

Engineering Characteristics of Sensitive Marine Clays - Examples of Clays in Eastern Canada

by

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List of Symbols

f_s : sleeve stress

q_c : tip stress

u_2 : pore pressure (position 2 for measurement behind the cone tip)

q_t : corrected tip stress

R_f : friction ratio

F_r : normalized friction ratio

B_q : pore pressure ratio

Q_t : normalized cone resistance

I_c : soil behaviour type index

S_u : undrained shear strength

N_{kt} : soil specific cone factor values for cone tip from corrected tip resistance

N_k : soil specific cone factor values for cone tip from uncorrected tip resistance

$N_{\Delta u}$: soil specific cone factor values for cone tip from excess porewater pressure

S_t : sensitivity of cohesive soils

P'_c : effective preconsolidation pressure

N_{st} : soil specific cone factor value

OCR: overconsolidation ratio

N_{60} : equivalent 60% energy efficient field N-value

FC%: fines content in percent

N_{su-40} : soil specific ball factor for 40 mm ball tip

N_{su} : soil specific ball factor for 113 mm ball tip

Abstract

Sensitive marine clay in Ottawa is a challenging soil for geotechnical engineers. This type of clay behaves differently than other soils in Canada or other parts of the world. They also have different engineering characteristic values in comparison to other clays. Cone penetration testing in sensitive marine clays is also different from that carried out in other soils. The misestimation of engineering characteristics from cone penetration testing can result. Temperature effects have been suspected as the reason for negative readings and erroneous estimations of engineering characteristics from cone penetration testing. Furthermore, the applicability of correlations between cone penetration test (CPT) results and engineering characteristics is ambiguous. Moreover, it is important that geotechnical engineers who need to work with these clays have background information on their engineering characteristics.

This thesis provides comprehensive information on the engineering characteristics and behaviour of sensitive marine clays in Ottawa. This information will give key information to geotechnical engineers who are working with these clays on their behaviour. For the purpose of this research, fifteen sites in the Ottawa area are taken into consideration. These sites included alternative technical data from cone and standard penetration tests, undisturbed samples, field vanes, and shear wave velocity measurements. Laboratory testing carried out for these sites has resulted in acquiring engineering parameters of the marine clay, such as preconsolidation pressure, overconsolidation ratio, compression and recompression indexes, secondary compression index, coefficient of consolidation, hydraulic conductivity, clay fraction, porewater chemistry, specific gravity, plasticity, moisture content, unit weight, void ratio, and porosity. This thesis also discusses other characteristics of sensitive marine clays in Ottawa, such as their activity, sensitivity, structure, interface shear behaviour, and origin and sedimentation.

Furthermore, for the purpose of increasing local experience with the use of cone and ball penetrometers in sensitive marine clays in Ottawa, three types of penetrometer tips are used in the Canadian Geotechnical Research Site No. 1 located in south-west Ottawa: 36 mm cone tip, and 40 mm and 113 mm ball tips. The differences in their response in sensitive marine clays will be discussed. The temperature effects on the penetrometer equipment are also studied. The

differences in the effect of temperature on these tips are discussed. Correlations between the penetrometer results and engineering characteristics of Ottawa's clays are verified.

The applicability of correlations between the testing results and engineering characteristics of sensitive marine clays in Ottawa is also presented in this thesis. Two correlations from the Canadian Foundation Engineering Manual are examined. One of these correlations is between the N_{60} values from standard penetration testing and undrained shear strength. The other correlation is between the shear wave velocity measurement and site class. Temperature corrections are suggested and discussed for penetrometer equipment according to laboratory calibrations. The significance of the effects due to radical temperature changes in Canada and Ottawa is discussed.

Some of the main findings from this research are as follows.

- The Canadian Foundation Engineering Manual presents a correlation between standard penetration tests (SPTs) and the undrained shear strength of soils. This relationship may not be applicable to sensitive marine clays in Ottawa.
- Another correlation between the site class, shear wave velocity, and undrained shear strength is presented by this same manual which may not be applicable to sensitive marine clays in Ottawa.
- The rotation rate for field vane testing as recommended by ASTM D2573 is slow for sensitive marine clays in Ottawa.
- Correction factors applied to undrained shear strength from laboratory vane tests may not result in comparable values with the undrained shear strength obtained by using field vane tests.
- Loading schemes in consolidation or oedometer testing may affect the quality of the targeted results.
- Temperature corrections should be applied to penetrometer recordings to compensate for the drift in the results of these recordings due to temperature changes.
- The secondary compression index to compression index ratio presented in the literature may not be the value obtained from this research.

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Chapter 1: Introduction

1.1 Problem Statement

Sensitive marine clays cover large areas in Canada, including Ottawa, the capital region. There has been a steady increase in the population of the Ottawa region, with the present population reaching slightly more than one million and still increasing at a steady rate. This growth has contributed to a significant increase in infrastructure facilities which include the construction of several residential areas, highways, pipelines, and light rail transportation facilities even in the problematic pockets of sensitive Ottawa clays, which were not avoidable. Therefore, there is an urgent need to thoroughly understand and reliably determine the geotechnical properties or behaviour of the sensitive marine clays in the Ottawa region.

The understanding and assessment of the geotechnical properties of the aforementioned clays is critical for the safe and cost-effective design of civil or geotechnical engineering structures, such as low/high buildings, shallow/deep foundations and retaining walls, and to estimate the stability of slopes (i.e. embankments and natural slopes) in Ottawa marine clay areas. However, our understanding of the geotechnical properties of Ottawa sensitive clay is limited and the characterization of the Ottawa marine clay has been a challenge to geotechnical engineers due to its complex behaviour as illustrated by a few examples below.

Sensitive marine clays in Ottawa behave differently than other marine clays in other parts of Canada or the world. Therefore, the behaviour of these clays should be characterized. Correlations based on field or laboratory testing to obtain engineering parameters may or may not be applicable to these clays. Two correlations are found in the Canadian Foundation Engineering Manual. These two correlations relate the standard penetration test (SPT) to the undrained shear strength on the one hand, and the shear wave velocity to the site class on the other hand.

Laboratory testing on sensitive marine clays in Ottawa may result in a wide range of values for engineering parameters. Therefore, the need to present suggested ranges for these parameters is evident. Differences between the undrained shear strength estimated from laboratory and field vanes are observed. The difference between the coefficient of consolidation estimated from

oedometer testing and pore pressure dissipation by cone penetration testing is also observed. Also, the ratio of the secondary compression index to the compression index is observed to be higher than the values reported by previous researchers.

Cone penetration testing is found to overestimate or underestimate the engineering parameters of these clays. The resultant soundings of cone penetration tests (CPTs) are found to render negative or erroneous values on many occasions. Therefore, the correlations between CPTs and these clays should be characterized and examined. Also, local experience on the use of cone and ball penetrometers should be improved in order to increase the number of approaches to achieve the desired engineering parameters. The effect of distance between the location of CPTs and field vanes should be addressed.

1.2 Research Objectives

The main objective of this thesis is to characterize the behaviour and engineering properties of sensitive marine clays in Ottawa. This will also provide key information to geotechnical engineers who need to work with these clays, such as information on the consolidation and shear behaviors, permeability, and other engineering properties. This thesis experimentally investigates the following objectives:

- to characterize the engineering properties of sensitive marine clays in Ottawa by using penetrometers,
- to understand the effect of temperature changes on penetrometer testing in marine clays in Ottawa,
- to assess existing correlations and empirical factors between CPTs and engineering properties of marine clays in Ottawa, with the view to developing guidelines for practicing engineers who work with these sensitive marine soils and other similar soils worldwide,
- to assess correlations between SPTs and undrained shear strength from the Canadian Foundation Engineering Manual, and
- to assess correlations between shear wave velocity and undrained shear strength to site class from the Canadian Foundation Engineering Manual.

1.3 Thesis Organization

This thesis is organized in the form of technical papers and contains five chapters, including this chapter (Figure 1-1).

- Chapter 2 provides the background information on the origin, geological settings, sedimentation, structure, and behaviour of sensitive marine clays in Canada, including the capital region. Some of the information mentioned in this chapter may not comprise the main topics of this thesis, but are still related to the thesis concepts and intended to provide as much information as possible on sensitive marine clays.
- Chapter 3 presents Technical Paper I, which provides a broad assessment and review of the behaviour and parameters of sensitive marine clays in Ottawa.
- Chapter 4 presents Technical Paper II, which deals with the characterization of sensitive marine clays in Ottawa by using cone and ball penetrometers. The chapter also addresses the applicability of correlations between penetrometer outcomes and engineering parameters of these clays.
- Chapter 5 presents the summary, conclusion, and recommendations of this thesis. The chapter also contains practical guidelines for the application of CPTs in the field for specific equipment owned by Stantec Consulting Limited.

It should be emphasized that some of the information is repeated because the main results of the thesis are presented as technical papers. This is because each paper is independently written and according to the manuscript preparation instructions of the corresponding publication medium.

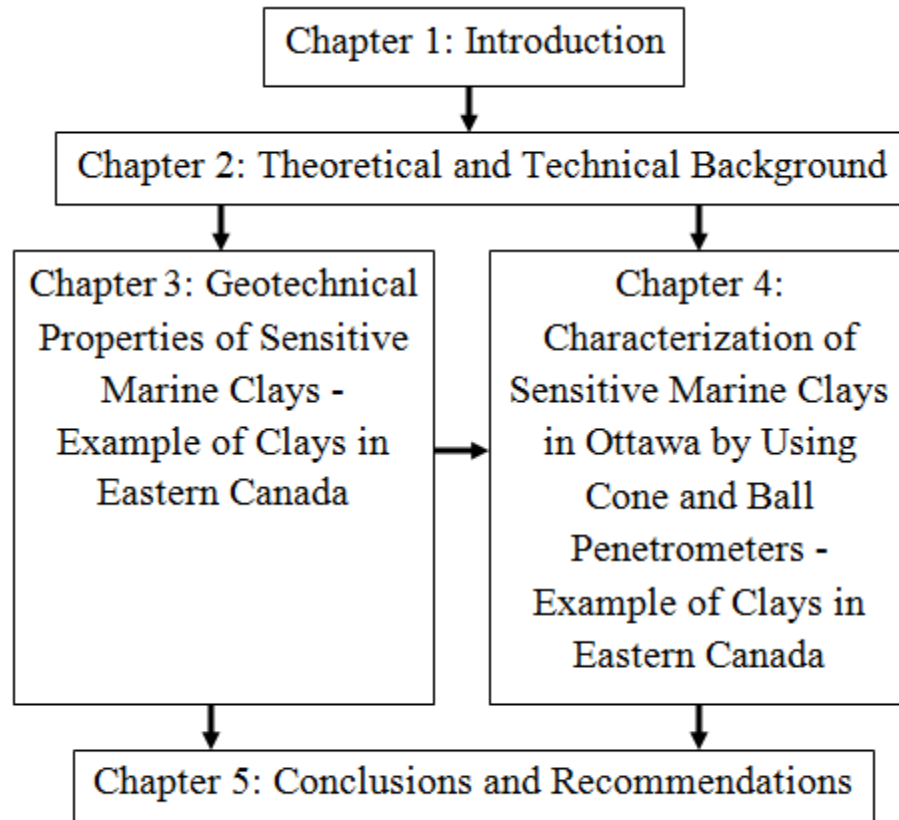


Figure 1-1 Schematic diagram that illustrates the organization of the thesis

Chapter 2: Technical and Theoretical Background

In order to better understand the results presented in the technical papers of this thesis, background information on some of the relevant fundamental theories, knowledge and techniques are given in this chapter. It should be emphasized that additional background information are given in Technical Papers I and II of this thesis. To avoid repetition, the information in those papers will not be discussed in this chapter.

2.1 Background on Sensitive Marine Clays in Canada

2.1.1 Geology and Distribution of Sensitive Marine Clays in Canada

2.1.1.1 Origin and Distribution of Sensitive Marine Clays in Canada

Sensitive marine clay is mainly the product of proglacial and post glacial sedimentation after the retreat of the Wisconsin Ice Sheet (Figure 2-1) 18000 to 6000 years Before Present (BP) (Quigley 1980). In general, the oldest sedimentation happened in the south region, while the youngest sedimentation happened in the north where glacial lakes still exist today.

It is estimated that the Wisconsin Ice Sheet was about 5000 m thick around 20000 to 18000 years BP. Therefore, there was a land depression of about 1000 m as a result of the ice sheet weight (Andrews 1972). The Wisconsin Ice Sheet retreated in random stages where glacial lakes or seas formed at its front. For example, the Champlain Sea covered the St. Lawrence and Ottawa lowlands. However, the sea shrunk as a result of the rise of the land. As the ice sheet retreated northward, isostatic rebound took place (Andrews and Peltier 1976; Andrews 1986). The proglacial and postglacial lakes and seas formed as a result of the volume of water produced from the ice melt, the rate of the rebound after the ice sheet retreat, and the damming of drainage terraces from these seas and lakes (Figure 2-2).

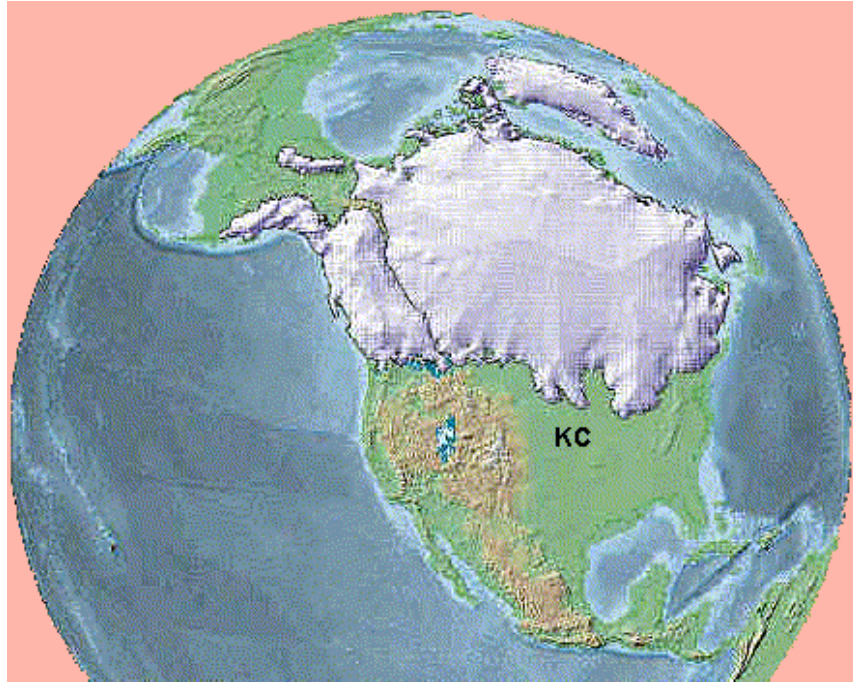


Figure 2-1 Wisconsin Ice Sheet (Aber 2005)

