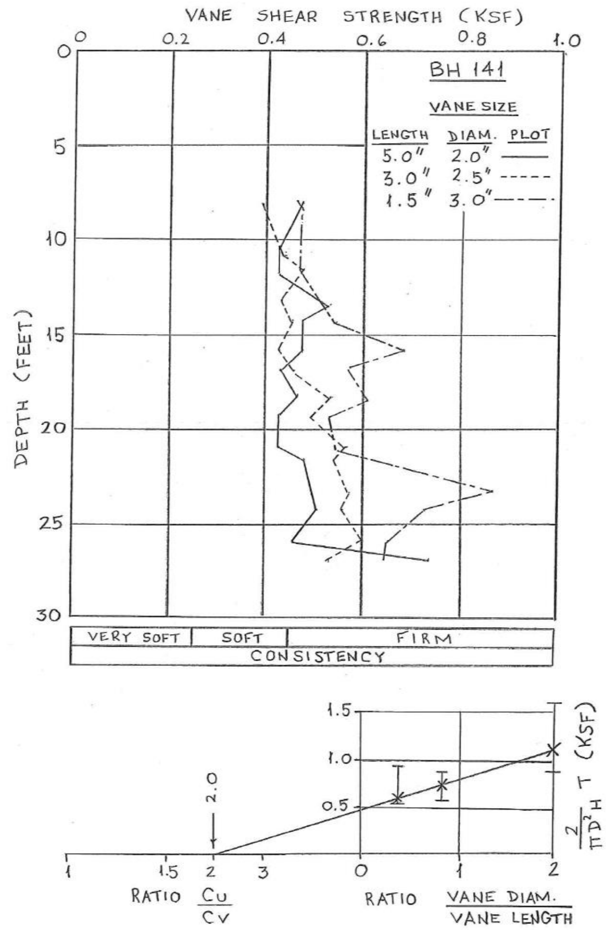

Figure 12 – Undrained shear strength versus depth (BH 141)

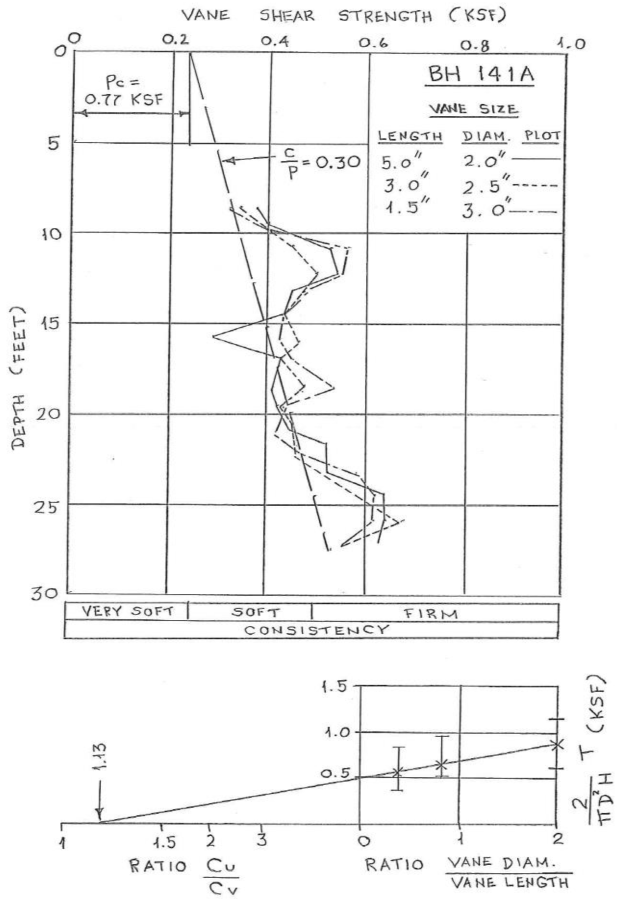

Figure 13 – Undrained shear strength versus depth (BH 141A)