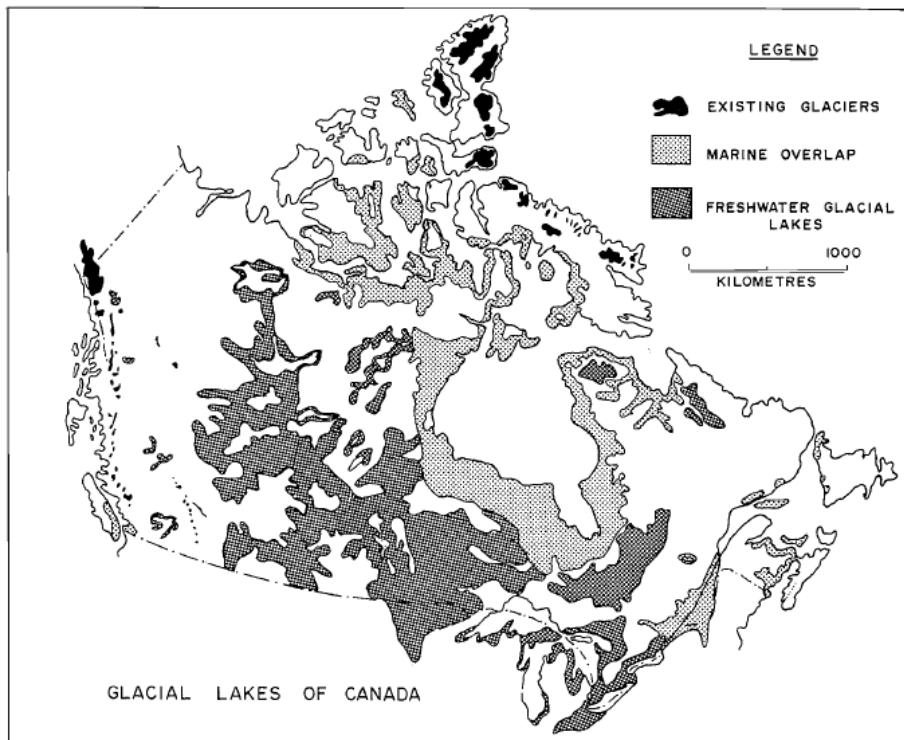


Figure 2-2 Glacial and marine distribution in Canada (Quigley 1980)

Early clay deposition took place in 13000 BP in the ice front in freshwater lakes, such as Lake Erie in southern Ontario. The clay deposit in Lake Erie is rich in illite, chlorite, calcite, and dolomite because its basin is dominated by Paleozoic shales and carbonates from the glacial erosion that produced these clay minerals. Chlorite, for example, can be oxidized into smectite through erosion. Smectite causes more activity in soil than chlorite which results in active surface soil compared to deeper soils (Fanning and Jackson 1966; Quigley 1976). At 11800 BP, there was an ice front overriding from the Great Lake into the clay sedimentation in the Champlain Sea, Lake Algonquin, and Lake Agassiz (Dreimanis 1977). Lake Algonquin covered the Georgian Bay, Lake Simcoe, and western Superior areas. Lake Agassiz covered the area that extended from Lake Nipigon and Hudson Bay into the United States (Figure 2-3).

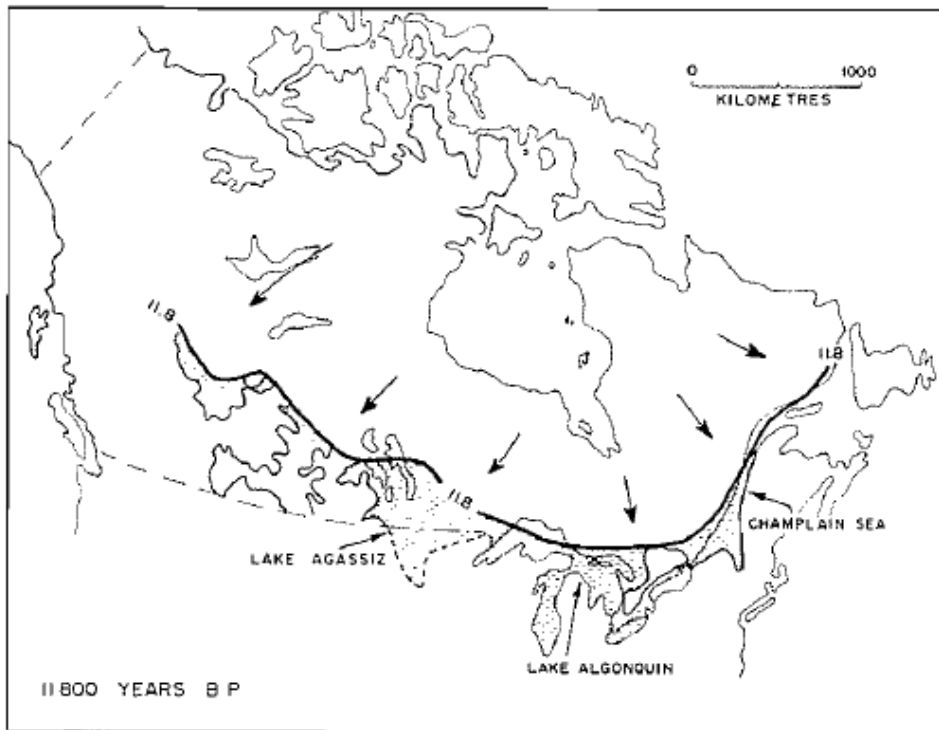


Figure 2-3 Ice front at 11800 years and Lake Agassiz, Lake Algonquin, and Champlain Sea (Quigley 1980)

The Champlain Sea covered the St. Lawrence lowlands (including Ottawa) from 12500 to 10000 years BP (Elson 1967; Gadd 1975; Cronin 1977). The Champlain Sea is believed to have invaded the St. Lawrence lowlands from the melting of an ice dam near Quebec City. The Champlain Sea mixed with fresh water, and marine clays were deposited. In addition, an ice

front formed at the north-west boundary of the Champlain Sea. The ice front experienced cycles of melting and freezing in some areas of the boundary, thus causing clay till deposition in those areas. These cycles caused layers of marine and fresh water deposits. A point to mention is that deep clay deposits can be found away from the boundaries of the Champlain Sea, but interlayer deposits of clay, sand and gravel (of fluvial or turbidity origins), and/or glacial till can be found near the boundaries.

Sedimentation in Canada comes from two major sources, which are the igneous rocks of the Canadian Shield or metamorphic rocks of the Appalachians. The sedimentation of the Champlain Sea is mainly derived from the Canadian Shield. As a result, the primary minerals of such deposits are quartz, feldspar, amphibole, mica, chlorite, smectite, and glacial amorphous material (Brydon and Patry 1961; Soderman and Quigley 1965; Gillott 1971; Bentley and Smalley 1978; Hendershot and Carson 1978; Yong et al. 1979). Also, carbonates are present in the clays that were affected by older limestones.

The Champlain Sea was expelled and the Laflamme Sea in the St. Jean region of Quebec started at around 10000 years BP by an ice front. Deep varved clay deposits in the Northwest Territories and the Prairie provinces are the result of this ice front (Quigley 1980).

As a result of the ice sheet weight, soil can be normally consolidated or overconsolidated. For example, the Cochrane glacial ice sheet readvance that occurred at 8200 years BP covered areas south of James Bay (Boissonneau 1966). It was noted that the same soil was overconsolidated to the north of the ice front while normally consolidated to the south of the ice front. It was also noted that there is a small clay proportion in the Cochrane area which leads to the conclusion that the ice retreat in that area was rapid.

The final sedimentation of the clay deposits in Canada happened in the Tyrell and Iberville Seas around the Hudson Bay. There was an extensive clay deposit in the southern regions in Fort Rupert (Ballivy et al. 1971; 1975). There was also a land rise of about 200 m after the glacial retreat which resulted in the exposure of marine deposits to weathering factors (Figure 2-4).

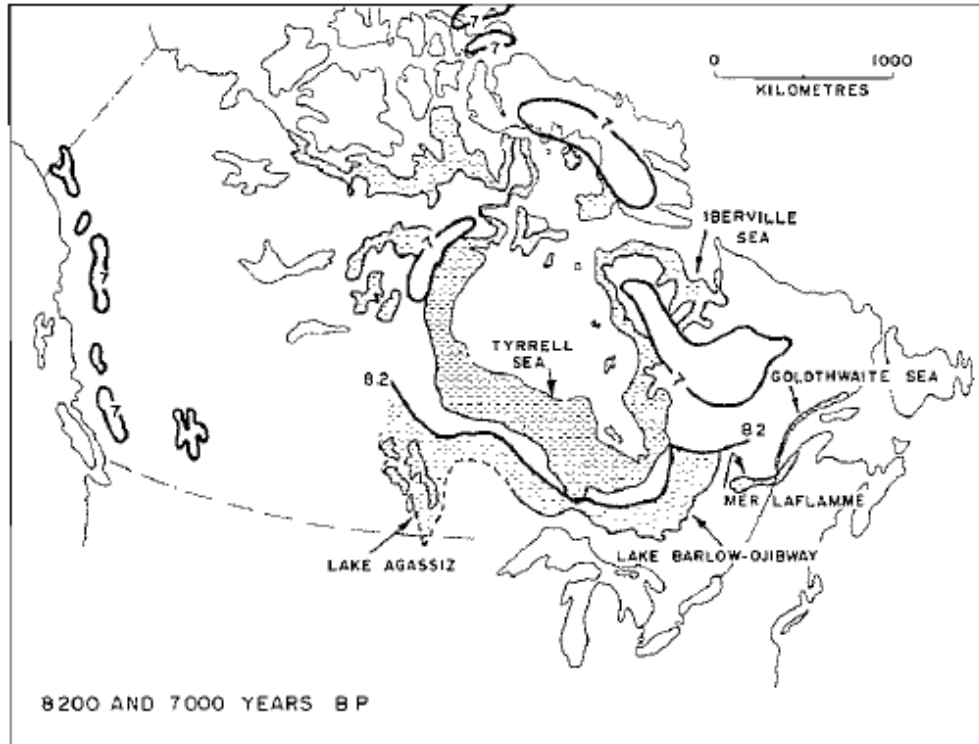


Figure 2-4 Iberville Sea, Tyrrell Sea, Laflamme Sea, Lake Barlow-Ojibway, and Lake Agassiz (Quigley 1980)

2.1.1.2 Sedimentology of Sensitive Marine Clay

Marine clays in Canada are the result of three main types of sedimentation processes which are waterlaid tills, lacustrotills, and mudflows. Waterlaid till is a stratified variety of till deposited in water that usually overlies hard till (Dreimanis 1976). Waterlaid till is lacustrine clay deposited below a shallow floating ice sheet, so grain size segregation is minimal (May 1977). Waterlaid till may have varied in depth from 1 m as in Alberta (May 1977) up to 30 m as in Sarnia, Ontario (Quigley 1976). On the other hand, lacustrotills are sediments deposited in a lacustrine environment or by a flow mechanism. Lacustrotills are submarine mudflows in glacial lakes that may include waterlaid tills. Lacustrotills may have dispersed towards the shores to form turbidity current deposits (interlayered with varved deposits) if the mudflow has enough momentum to travel under water (Morgenstern 1967).

Varved clays are layers of sediments that were deposited into glacial lakes. Each layer represents one year of silt deposition in the summer and clay deposition in the winter. In the summer, when

the cold inflow (0-6°C) from a storm or heavy ice melt of high density (1 g/L or more) entered the glacial lake, heavy density flow would occur as shown in Figure 2-5. The reason is that water at this temperature is close to its maximum density. This turbidity is able to flow for miles even on flat lake floors. The resulting deposits were graded silt sand and silt that might have varied between days to short daily periods (Kenney 1976).

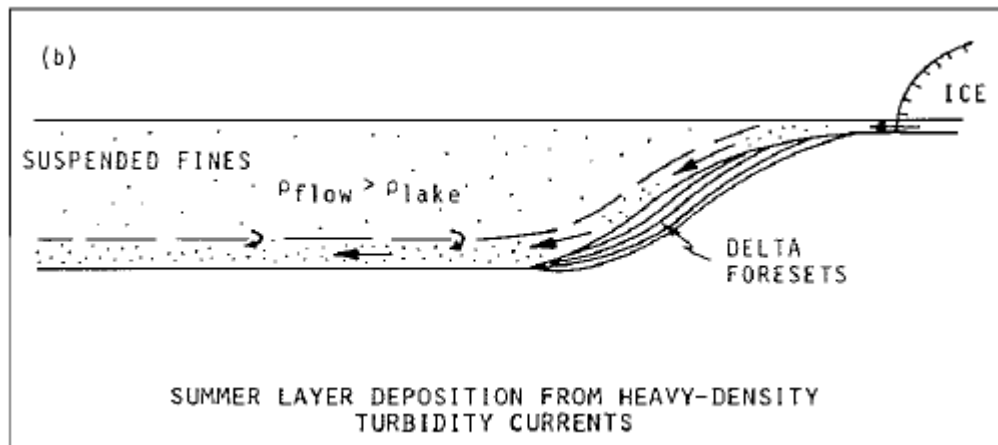


Figure 2-5 Summer heavy density flow into glacial lakes (Quigley 1980)

In the winter, the inlet flow is of low sediment density. Therefore, overflow would occur, which resulted in undisturbed clay particle sedimentation as shown in Figure 2-6.

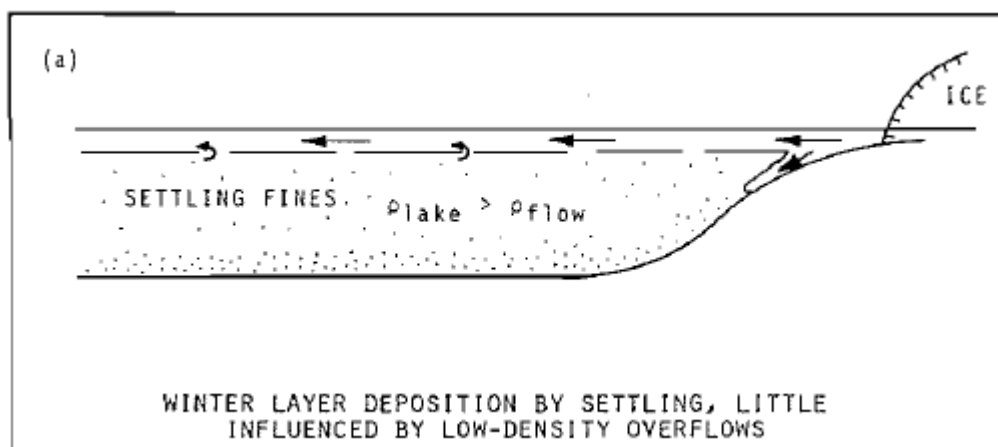


Figure 2-6 Winter overflow into glacial lakes (Quigley 1980)

In postglacial lakes, the sediment density in the water was about 0.1 g/L which is low density relative to proglacial lakes. As a result, inlet flow would enter the lakes with inflow or overflow

turbidity. Heavy density turbidity would only occur in the case of flooding or submarine slump. For example, in the Hector Lake in Alberta, summer and winter inlets inflow and overflow the lake, and thus override the heavier cold water (Smith 1978). However, the deposition is similar to that in proglacial lakes as it also consists of fine sands and silts in the summer and clays in the winter. However, this varved deposition happens only near the inlets on the stream to the lake. The deposition is deep clay after only 2 km away from the delta in Hector Lake (Smith 1978).

The structure of a layer of varved soil for one year consists of silt (80% $>2 \mu\text{m}$) and clay (80% $<2 \mu\text{m}$) layers, and the transition layer in between (Figure 2-7). The transition and fine layers are the results of sedimentation in the autumn and winter. The high moisture content can be explained by the open flocculation of the clay structure (Quigley 1976).

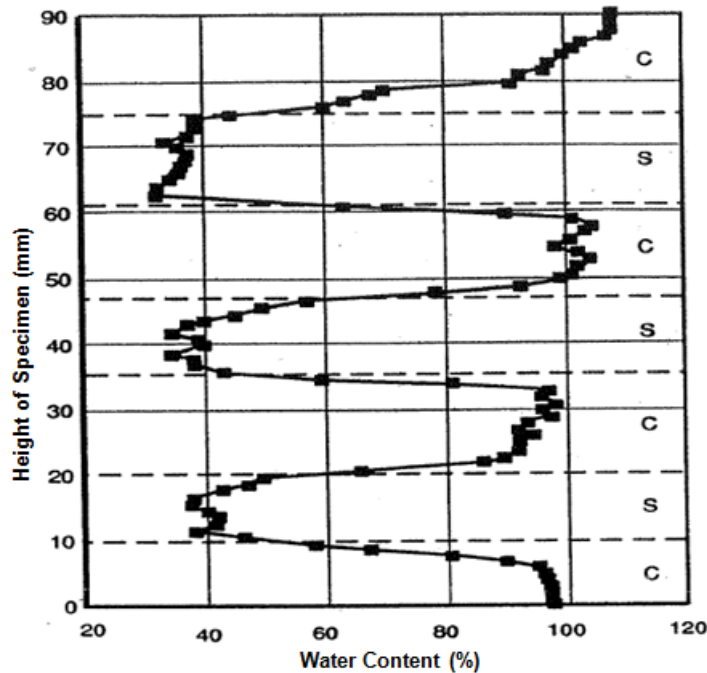


Figure 2-7 Water content profile for varved clay (Leroueil 1999)

In the case of seas bounded by ice fronts (the Champlain Sea, for example), all freshwater streams would have entered the seas as overflows. The reason is that the sea water is high in density as a result of dissolved salt (1020 g/L or 35% salinity), while the melt water is low in density (1 to 2 g/L). The surface freshwater flow will be mixed with the saltwater by diffusion and turbidity up to a depth of 5 m. Over a depth of 5 m, the flocculation of the clay particles

occurs. Below a depth of 5 m, biological activities modify the clay particle flocculation until sedimentation at the floor of the lake (Syvitski 1978; Syvitski 1980; Syvitski and Murray 1980).

2.1.2 Chemical, Mineralogical and Structural Characteristics of Sensitive Marine Clays

The chemical and structural characteristics of Ottawa sensitive clays are reviewed in Technical Paper II. They will not be discussed in this chapter to avoid repetition.

2.1.2.1 Mineralogical characteristics

The main minerals that could be found in sensitive marine clays are quartz, plagioclase, feldspar, amphibole, calcite, dolomite, phyllosilicates and amorphous matter (Leroueil 1999). It was found that tectosilicate minerals, such as quartz, feldspar, and plagioclase, dominate the mineral content of the Champlain Sea clay (Locat 1996; Torrance 1988; Berry 1988). Quartz, feldspar, and plagioclase minerals differ in quantities from one region to another. So, any one or two of them could be present in more dominant quantities than the others. However, amphibole is present in trace quantities or less than 2%. By dividing the soil into three size fractions, tectosilicates dominate the $> 4 \mu\text{m}$ fraction, clay minerals dominate the $< 2 \mu\text{m}$ fraction, and there are different quantities of tectosilicates and clay minerals in the 2-4 μm fractions. There is a variation of clay minerals and tectosilicate amounts with depth in the 2-4 μm fractions. These results are similar in samples obtained from Gloucester, Henryville, and the St. Barnabe, and Ste-Seraphine regions (Berry 1988; Bentley and Smalley 1978). For clay minerals, in some sites such as St. Barnabe, illite is present in more dominant abundance than chlorite and expansive clay minerals such as montmorillonite. In addition, there is variation of clay minerals with depth. It was observed that when the amount of illite increases, the amounts of the other clay minerals decrease. In other sites such as Henryville, illite and chlorite are dominant in samples from all depths, but expansive clay minerals dominate the first meter of the surface layer and the base of the deposit soil. This indicates that the surface layer had been subjected to weathering and leaching.

Aluminum and iron oxides increase as the particle size decreases as shown in Figure 2-8. Oxides provide coating around clay minerals and contribute to bond mechanism and cementation (Yong and Silvestri 1979). Magnetite was detected at different depths and regions, but not at a depth of

1 m in the St. Barnabe as a result of oxidative destruction by weathering. Therefore, it is believed that clay minerals are the result of deposition and post-depositional weathering (Berry 1988).

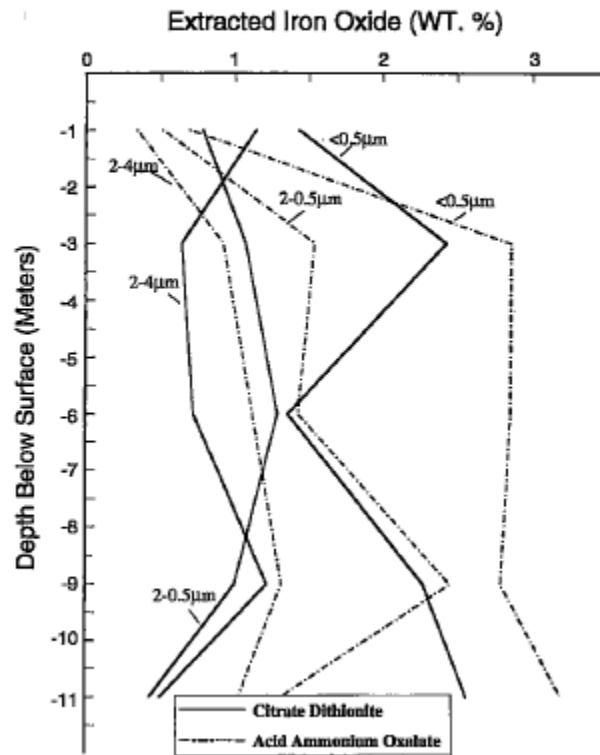


Figure 2-8 Iron and aluminum oxides with depth (Berry 1988)

2.1.2.2 Porewater chemistry

According to a study by Torrance (1979), the pre-Ottawa River played a role in eroding and redepositing large areas of marine deposits. Related evidence includes abandoned channels and terraces that contain marine deposits. These deposits have fresh water in their pores although they still contain marine remains. The content of these deposits reflects the original properties, deposition environment, and weathering. However, it is difficult to distinguish between deposited and redeposited clays because the same characteristics could be the results of weathering or leaching. The redeposited soil has closely spaced fissures, iron stains on the

fissures, low sensitivity, and little or no carbonate content as opposed to the deposited soil. As a result, salinity would be high in the marine deposits, but low in the fresh water deposits.

There is a minimum concentration of salinity in the porewater of the soil that can induce flocculation. This minimum is 2-3% on average and depends on the concentration of the suspended material.

In general, information about the salinity concentration can be obtained from the porewater of the soil or fossil evidence. The actual salinity concentration in Ottawa is about 21%. However, the fossil concentration is higher than the actual concentration which means that leaching had taken place.

In Ottawa, four sites were studied, including Treadwell, Touraine, Chelsea, and Angers. These sites are representative of the general marine clay deposits in Canada. The first site is marked as having high salinity and the other three sites as having low salinity due to leaching and weathering. Borings were obtained from different depths on each site.

In the Treadwell site, the boring was obtained in the southern part of the Ottawa River. Figure 2-9 illustrates the boring profile.

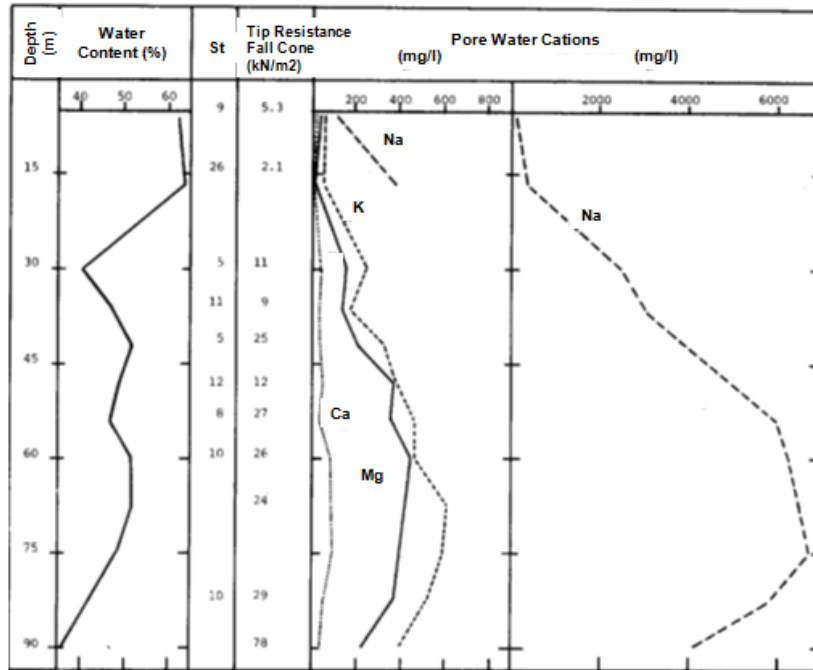


Figure 2-9 Boring profile of cation concentration and other parameters for Treadwell-Ottawa where St depicts the sensitivity (Torrance 1979)

Four salt cations, which are sodium (Na), potassium (K), calcium (Ca), and magnesium (Mg), seem to follow the same pattern with depth. The pattern seems to be an increase in concentration to about a depth of 75 m, and then a decrease in the concentration.

The increasing concentration of salts downwards to a depth of 75 m of the soil layer suggests that there had been salt removal by surface water flow and diffusion towards a low salt concentration at the surface. Also, the decrease in salt at depths greater than 75 m could be explained by the diffusion of the salts towards the bedrock, which suggests that there had been water flow at the bedrock (Torrance 1979).

In the 6-18 m layer, there are fissures and decreases in Ca and Mg concentration, unlike the increase of salinity in the general profile up to a depth of 75 m. This suggests that weathering had taken place in this layer. In addition, there could have been downward water movement controlled by the nearby Ottawa River which is 10 m deeper than the current surface elevation of this location (Torrance 1979).

In the Touraine site, similar to the Chelsea and Angers sites, the boring was obtained to a depth of 25 m as shown in Figure 10. This is the area where the two bodies of the Ottawa and the Gatineau Rivers confluence during the period when the Ottawa River was flooding 70 m above the sea level or 25 m above the current level (Figure 2-10).

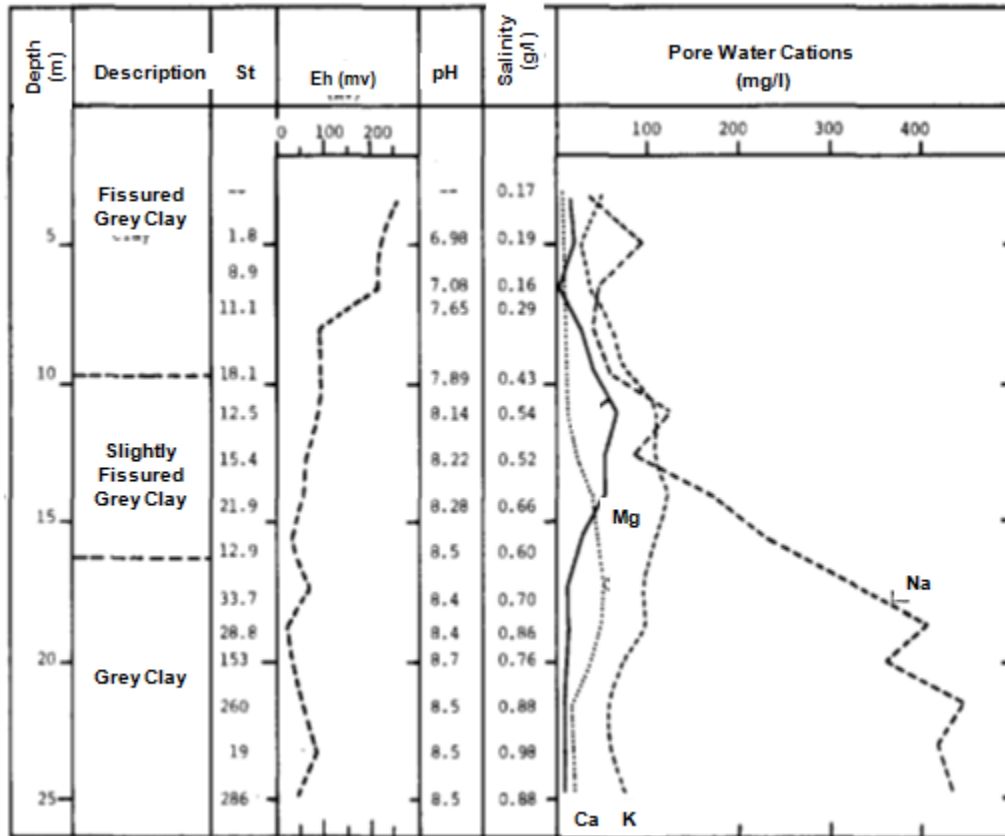


Figure 2-10 Boring profile of cation concentration and other parameters for Touraine-Ottawa where St depicts sensitivity (Torrance 1979)

In this location, the soil is marine clay covered with sand in most areas. There is clearly the development of cracks at depths of 9-10 m, then weak cracks at depths of 10-16 m, and finally, no cracks over a depth of 16 m. This implies that the soil layer experienced leaching by means of rain water infiltration. More evidence of weathering can be found in the concentration of oxidation and desiccation reactions which significantly changed in the transition layers at 9-10 m and under 16 m. It is worth mentioning that at a depth less than 16 m, the concentrations of Ca and Mg increased and the pH became constant which are indications of mild weathering. Also, the sensitivity is mild at a depth greater than 16 m and higher at a depth less than 16 m.

All four sites had experienced leaching. An uplift of the regions of the Champlain Sea occurred which caused these regions to be exposed to humidity and weathering. The domain leaching mechanism is rain fall percolating through the soil and washing salinity downwards.

A second mechanism that can cause leaching is the hydraulic pressure at the base of the soil layer as a result of the weight of the overburden soil. This mechanism depends on the hydraulic gradient and the hydraulic conductivity of the soil (Torrance 1979).

A third mechanism that can cause leaching is that salinity can decrease by diffusion and movement towards low concentration zones. This type of leaching is slow compared to rain fall percolation and hydraulic gradients (Torrance 1979).

In the case of rainfall leaching, the diffusion would reduce the rate at which the salt is being removed. So, the rain fall will be washing the salinity downwards while diffusion will be redistributing the salinity so that it reaches equilibrium in the soil.

The remolded shear strength of the soil decreases as the salinity decreases. Also, the sensitivity increases as the salinity decreases. So, leaching may increase the sensitivity and the soil may behave like a liquid when a low salinity concentration is reached. For example, at the Treadwell site, the sensitivity was about 12 at the highest salinity concentration, but reached 26 at a depth of 15 m where the salinity was 1.5% (Torrance 1979).

Laboratory tests showed that the addition of salt to marine clays increases the remolded shear strength at constant water content. In other words, the remolded shear strength of the Chelsea landslide increased from 0.1 to 1.5 kN/m² by only adding salt. However, the salinity concentration is not the only factor in clay sensitivity (Torrance 1979).

Salinity increase may also occur in fresh water that covers marine deposits. It is unlikely that salinity had affected the surface layers because there is hydraulic pressure and downward movement from the water body. Salinity may also move by diffusion from the marine deposits to the fresh water body or to the underlying soil layers (Torrance 1979). Also, the reason behind low sensitivity at great depths could be the increase of salinity or the overburden weight of the top soil layers.