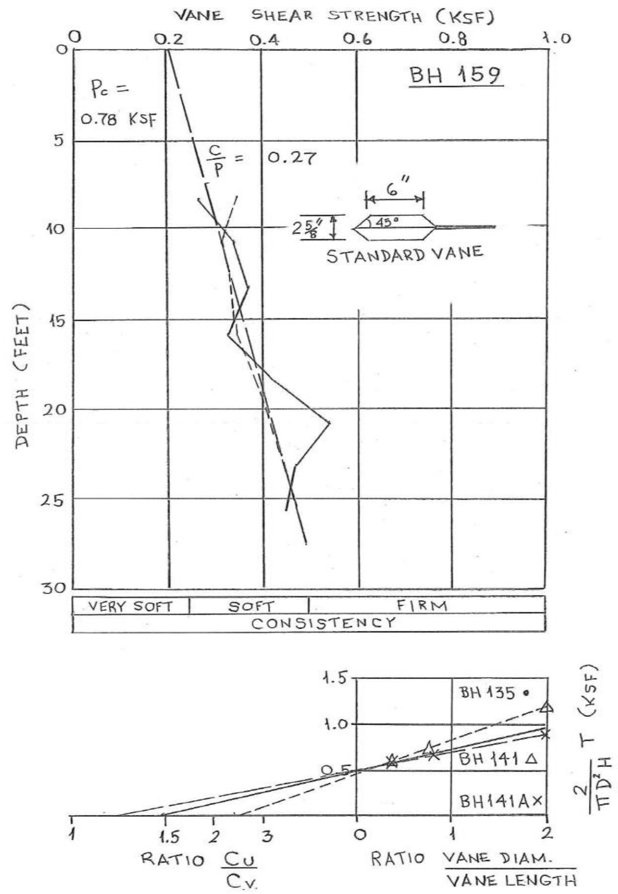

Figure 14 – Undrained shear strength versus depth (BH 159)

6 DISCUSSION OF THE RESULTS

At the bottom of each plot in Figures 11 to 14, inclusive, is an analysis of the results in a format introduced by Aas in 1965 to determine the ratio of the shear strengths on the horizontal and vertical planes (i.e., Ch/Cv). These indicate shear strength ratios Ch/Cv of 1.13, 2.0 and 2.3.

Ratios of horizontal to vertical shear strengths were also calculated by comparing the results obtained using the 1.5" x 3.0" vane (shearing primarily in the horizontal plane) versus using the 5" x 2" vane (shearing primarily in the vertical plane) yielded Ch/Cv ratios ranging from 0.93 to 1.76, as detailed in Table 2:

Table 2 – Ratios of horizontal to vertical shear strengths

BH No.	Ratio of Undrained In Situ Shear Strength as Measured by Vane Sizes 1.5" x 3.0" vs 5" x 2"	As Determined by the Aas Method
135	1.00 – 1.76	2.3
141	1.00 – 1.72	2.0
141A	0.93 – 1.47	1.13

To assess possible natural variations in the shear strength over short distances, two series of vane tests were conducted in BH 159 using a standard vane which was employed throughout the site. The results are given in Figure 14, which also shows combined average results obtained with other types of vanes. This shows that consistent results were obtained. It was concluded that the average shear strength recorded by different vane profiles at any one location reflects the anisotropy of the clay. The results suggest that the anisotropic effect may be quite high in places with the ratio of Ch/Cv reaching a value of approximately 2.

As a result of these findings, it was also surmised that the measured anisotropic effect may at least give a partial explanation of the discrepancy between shear strengths as determined by the vane tests and those determined by compression tests on block samples for this project.

Various modifications of undrained shear strength, as measured by field shear vane testing that may be required, are available in the literature (e.g., Kulhawy and Mayne 1990).

7 CONCLUSIONS

Field (in situ) vane tests performed on Champlain Sea clay south-east of Ottawa using different shapes of vanes (different diameter to length ratios) showed that the ratio of the horizontal to vertical shear strengths varies from a minimum of 1.13 to 2.3. This gives a strong indication of significant anisotropic properties in shear strength for the clay. This can have significant implications, among others, in slope stability investigations and calculations involving the selection of shear strengths along slip surfaces in Champlain Sea clay.

8 REFERENCES

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