If leaching starts from the surface, the salinity concentration would start at a minimum from the surface and increase with depth. The extreme effects of weathering are at the first 1 m layer of the soil. Looking at the salinity concentration profile in all of the sites, the salinity increased from the surface to a certain depth, and then decreased. It is believed that the maximum salinity concentration existed at the surface at some point in time, and then reduced due to weathering and leaching.

At Touraine, Chelsea, and Angers, when the Ca and Mg concentrations increased, the sensitivity also increased. Also, at Angers and Touraine, the sensitivity increased with an increase in the Na concentration.

Samples were taken from Angers at depths of 18 and 30 m. The samples were examined after extruding directly and after a period of two months. The samples were kept at high humidity and a constant temperature of 7°C. In a comparison of the results from right after extruding to after two months, the shear strength increased, which means that the sensitivity decreased and the K and Na concentrations increased. However, the water content, and Ca and Mg concentrations remained constant. Therefore, even though the Na concentration increased, the sensitivity decreased.

2.1.2.3 Structure of Sensitive Marine Clay

According to a study by Penner (1965), sensitive marine clay consists of negatively surface charged particles. These negative charges attract positively charged hydrogen ions of the water molecules. The bonding between the clay particles and water molecules causes repulsion between the soil particles, and therefore the swelling of the soil.

A double layer concept was proposed. The concept stated that there is a fixed negatively charged layer at the surface of the clay particles. Then, the charge potentially drops exponentially in moving away from the centre of the soil particles. After the fixed negatively charged layer, there is a stern layer and then a positively charged diffusion layer. The diffusion layer attracts negative electrons or cathodes, such as water molecules. The negative particles in the absorbed water layer are called the immobile layer. The immobile layer attracts positive electrodes or anodes (Figure 2-11).

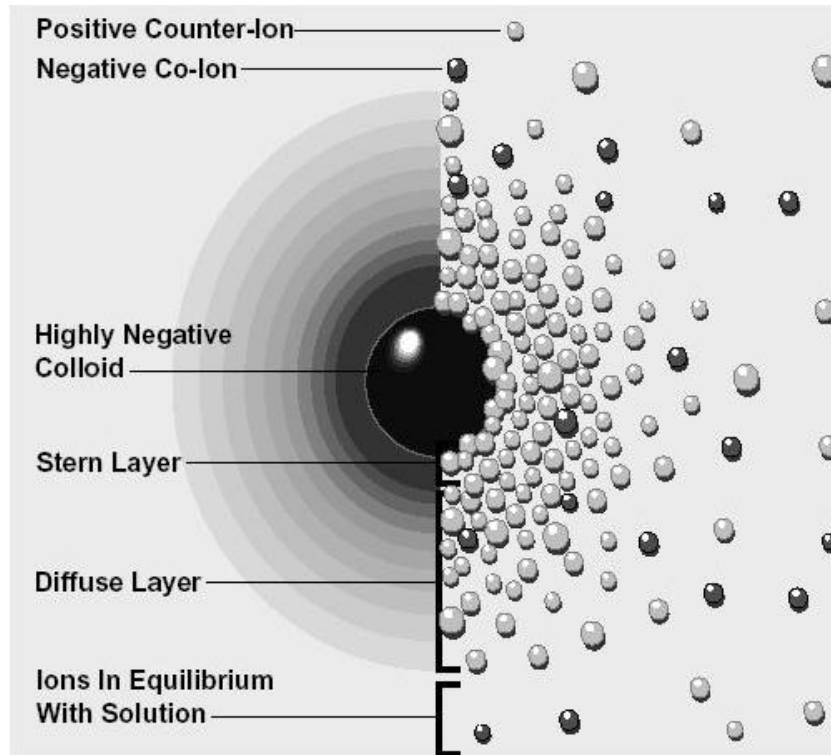


Figure 2-11 Double layer concept

The potential difference between the clay particles and the surrounding liquid is called the electrokinetic potential. If the electrolyte concentration of the water increases, the diffusion layer compresses and the repulsion between the soil particles decrease.

Electrokinetics could be measured by four means: electro-osmosis movement of liquid relative to solid by an external electric field, electrophoresis movement of solid relative to liquid by an external field, streaming potential movement of liquid to solid by mechanical means, and sedimentation potential of solid by mechanical means. The electro-osmosis method is preferred because it does not involve the changing of the original structure of the soil like in electrophoresis where the soil has to be diluted which will change its electrokinetics.

Sensitivity and electrokinetics depend on the nature of the electrolytes and the soil. The sensitivity of sensitive marine clay fairly increases with electrokinetics. Also, soils with lower than average surface area are more sensitive than fine soils at the same potential. In addition, even though sensitivity is fairly related to electrokinetics, sensitive marine clay exhibit sensitivity behavior unrelated to electrokinetics. Electrokinetics is well explained by the double

layer theory. The structural break down of sensitive marine clay is easy to achieve, but recombining the particles in random open structures is difficult due to the electrokinetic potential.

The structural behavior of sensitive marine clay can also make it prone to landslides (Roy et al. 1981). It has open flocculation at the particle level and strong cementation bonds at the interparticle level. The structural behavior of the soil is dominated by the fabric bonds between the soil particles, and these bonds could be destroyed by applying high confining pressure on the soil.

2.2 Background on Cone and Ball Penetrometers

2.2.1 Introduction

Cone penetration testing is an alternative to conventional soil investigation methods (Mayne 2009). The advantages of cone penetration testing are that it is efficient, economical, and quantitative. Another advantage of cone penetration testing is that it can collect five different readings at the same time. These readings are cone tip resistance (q_t), sleeve friction (f_s), penetration porewater pressure (u_2), rate of decay (t_{50}), and shear wave velocity (V_s). These readings are continuously directly recorded into a computer. Immediate analysis and on site decisions can be made.

The penetrometer is pushed into the soil at a constant rate of 20 mm/s. Penetrometers contain cells to measure the axial force, friction, pressure, and inclination. Tip resistance and sleeve friction are obtained from the penetrometer. Also, in high porewater pressure soils, such as silt and clay, penetration porewater pressure (u_m) is measured. Porewater pressure can be measured from the tip of the cone (u_1), from the shoulder above the cone tip (u_2), or from behind the sleeve (u_3). The recommended porewater pressure measurement is u_2 as it can be used to correct for the total tip resistance. Cone recordings such as cone tip resistance, sleeve friction, and penetration porewater pressure can be plotted with depth directly from the cone readings. Units of these readings are kPa or MPa.

2.2.2 Background on Cone Penetration Testing

In this section, the most common soil engineering properties and characteristics that are estimated from CPT readings, which include the undrained shear strength, sensitivity, soil classification, and preconsolidation history, will be discussed. These have been discussed and examined by many previous researchers, and may be affected by temperature changes from CPT readings.

2.2.2.1 Undrained Shear Strength

Undrained shear strength depends on the soil anisotropy, strain or penetration rate, and stress history. The effect of anisotropy is important for sensitive clays while the effect of the penetration rate is important for high plasticity clays. Therefore, both effects are significant for Ottawa marine clays. Empirical correlations estimate the undrained shear strength (s_u) by using the tip resistance (q_c), corrected tip resistance (q_t), and/or excess pore water pressure (Δu). Excess pore water pressure is equal to the difference between the pore water pressure measured behind the cone tip (u_2) and the hydrostatic water pressure (u_o) measured by the water level from the monitoring well readings. S_u can be estimated by using the following equations (Lunne et al. 1997):

$$s_u = \frac{(q_c - \sigma_{vo})}{N_k}$$

$$s_u = \frac{(q_t - \sigma_{vo})}{N_{kt}}$$

where q_c is the tip resistance, q_t is the corrected tip resistance, σ_{vo} is the total overburden stress, and N_k and N_{kt} are the empirical cone factors specific to the soil. S_u can also be estimated by using excess pore water pressure. The following relationship has been proposed by using $N_{\Delta u}$ as a cone factor specific to soil (Randolph and Wroth 1979; Battaglio et al. 1981; Massarsch and Broms 1981; Campanella et al. 1985; Lunne et al. 1997):

$$s_u = \frac{\Delta u}{N_{\Delta u}}$$

The cone factors specific to the soil (N_k , N_{kt} , $N_{\Delta u}$) can be estimated by referring to the undrained shear strength obtained by using field or lab vane tests, compression or tension triaxial tests, direct shear tests, or any other field or laboratory method. However, the accuracy of the testing or the quality of the samples may affect the range of the different parameters, their average, or standard deviation.

2.2.2.2 Sensitivity

Sensitivity is defined as the ratio of the initial undrained shear strength to the remolded undrained shear strength. The tip resistance and excess pore water pressure are functions of the undrained shear strength while sleeve friction is a function of the remolded shear strength. Therefore, the following relationship can be used to estimate sensitivity (Schmertman 1978):

$$s_t = \frac{N_s}{R_f}$$

$$R_f = \frac{f_s}{q_t}$$

where S_t is sensitivity, N_s is a constant, R_f is the friction ratio, and f_s is the sleeve friction. As the sleeve friction is believed to be equal to the remolded shear strength, sensitivity can also be directly estimated by using the S_u from q_c , q_t , or Δu and the sleeve friction (Lunne et al. 1997):

$$s_t = \frac{S_u}{f_s}$$

2.2.2.3 Soil Classification

The SPT is the most common field test (Lunne et al. 1997). Considerable research and work have been carried out to obtain SPT results. Therefore, it is useful to correlate SPT to CPT. Robertson and Campanella (1983) proposed a correlation between tip resistance and mean soil grain size. It was found that the mean grain size increases with increasing tip resistance with an increase in the scatter. A similar correlation was shown by Kulhawy and Mayne (1990) and a higher scatter level was found. Also, the studies showed that tip resistance decreases with decreasing fines content (grain particles that pass through a 2 mm sieve). A study by Robertson et al. (1986) proposed 12 soil behavior classification zones that are based on dimensionless stress as follows:

$$\frac{q_c / P_a}{N_{60}}$$

where P_a is the atmospheric pressure (about 101.3 kPa) and N_{60} is the number of blows from SPTs that correspond to a 60% energy ratio. However, according to this correlation, few soil classes or zones can carry the same $(q_c/P_a/N_{60})$ value. Jefferies and Davies (1991) suggested the use of a soil behaviour index (I_c) with well defined zones and soil classes from 2 to 7. Their method has been modified with the classification zones by Robertson (1990).

$$I_c = [(\log F_r + 1.22)^2 + (3.47 - \log Q_t)^2]^{0.5}$$

$$F_r = \frac{f_s}{(q_t - \sigma_{vo})}$$

$$Q_t = \frac{(q_t - \sigma_{vo})}{\sigma'_{vo}}$$

where F_r is the normalized friction ratio, Q_t is the normalized tip resistance, and σ_{vo} is the effective overburden pressure. By combining these soil classification zones with the relation of the fines to the friction ratio (F_r) provided by Suzuki et al. (1995), a relation between the CPTs and fines content (FC%) can be obtained (Lunne et al. 1997):

$$FC\% = 1.75I_c^3 - 3.7$$

Lunne et al. (1997) suggested that since soil type and the response of fines content to CPTs are affected by plasticity and the mineralogy, these correlations should be calibrated with local experience.

2.2.2.4 Preconsolidation Pressure and OCR

The OCR is the ratio of the preconsolidation pressure (P'_c) to the effective overburden pressure (σ'_{vo}) at a given depth. Many correlations have been proposed in the literature, but none apply to all soils. Lunne et al. (1997) proposed the following correlation:

$$P'_c = k(q_t - \sigma'_{vo})$$

$$OCR = k \frac{q_t - \sigma_{vo}}{\sigma_{vo}}$$

where k is a correlation factor between the normalized tip resistance and OCR. They also proposed that Q_t falls between 2.5 and 5 for normally consolidated clays and in a larger range for overconsolidated soils.

2.2.2.5 Pore Pressure Dissipation Tests

The porewater pressure measured depends on the draining conditions that surround the probe. Therefore, drainage conditions are considered to be drained when penetrating high permeability soils, such as sand, but undrained when penetrating low permeability soils, such as clay. As the cone penetrates low permeability soil, excess water accumulates, which causes high local porewater pressure around the cone. When the penetration is paused, porewater pressure starts to dissipate back into the soil with time in a process called pore pressure dissipation. Pore pressure dissipation will occur until equilibrium is reached with atmospheric pressure. Pore pressure dissipation is usually plotted in a logarithmic scale as it takes a long time. The test is usually stopped at 50% completion as it is impractical to wait for 100% dissipation. Dissipation to 50% is accomplished when half of the difference between the porewater pressure at the start of dissipation test and the hydrostatic pressure is achieved. Figure 2-12 shows a typical pore pressure dissipation test. When dealing with overconsolidated soils, dilatancy behavior is observed where peak porewater pressure is reached before the decay of the porewater pressure.

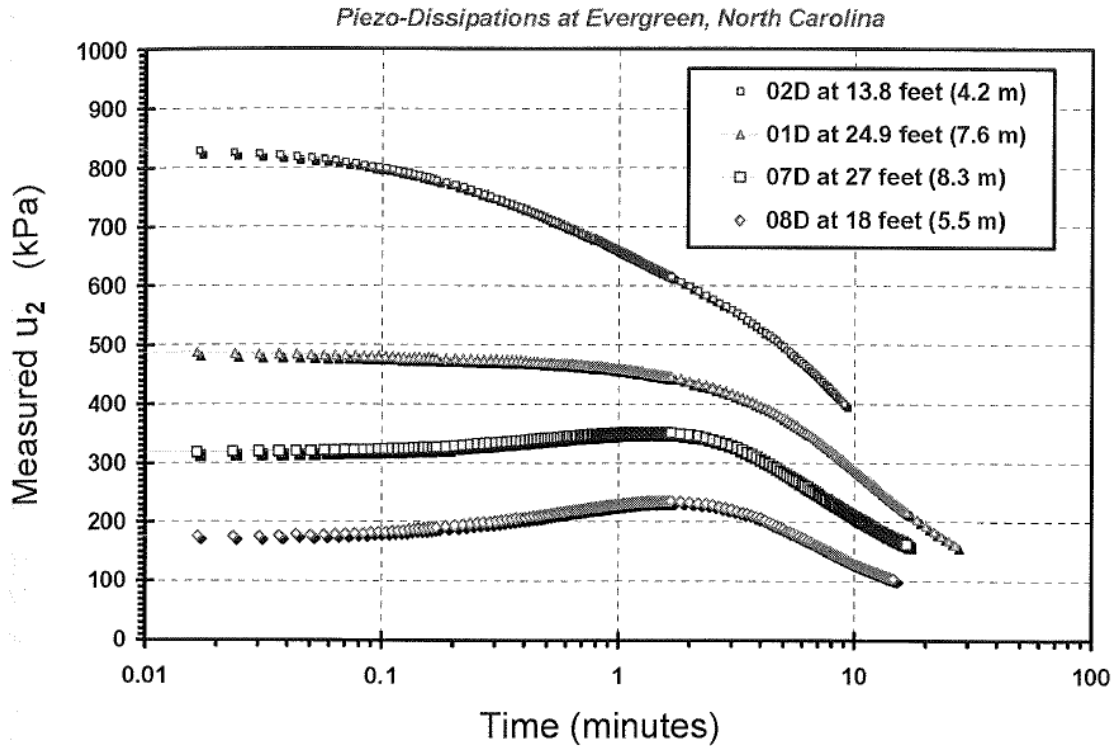


Figure 2-12 Example of a pore pressure dissipation test (Mayne 2009)

2.2.2.6 Seismic Cone Testing

Seismic cone penetration tests (SCPTu) combine the typical readings for CPTs with geophysical shear wave velocity measurements (V_s). However, tip resistance, sleeve friction, and porewater pressure are continuously measured, while the shear wave velocity is measured at certain depth points with the Down Hole Test (DHT). When the penetration is temporarily paused, a surface pulse is generated at the surface to create a shear wave in the ground. The wave is monitored by using a velocity transducer or geophone located on the penetrometer. The shear wave is vertically propagated and in a horizontal polarized mode (Mayne 2009). The plane of the surface generated wave has to be parallel to the axis of the geophone in order to reduce noise in wave detection. Multiple geophones in different axes can be used to collect information on shallow compression waves or P-waves. Figure 2-13 shows shear wave velocity with depth.

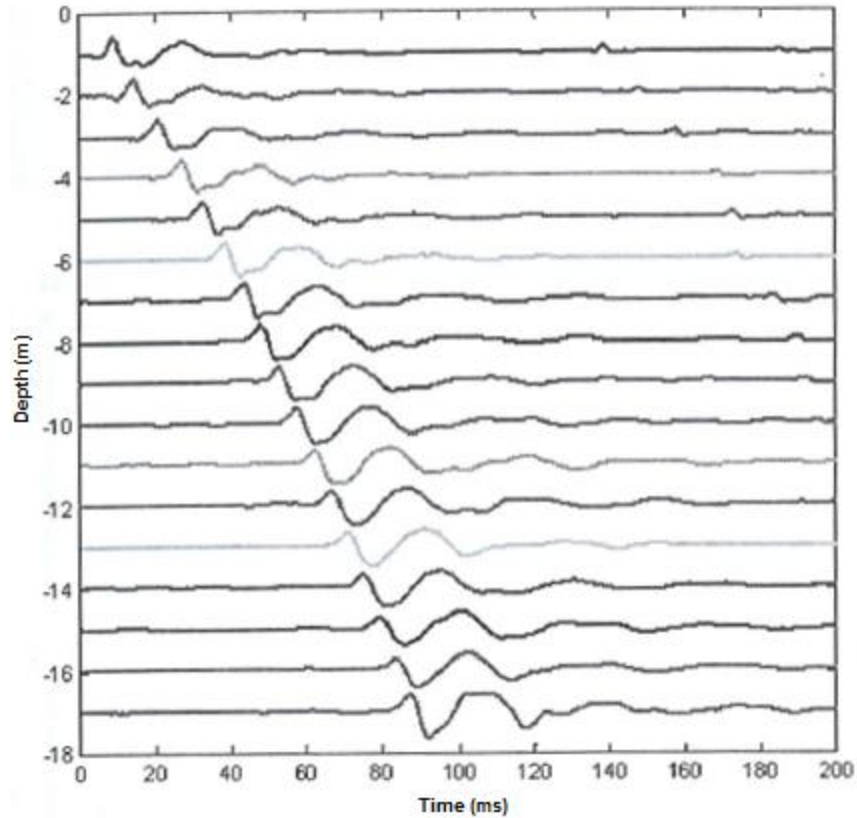


Figure 2-13 Arrival of shear waves (Mayne 2009)

2.2.2.7 Geostratification

As the continuous profiles are derived from the three CPT readings with depth, it is possible to identify the subsurface layers. Also, by analyzing the tip resistance, it is possible to detect weak zones, thin lenses, and layer consistency.

There is no sampling procedure in CPT. Indirect methods are used instead to identify soil types. Soil types can be identified through correlation with existing profiles or borehole logs, empirical soil behavior type (SBT) charts, probabilistic methods, or rule-of-thumb.

An example of a CPTu profile from New Orleans soil is shown in Figure 2-14. For tip cone resistance, values <5 MPa indicate a clay layer, while values >5 MPa indicate a sand layer. It can be seen that the first few meters of the soil alternates between sand and clay, then a thick clay layer, and then a thick sand layer. For the sleeve friction, the friction ratio was plotted with depth (friction ratio(FR%)= f_s/q_t). Sands have $FR < 1\%$, while clays have $FR > 1\%$. So, soil identification

can be verified by comparing the tip cone resistance with the FR profiles. On the other hand, looking at the porewater pressure profile, it can be seen that the water table is about 0.5 m below the surface. A reference line is drawn in the profile to represent the hydrostatic pressure (u_0). Above the water table, u_0 is usually taken as zero. However, in fine soils such as clay, it could be negative due to the capillary activity and the degree of saturation. In sands, porewater pressure is usually close to u_0 . However, porewater pressure in dense sands could be below u_0 . Clays, on the other hand, usually exhibit porewater pressure values higher than u_0 as shown in the porewater pressure profile compared to the tip resistance and the sleeve friction profiles. It is worth mentioning that stiff, fissured, and overconsolidated clays may have negative porewater pressure values.

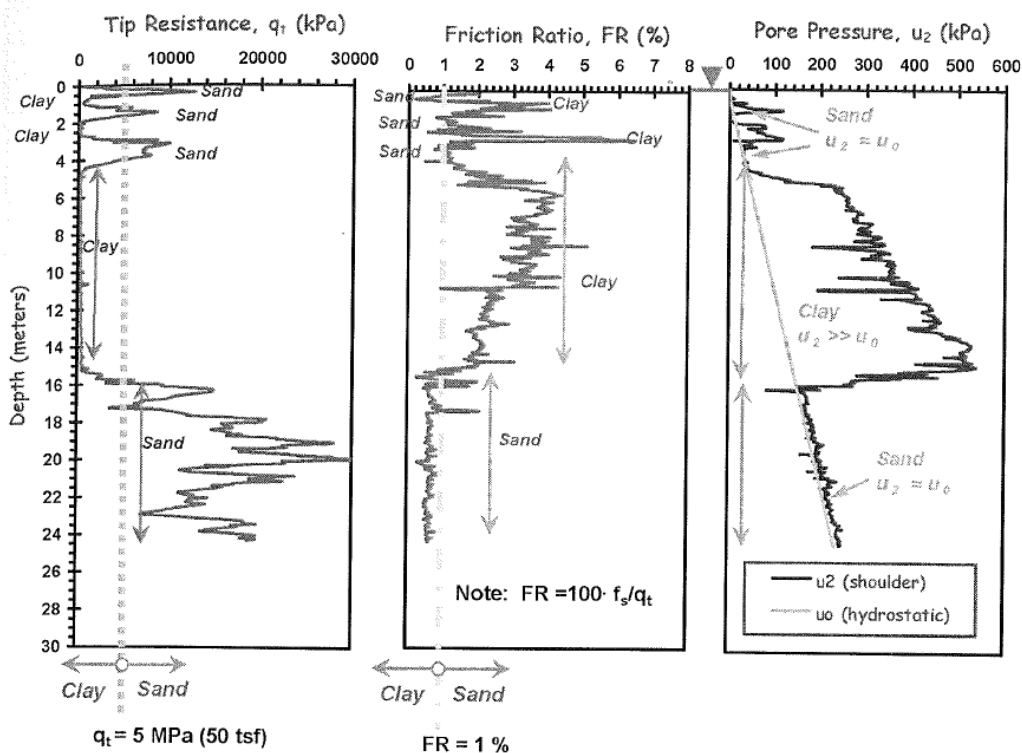


Figure 2-14 CPT results for New Orleans soil (Mayne 2009)

Computer software can be used to identify soil types. SBT charts have been proposed (Lunne et al. 1997; Robertson 2004; Schneider et al. 2008). Figure 2-15 presents an SBT zone system that depends on the CPT readings. σ_{v0} is the total overburden stress.

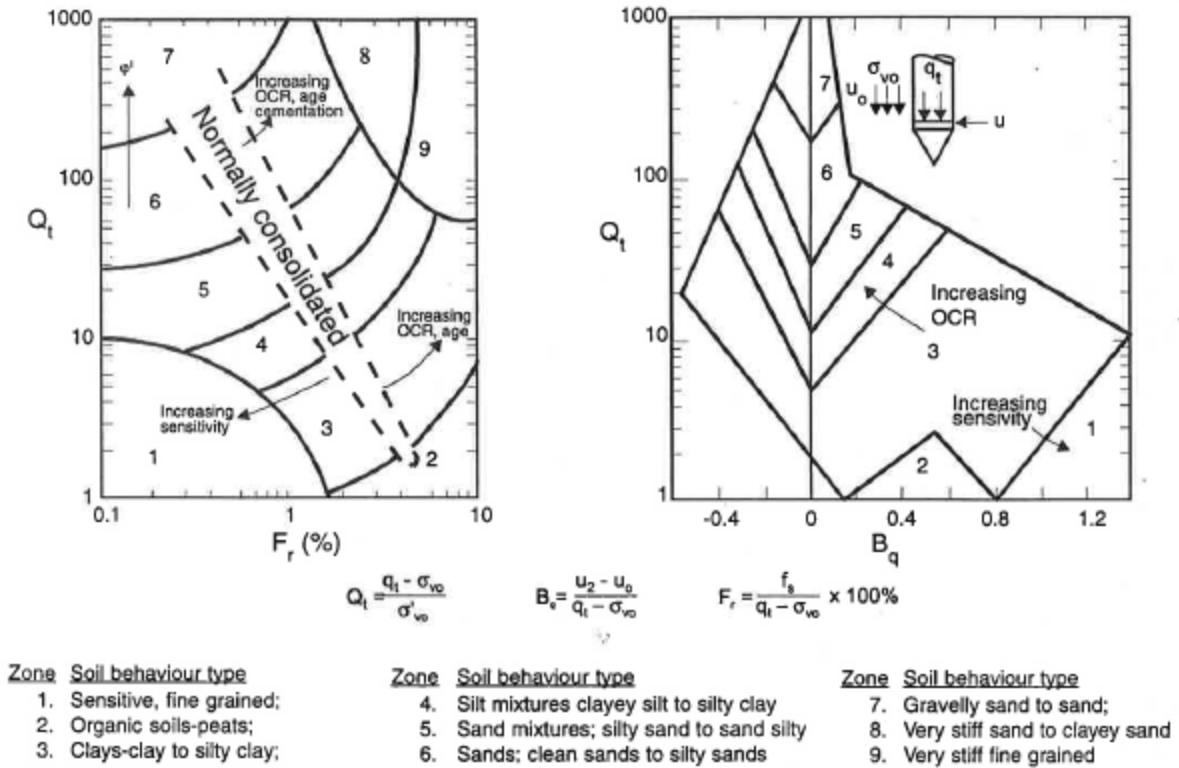


Figure 2-15 Soil identification by using soil behavior type (Lunne et al. 1997)

2.2.3 Background on ball penetrometers

The initial idea of a ball penetrometer was taken from a miniature T-bar that was used in centrifugation (Boylan et al. 2011). Later, full flow penetrometers were used to improve resistance by increasing the tip projected area (Stewart and Randolph 2001). Then, full flow penetrometers were used on offshore soils as they are a soft type of soil (Randolph et al. 1998). Ball penetrometers which were introduced as the tip of T-bar penetrometers may bend as they are driven into the ground (Watson et al. 1998). The main advantage of full flow penetrometers over conventional cone penetrometers is the increase in the projected area. This increase in the projected area reduces the effects of temperature and overburden stress as the soil would flow around the ball or T-bar tip instead of being displaced as in the case of a cone tip (Boylan et al. 2011).

2.2.3.1 Penetration Resistance Correction

The tip resistance of ball penetrometers needs to be corrected for pore water pressure and overburden stress that do not act on top of the ball because of the push rods, as soil does not flow but displaces around them (Randolph 2004; Randolph et al. 2007; DeJong et al. 2009, 2010 and 2011; Low et al. 2010; Boylan et al. 2011):

$$q_{ball} = q_m - [\sigma_v - u_2(1-a)] \frac{A_s}{A_p}$$

where q_{ball} is the corrected tip resistance, q_m is the measured tip resistance, σ_v is the total overburden pressure, a is the area ratio obtained in accordance with ASTM D 5778, A_s is the projected area of the push rods, and A_p is the projected area of the tip. DeJong et al. (2010) suggested the use of hydrostatic pore water pressure for unmeasured pore water pressure during testing in the field. However, porewater pressure at Position 2 (u_2) is measured for all of the penetrometers in this study.

2.2.3.2 Undrained Shear Strength

Undrained shear strength (s_u) can be obtained by field vane or laboratory testing. The following formula is suggested which uses N_{su} as a strength factor (Randolph 2004; Randolph et al. 2007; DeJong et al. 2009, 2010 and 2011; Boylan et al. 2011; Low et al. 2010):

$$s_u = \frac{q_{ball}}{N_{Su}}$$

Studies have shown that sensitivity and rate of penetration may affect the calculation of N_{Su} (DeJong et al. 2008 and 2009).

2.3 Correlations from the Canadian Foundation Engineering Manual

Table 2-1 is taken from Table 3.3 in the Canadian Foundation Engineering Manual. This table correlates the undrained shear strength of clays to N60 values from standard penetration testing. Table 2-2 is taken from Table 6.1A in the same manual. This table correlates the shear wave velocity from downhole seismic testing and undrained shear strength (from vane tests for example) to site class.

Table 2-1 Table 3.3 in the Canadian Foundation Engineering Manual (2005)

Consistency	Undrained Shear Strength (kPa)	SPT N-Index (blows/0.3m)
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Table 2-2. Table 6.1A in the Canadian Foundation Engineering Manual (2005)

		Average Properties in Top 30 m as per Appendix A NBCC 2005		
Site Class	Soil Profile Name	Soil Shear Wave Average Velocity (m/s)	Standard Penetration Resistance N_{60}	Soil Undrained Shear Strength S_u
A	Hard Rock	$V_s > 1500$	Not applicable	Not applicable
B	Rock	$760 < V_s < 1500$	Not applicable	Not applicable
C	Very Dense Soil and Soft Rock	$360 < V_s < 760$	$N_{60} > 50$	$S_u > 100$ kPa
D	Stiff Soil	$180 < V_s < 360$	$15 \leq N_{60} \leq 50$	$50 \text{ kPa} \leq S_u \leq 100$ kPa
E	Soft Soil	$V_s < 180$	$N_{60} < 15$	$S_u < 50$ kPa
E	Any profile with more than 3 m of soil with the following characteristics:			

- Plastic index >20
- Moisture content $\geq 40\%$, and
- Undrained shear strength < 25 kPa

F

Others

Site specific evaluation required

Other correlations between penetrometers, and other laboratory and field testing, and engineering characteristics of sensitive marine clays, are briefly reviewed in Technical Papers I and II.

Therefore, they will not be reviewed here to avoid repetition.

2.4 Summary and Conclusion

Sensitive marine clay is mainly the product of proglacial and post glacial sedimentation after the advancement of the Wisconsin Ice Sheet. Postglacial depositions of Ottawa sensitive marine clays are predominantly done so by the Champlain Sea which covered Ottawa from 12500 to 10000 years BP.

As a result of proglacial and postglacial deposition, varved marine clays formed. A layer of one year of deposition consists of the silt (80% $>2 \mu\text{m}$) and clay (80% $<2 \mu\text{m}$) layers, and the transition layer in between.

The main minerals of the Ottawa clays are quartz, feldspar, amphibole, mica, chlorite, smectite, and glacial amorphous material. Carbonates may also be present in these clays. Three main types of sedimentation processes in general, are responsible for marine clays in Canada, which are waterlaid tills, lacustrotills, and mudflows.

Salinity and other cations, such as Na, K, Ca, and Mg, affect the behavior and sensitivity of marine clays. A double layer of marine clay particles also affect the behavior of these clays.

Penetrometers are increasingly being used in marine clays. Correlations have been established by researchers to relate the outcomes of penetrometer testing and the engineering characteristics of these clays.

The behaviour of sensitive marine clays in Ottawa is still not well understood. Readings by cone and ball penetrometers are significantly affected by temperature changes. The rate of rotation for

the field vanes may not be adequate to estimate the remolded shear strength of these clays. Laboratory vane results may misestimate the shear strength of these clays when compared to that of field vanes. Also, from the results of this research, the Canadian Foundation Engineering Manual may have underestimated the undrained shear strength of sensitive marine clays in Ottawa. These are the reasons that this research has been conducted.

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Chapter 3: Technical Paper I - Geotechnical Properties of Sensitive Marine Clays - Examples of Clays in Eastern Canada

N.Athir, M. Fall, R. Hache

Abstract

This article presents a comprehensive description and review of the geotechnical properties of sensitive marine clays in the Canadian capital region. The general behaviour of these clays is discussed which provides the engineers who work with these clays with a better understanding of the geotechnical properties of sensitive marine clays in eastern Canada. Fifteen sites are taken into consideration. Geotechnical investigations have been carried out in these sites. These geotechnical investigations included cone and standard penetration tests, split spoon and undisturbed sampling, field vanes, and shear wave velocity measurements. Laboratory tests are also carried out on the samples. These laboratory tests include consolidation testing, determination of grain size distribution, porewater chemistry analysis, and determination of physical and index properties. This research compares these results with available data from other research along with other characteristics, such as sensitivity, activity, interface shear strength, and hydraulic conductivity, depending on the availability of data from each site.

It is found that the Canadian Foundation Engineering Manual may have underestimated the undrained shear strength of these clays. The moisture contents of these clays are close to or exceed the liquid limits which may contribute to the sensitive behaviour of sensitive marine clays in Ottawa. The activities of these clays fall in the normal and inactive areas of the activity chart. The ratio of the secondary compression index to the compression index presented in the literature for marine clays in Canada may not be applicable for sensitive marine clays in Ottawa. It is also discussed that laboratory vanes may have overestimated or underestimated the undrained shear strength compared to field vanes. A wide range of soil specific cone factors is also presented in this research.

Keywords

Engineering characteristics, Laboratory testing, In-Situ, Gloucester, Ottawa, Leda Clay, Sensitive Marine Clay

3.1 Introduction

Marine clays cover large areas in the world, such as Canada, Norway, Sweden, Scandinavia, and Russia. In Canada, marine clays cover large areas in the provinces of Ontario and Quebec. These clays are commonly called Leda clays in Ontario and Champlain Sea clays in Quebec. The soft consistency and sensitive behavior of these clays make them challenging to geotechnical engineers. These clays can change from a solid to liquid consistency only by disturbance which makes them prone to landslides and foundation damage.

It is important to increase local experience on the behavior and engineering characteristics of marine clays. There is a demand for local experience as a result of the infrastructure expansion in the capital region. Many researchers have examined the engineering characteristics of marine deposits in the Canadian capital region. However, there has not been an attempt to collect and analyse the available data for reference. Consequently, this is the reason for the initiation of this research.

Marine deposits in Ottawa exhibit unique behaviour that is different from similar deposits in different parts of the world or even different parts of Canada. Therefore, the correlations that are made on similar marine deposits should be examined and verified for marine deposits in Ottawa. Fifteen sites have been selected from the Canadian capital region. These sites were originally investigated for geotechnical design purposes. Field investigations and laboratory tests are carried out on the sites and samples obtained from them. The field tests include cone and standard penetration testing, split spoon and undisturbed sampling, and field vanes and monitoring well testing. Laboratory tests include testing of consolidation, grain size distribution, porewater chemistry, specific gravity, plasticity, moisture content, and unit weight. This research compares the outcomes of field and laboratory tests with previously reported results. The work analyzes plasticity, activity, sensitivity, hydraulic properties, shear strength behaviour and interface, and physical characteristics. It also reports on the outcomes of the cone and standard

penetration tests, and vane tests. The study reports the values of the consolidation history of the soil obtained from the site under study including Canadian Geotechnical Research Site No. 1 (or Site 6 as named in this research). This research also discusses the underestimation of the shear strength of sensitive marine clays in Ottawa by the Canadian Foundation Engineering Manual.

This paper is organized as follows: first, the geographical and geological characteristics of the area under study are discussed. Second, the physical characteristics, Atterberg limits, and activity of sensitive marine clays in Ottawa are examined. Then, the geotechnical in-situ testing is presented. This is followed by an outline of the shear strength and deformation behaviour. After that, the interface shear strength and behaviour are discussed. Then, the consolidation behaviour is presented, followed by a discussion on the sensitivity of marine clays in Ottawa. Finally, the permeability will be examined.

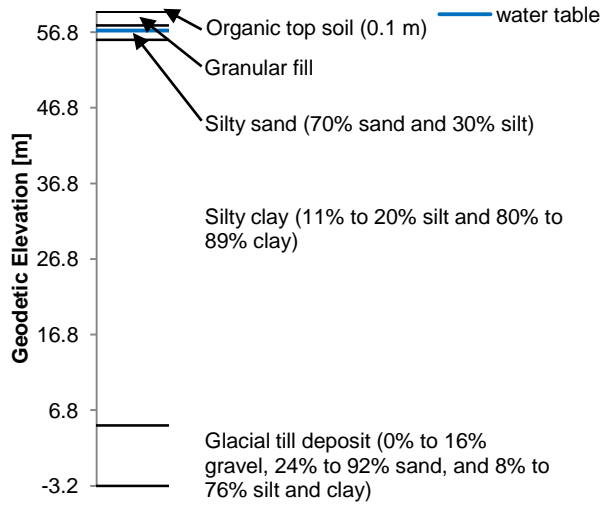
3.2 Geographical and Geological Characteristics

3.2.1 The Sites

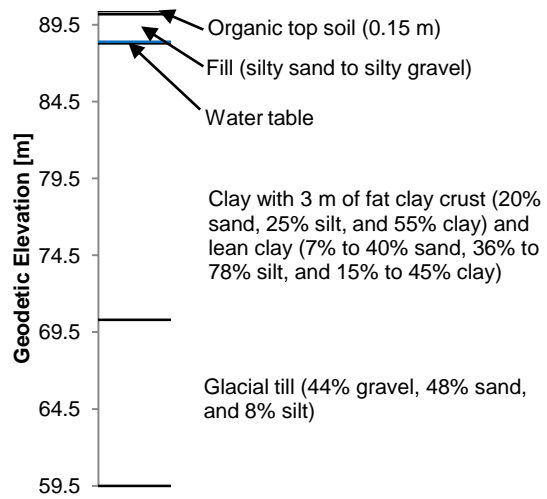
Fifteen sites (Sites 1 to 15) are selected for this study. The locations of the sites are shown in Figure 3-1. The sites are well spread out on the most developed areas in Ottawa which allow for a rather comprehensive study of the marine deposits in the region. All of the site stratifications from the geodetic surface elevation to the bedrock are shown in Figure 3-2. However, no dimensions are given for Sites 14 and 15, but the stratification on these two sites from the surface to the bedrock are as follows: silty sand to sand and gravel fill with traces of organics in some areas of the sites, clay with 0.9 m of crust (0% to 1% gravel, 2% to 11% sand, 18% to 26% silt, 62% to 80% clay), and till or silty sand with gravel and traces of cobbles and boulders (10% to 22% gravel, 39% to 48% sand, 30% to 51% silt and clay). Bedrock is found at depths of 2.1 to 7.6 m. The sites have geodetic elevations of 66.4 m to 71.5 m and water table depths of 2.9 to 4.7 m. These results in terms of the stratification are consistent with those obtained by Eden and Crawford (1957), who made five boreholes that are well spread over Ottawa.



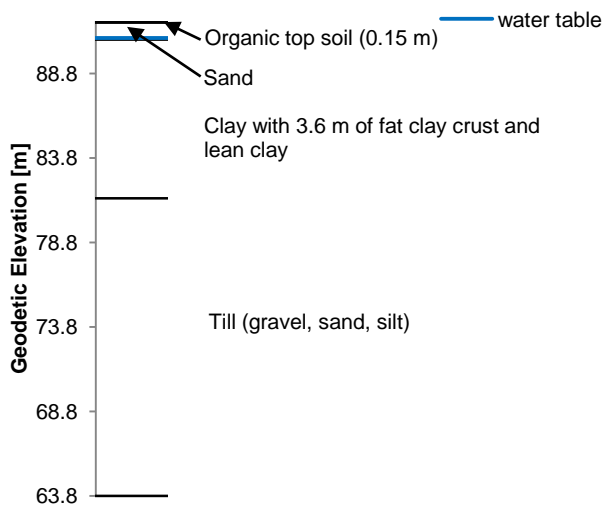
Figure 3-1 Location map of sites obtained from Google Earth



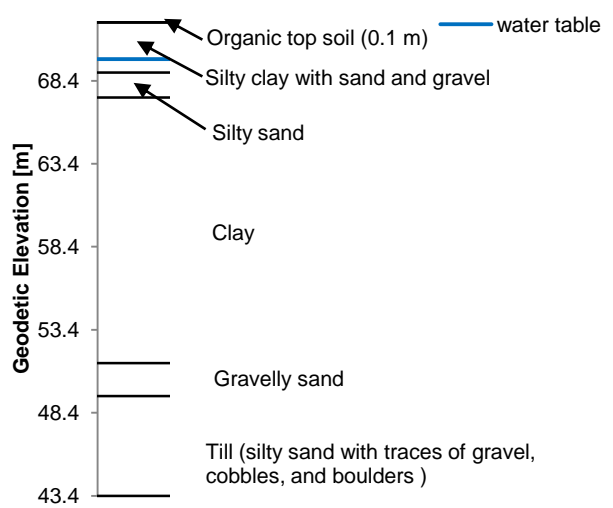
(a)



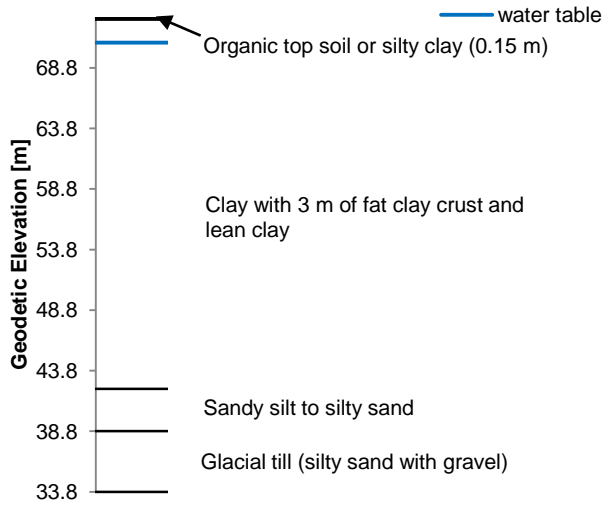
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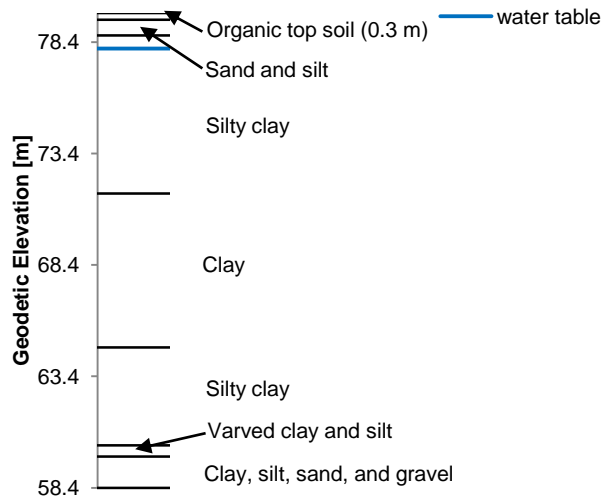
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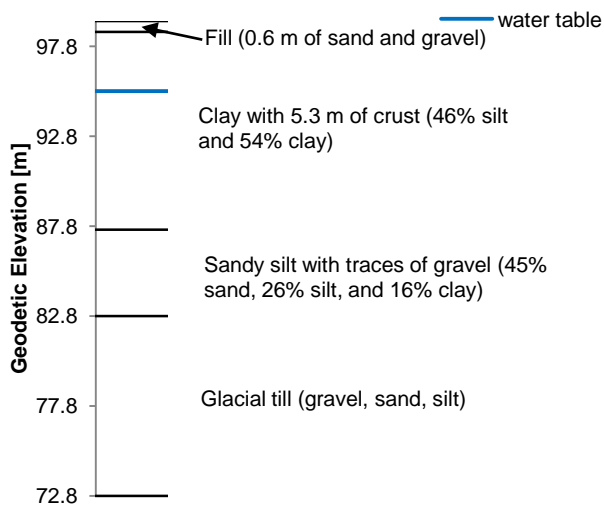
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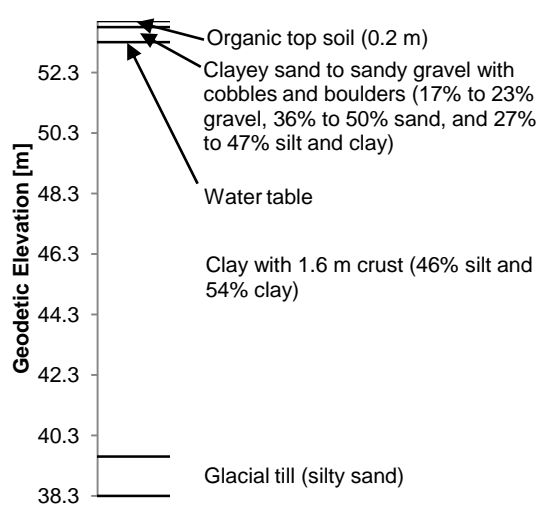
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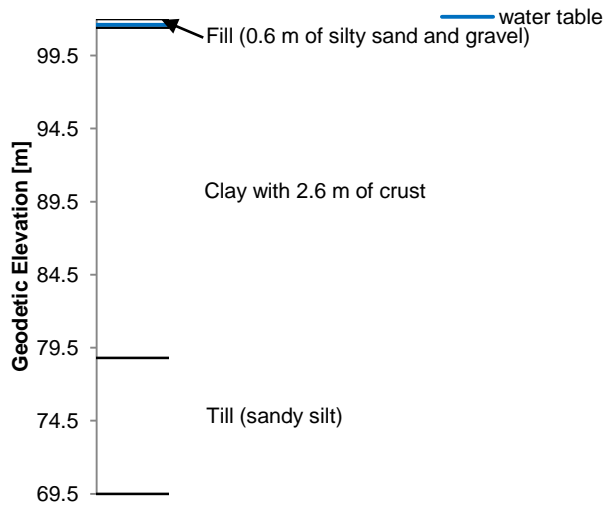
(f)



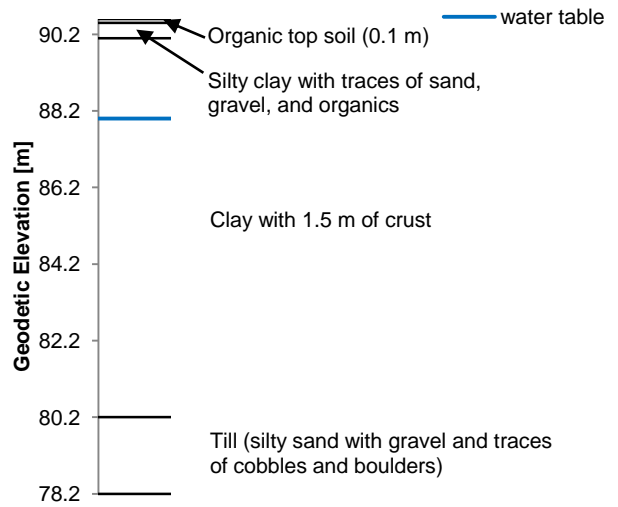
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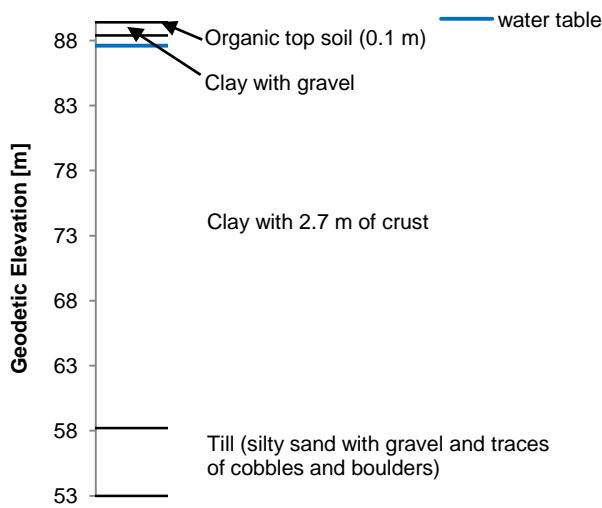
(h)



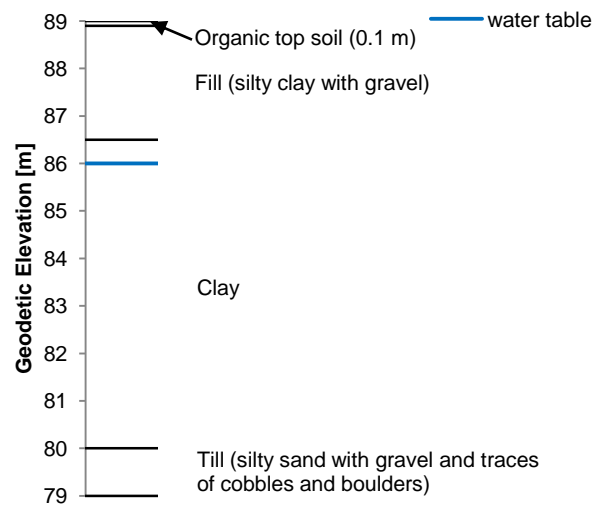
(i)



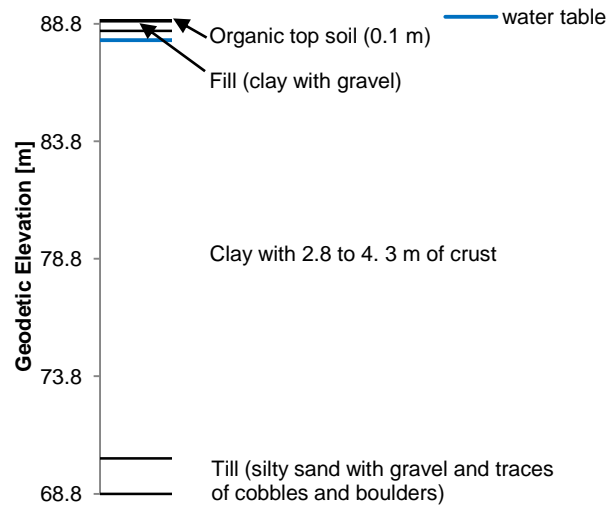
(j)



(k)



(l)



(m)

Figure 3-2 Stratification for (a) Site 1 (b) Site 2 (c) Site 3 (d) Site 4 (e) Site 5 (f) Site 6 (g) Site 7 (h) Site 8 (i) Site 9 (j) Site 10 (k) Site 11 (l) Site 12 (m) Site 13

3.2.2 Origin and Geological Setting

Quigley (1980) mentioned that soils in Canada are the products of sedimentation in proglacial and postglacial water bodies that existed during the retreat of the Wisconsin Ice Sheet between 18 000 and 6000 years before present (BP). The retreat of the ice sheet happened in stages and then stopped in some periods. The retreat was accompanied by glacial lakes that existed at the ice front and seas in the lowlands. The Champlain Sea covered Ottawa between 12500 and 10000 BP as a result of ice front retreat and re-advance, and was "expelled" as result of "crustal rebound" (Cronin, 1977; Elson, 1967; Gadd, 1975; Hillaire-Marcel, 1979). A rebound of 200 m is estimated for Ottawa. Ice front readvances in 13500, 11800, and 8200 years BP are believed to have deposited soft clays during those periods of time. The formation of glacial lakes and seas are affected by the volume of water supplied by the ice melt, rate of crustal rebound, and existence of outlets from one water body to another. The Champlain Sea is believed to have covered Ottawa after the melt down of the ice dam near Quebec City (Gadd 1975). An ice front is considered to have bordered the northern side of the sea for hundreds of years. Deposits close to the centre of the Champlain Sea are thick clay deposits while those away from the centre are clay interlayered with fluvial sand and gravel or glacial till.

Gadd (1975) divided deposits according to their origin into five types: waterlaid tills, lacustrotills, mudflow deposits, turbidity current deposits, and varved and marine clays. Waterlaid till is believed to be lacustrine clay deposited below shallow ice sheets with minimum particle gradation in a study performed by May (1977) on Alberta clays. Waterlaid till is usually underlain by hard till. In the same study, lacustrotills are indicated as soft till soils deposited by submarine mud flows in glacial lakes. Lacustrotills may contain waterlaid tills as they are deposited by turbidity currents. Depending on the rate of acceleration of the mud flow, lacustrotills may interlay with clay deposits (Morgenstern, 1967). The term "varved clays" denotes deposits in layers of summer silt and winter clay in glacial freshwater lakes. Varved clay sediments in proglacial clays are controlled by sediment concentration in the inlet flow. In the summer, a cold water flow of 1 g/L or more of sediment concentration enters the lake. This flow is higher in density than the lake which results in submarine flow that can run for miles, thus depositing a layer of silt and sand. In the winter, the inflow is low in sediments, which results in overflow in the lake, thus resulting in slow sedimentation of clay particles (Kenney et al. 1976). In postglacial lakes, sediment concentration is reduced to less than 0.1 g/L. Therefore, there were overflow and inflow rather than submarine flow and the deposits became controlled by "thermal-density stratifications". As a result of the reduced sediment concentrations and underflow, couplets are clearly found near the outlet, but summer deposits became thin away from the outlets.

Summer deposits contain less than 80% passing 2 μm while winter deposits contain more than 80% passing 2 μm (Chan and Kenney, 1973). There are also transition layers in between. In general, higher moisture contents, and liquid and plastic limits are observed for couplets in the winter as opposed to the summer. A high moisture content of 75% indicates an "open flocculated structure" (Quigley and Ogunbadejo, 1972). A schematic figure by Quigley (1980) shows that the coarser couplet in the summer is thicker than the clay layer in the winter and can be up to 1 m in thickness. Due to depositions in salt or sea water, freshwater inlets would enter the sea as overflows as a result of the high density of the sea water from high salinity concentration. Sea water has a salinity concentration of 30‰, primarily of sodium chloride (NaCl) with a density of 1020 g/L while the concentration of salinity in freshwater can be up to 4 g/L. Therefore, subglacial flows would rise to the surface and there would be little turbidity current activity (Quigley, 1980). Ottawa lands had a crustal rebound of about 200 m (Andrews, 1972). The

Champlain Sea was very deep in its early stages (Kenney, 1964). The Sea had a salinity of approximately 35 g/L. When freshwater from ice front melting or inlets enter the seawater or salt water, it is mixed by diffusion and turbulence in the first 5 m of the surface layer of the sea. In the same layer, clay particle flocculation begins. Below this layer, pelletization (biological activities) affects the clay floccules (Syvitski, 1978; Syviski and Murray, 1980; Syvitski et al. 1980). These clay floccules contain organic matter and siliceous and calcareous shell fragments. "Black mottling" common in marine clays is attributed to bacterial activity in organic matter. The organic matter is reported to be between 0.4% to 1% (Donovan and Lajoie, 1979; Laventure and Warkentin, 1965).

3.2.3 Porewater Chemistry

A porewater chemistry analysis was performed by Haynes and Quigley (1978) on sensitive marine clay in Hawkesbury about 90 km to the east of Ottawa. The study was also mentioned by Taha et al. (2010). The clays primarily contain sodium (Na^+) with magnesium (Mg^{2+}), calcium (Ca^{2+}) and potassium (K^+) ions. The salinity ranges from 2 g/L at the surface to 15 g/L at a depth of 30 m.

Taha et al. (2010) reported NaCl concentration of 3 to 4 g/l in their marine clay samples from Ottawa. Dayal (1970) reported salt content from other research work, such as that by Crawford (1961), Crawford (1968), and Crawford and Eden (1965), for sensitive marine clays to be between 0.3 to 15 g/l.

Sulphate, chloride, pH, and resistivity have been analyzed for a few samples from the sites studied as shown in Figure 3-3. The sulphate content ranges between 10 and 707 ppm which is less than 1000 ppm. The pH ranges between 7.2 and 9.2 which is considered to be in the normal range of 5.5 to 9. The amount of chloride and resistivity ranges from 4 to 1100 ppm and 4 to 160 ohm-m respectively.

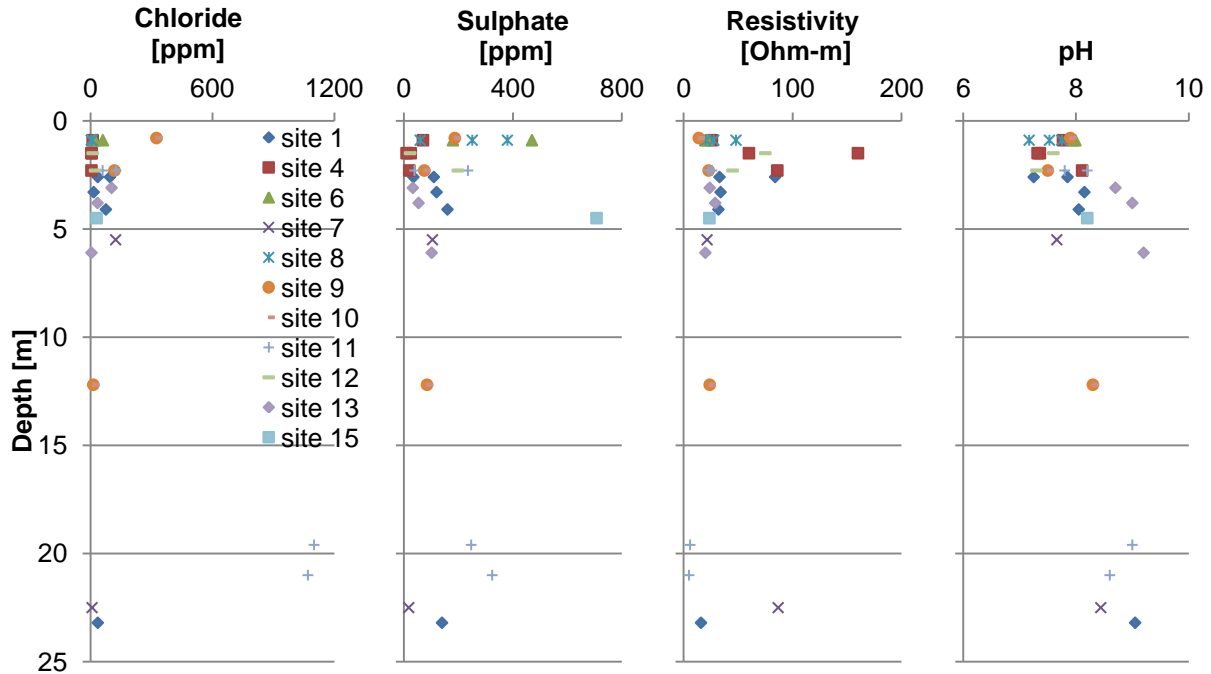


Figure 3-3 Amount of chloride and sulphate, and resistivity and pH with depth

3.2.4 Mineralogical Composition of Sensitive Marine Clay

Champlain Sea clays contain high amounts of illite, chlorite, and amphibole, with smaller amounts of smectite (Brydon and Patry, 1961; Gillott 1970). The clays may contain small amounts of swelling minerals, and biotite may be the most unstable mineral in these clays. Carbonate precipitation may have significant effects on bonding (Townsend et al. 1969).

Sensitive marine clays in Ottawa may contain black spots that represent organic matter with silica and calcium carbonate materials (Taha et al. 2010). Delage and Lefebvre (1984) noticed anisotropy between the vertical and horizontal micro structures of marine clays which would allow for high moisture content. In the vertical direction, there are silt particles covered by clay plates that form porous networks. In the horizontal direction, the clay plates are larger, but there are more silt particles, which makes the silt particles more apparent. However, the clay particles are greater in volume in both the horizontal and vertical directions. It was also found that the soil structures stay in the same format, but have smaller porosity after remolding.

Eden and Crawford (1957) mentioned that sensitive marine clays in Ottawa are composed of illite with mica, plagioclase feldspar, quartz, and chlorite according to studies performed by the

Department of Agriculture Canada and the Massachusetts Institute of Technology. Sensitive marine clays also contain stratifications of illite and chlorite minerals. They also commented that a few other studies have indicated that there may be the presence of montmorillonite and kaolinite.

3.3 Physical Characteristics, Atterberg Limits and Activity

3.3.1 Physical Characteristics

3.3.1.1 Void ratio and porosity

Void ratio and porosity values were obtained from one dimensional consolidation tests in accordance with the ASTM D 2435 standard as shown in Figure 3-4. A profile of the void ratio was supplied by the National Research Council Canada and reported by Hinchberger and Rowe (1998) for the first stage of the embankment of Site 6. The profile is graphed and converted into porosity as shown in Figure 3-4. The void ratio ranges between 0.82 and 2.80 for all of the sites, and the porosity ranges between 45.0% and 73.7%. There is a general pattern of decrease with depth for the void ratio and porosity in Sites 1, 2, 5, 6, 9, 10, and 12. However, there is a general pattern of increase with depth for Sites 8 and 11. A high void ratio can result in high deformation of marine clays in shearing (Taha 2010).

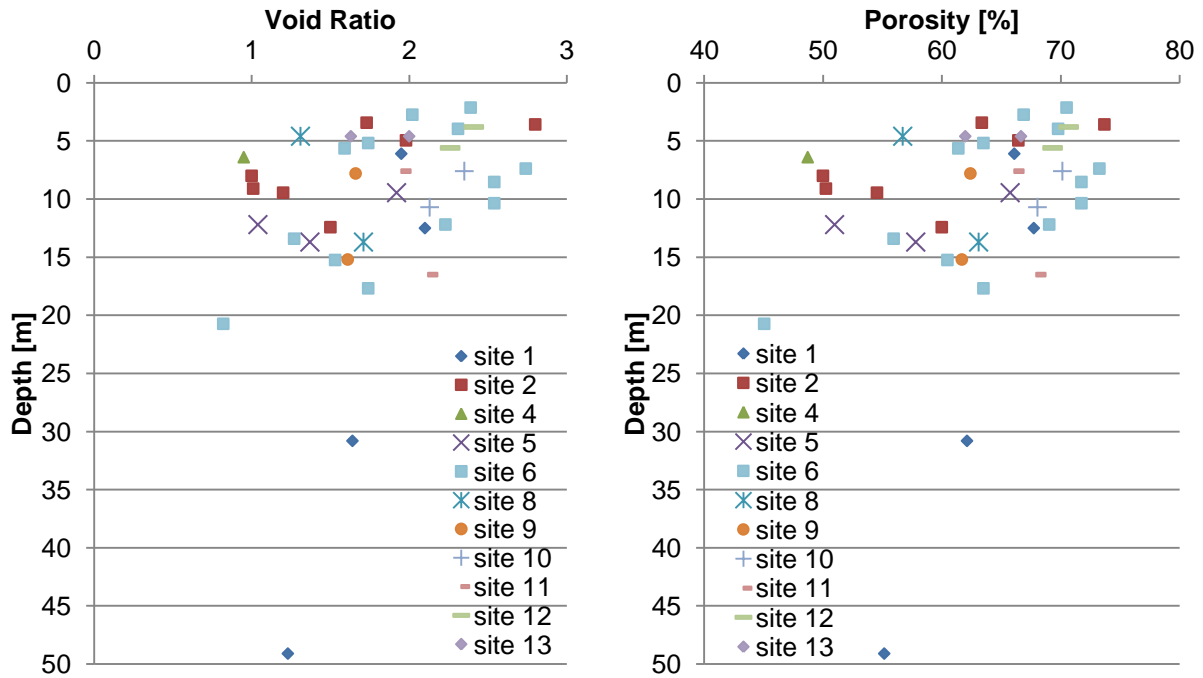


Figure 3-4 Void ratio and porosity with depth

3.3.1.3 Unit Weight

Lo et al. (1976) reported average unit weight values between 14.9 and 16.2 kN/m^3 for Site 6. Rasmussen (2012) collected ten Shelby tube samples from the Ottawa area. She reported unit weight ranges between 15 and 16.4 kN/m^3 . In the present study, the unit weight was determined in the laboratory and shown in Figure 3-5. The unit weight ranges between 14.4 and 21.0 kN/m^3 .

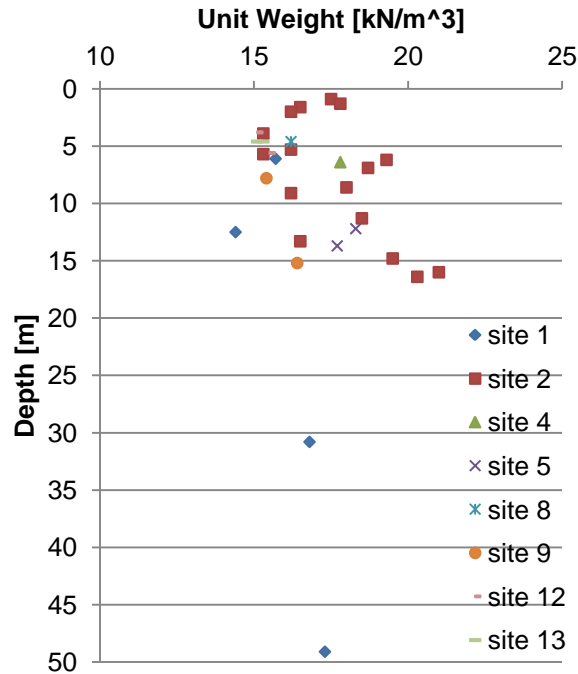


Figure 3-5 Unit weight with depth

3.3.1.4 Clay Fraction

The clay fractions were determined from hydrometer testing for the available data from all of the sites. The clay fractions range between 56% and 97% as shown in Table 3-1. These values are in agreement with those reported in the literature. For example, Taha (2010 and Taha et al. (2010) reported a fraction of 84% for a particle size of 2 μm in the Ottawa marine clays that they studied, which means a high clay content. Mitchell (1970) reported clay contents of 71%, 60%, and 60% for Site 6 from block samples obtained at depths of 13.7, 16.1, and 19.0 m respectively. Quigley and Thompson (1966) studied a block sample obtained from sensitive marine clay in Ottawa during a sewer installation. They reported a clay portion of 86% and silt portion of 14%. Dayal (1970) reported clay portions from other research carried out, such as that by Crawford (1961), Crawford (1968) and Crawford and Eden (1965) for sensitive marine clays to be between 36% to 88%. He reported a clay portion of 86% for his sample. Rasmussen (2012) collected ten shelly tube samples from the Ottawa area. She reported a clay fraction range of 56% to 84%. A point to mention is that summer deposits contain less than 80% clay while winter deposits contain more than 80% clay (Chan and Kenney, 1973).

Table 3-1 Clay fractions where each value represents a laboratory test

Site	Clay Fraction [%]
1	67, 67, 69, 73, 73, 74, 75, 76, 76, 78
2	66, 77, 81, 92, 93, 94
3	56, 97
4	83, 93
6	57, 59, 68, 70, 78, 81, 87, 87, 88, 91, 92
7	88
9	88, 95, 97
10	95
11	86, 95
12	91, 93, 95
13	91
14	57, 64

3.3.1.5 Specific Gravity

The specific gravity was estimated from laboratory tests which were carried out in accordance with ASTM D854 and shown in Figure 3-6. The specific gravity ranges between 2.70 and 2.80 in general for all of the sites as expected for clay. These values are in agreement with those published by several authors, such as Leroueil, (1999), Taha 2010, and Taha and Fall (2010) (2010), Quigley and Thompson (1966) and Rasmussen (2012). For example, Rasmussen (2012) collected ten shelby tube samples from the Ottawa area and reported a specific gravity range of 2.82 to 2.90. She attributed the relatively high values of specific gravity to the amount of rock flour in these sensitive marine clays as a result of bedrock erosion. There is a general trend of increase in the specific gravity with depth.

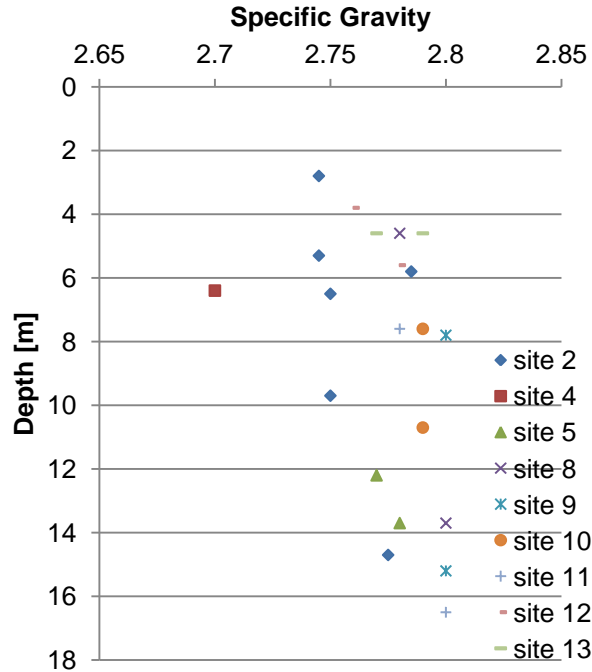


Figure 3-6 Specific gravity with depth

3.3.2 Atterberg Limits and Water Content

The Atterberg limits were determined in the laboratory in accordance with ASTM D4318. The obtained results are shown in Figure 3-7. In general, all of the sites have increasing moisture contents from below the plastic limits to approximately equal to or above the liquid limits from the surface to a depth of approximately 6 m, and then they decrease. This increasing pattern is usually within the fill layer above the clay layer or within the crust part of the clay layer as the water table is above the clay layer or within the crust layer of the clay. The clay deposit layer in all of the sites has moisture content close to or exceeding the liquid limit of the clay. The decrease in the moisture content generally occurs in the till layer beneath the clay layer. Fill layers have a moisture content that ranges from 1% to 38%. The till layers have a moisture content that ranges from 7% to 26%. The clay layers have a moisture content that ranges from 17% to 85%. The upper crust layer carries the lower end of the moisture content generally from 17% to 75% while the deeper clay layer carries the higher end of the moisture content that range from 30% to 85% or can be as high as 98% as in Site 3 at a depth of 4.1 m. The plastic and liquid limits range from 14% to 28% and 19% to 80.8% respectively. All of the available Atterberg

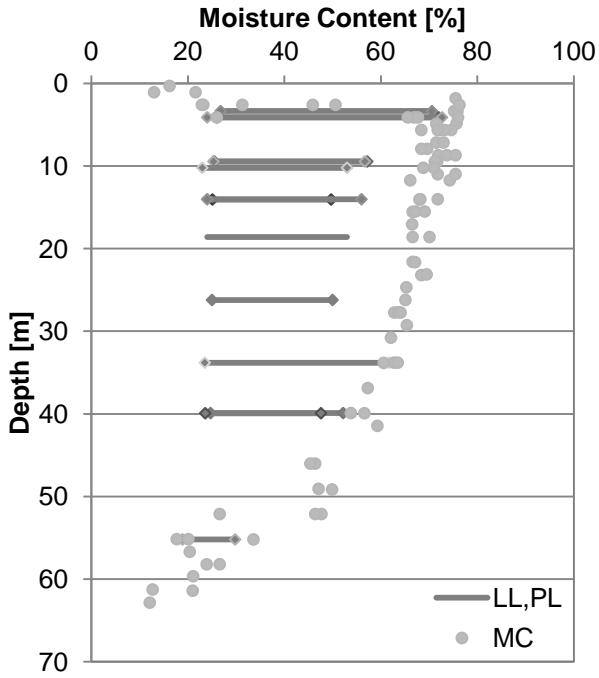
limits are shown in Cassagrande's plasticity chart in Figure 3-8. According to Cassagrande's plasticity chart, sensitive marine clays in Ottawa can be classified as clay of low to high plasticity. These findings are in accordance with the conclusions of previous studies.

Taha (2010), and Taha and Fall (2010) collected samples from the Ottawa area at a depth of 8 to 12 m. They reported a liquid limit of 66% and a plastic limit of 25%. The liquidity index was also reported to be 1.4 which gives the soil a quick behavior when remolded (Law and Bozozuk 1988). They also mentioned that the moisture content which is 82% for all of their samples exceed the liquid limits. They attributed this to the open fabric structure of marine clays which allows them to hold high amounts of water and makes the soil collapsible when remolded. According to the Unified Soil Classification System, the samples taken by Taha and Fall (2010) (2010) are inorganic clays of high plasticity (CH). According to Leroueil (1999), the liquid limit of sensitive marine clays in eastern Canada is less than 83%, plastic limit is between 17% and 35%, and liquidity index is higher than unity. The samples were obtained from Ontario, Quebec, and British Columbia, and all were found to fall above the A line in Cassagrande's plasticity chart (Bell 2000). Salinity can also affect the plasticity of marine clays. There is a negligible effect as the salinity is increased from 4 to 10 g/l, but there is a decrease of approximately 10% in the liquid and the plastic limits as the salinity is increased from 10 g/l to 20 g/l (Taha et al. 2010; Torrance 1975). Dejong et al. (2011) reported values of 71, 27 and 24, and 62, 28 and 34 for the liquid limit, plastic limit, and plasticity index respectively for depths of 4.4 and 8.3 m respectively for Site 6.

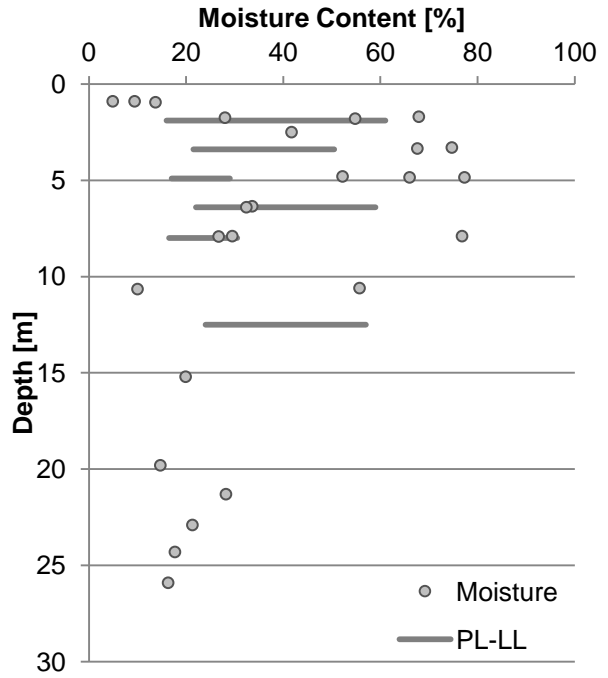
Eden and Crawford (1957) mentioned that sensitive marine clays in Ottawa may have a liquidity index of more than unity which may contribute to the sensitivity. Mitchell (1970) obtained block samples from Site 6. He reported moisture contents of 55%, 45%, and 48%, liquid limits of 47, 31, and 32, and plastic limits of 22, 22, and 22 for depths of 13.7, 16.1, and 19.0 m respectively. Quigley and Thompson (1966) studied a block sample obtained from sensitive marine clay in Ottawa during a sewer installation. They reported a moisture content of 78%, liquid limit of 56, plastic limit of 29, plasticity index of 27, and liquidity index of 1.8. Rasmussen (2012) reported a moisture content range of 53% to 83.4% for samples obtained from the Ottawa area. She also reported liquid limit, plastic limit, plasticity index, and liquidity index ranges of 34 to 73, 19 to 26, 15 to 49, and 1.1 to 2.3 respectively.

According to a profile on plasticity limits and water contents reported by Lo et al. (1976) for Site 6, all of the moisture contents are past the liquid limits. They reported the average values, and cited a moisture content that ranged from 60% to 82%, liquid limit from 48 to 58, and plasticity index from 23 to 30. They also reported an average liquidity index of 1.5 to 1.9.

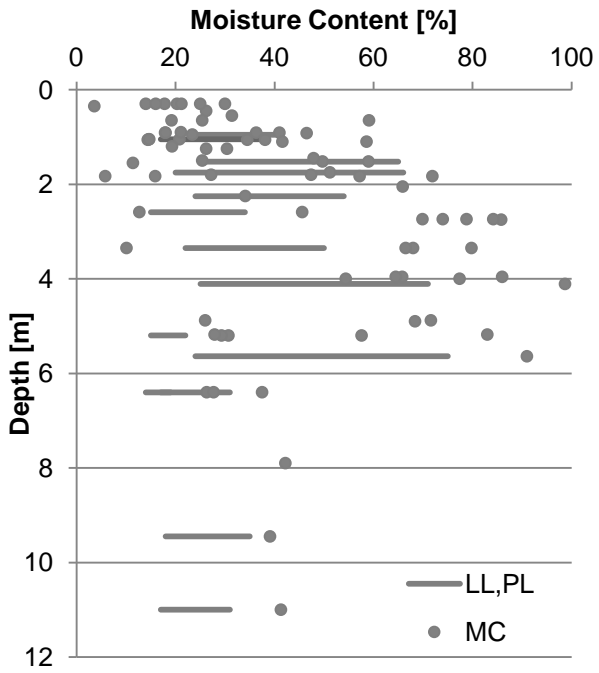
A profile of the moisture contents is supplied by the National Research Council of Canada and reported by Hinchberger and Rowe (1998) for the first stage of the embankment of Site 6. The profile is graphed in Figure 3-7o. The moisture contents range between 30.6% and 101.6% at a depth of 2.13 to 20.73 m. There is a general trend of decrease in the moisture content from the surface downwards. There are lower values of the moisture content beyond a depth of 20 m as there is a layer of coarser silty clay or sandy clay as reported by Yafrate and Dejong (2005).



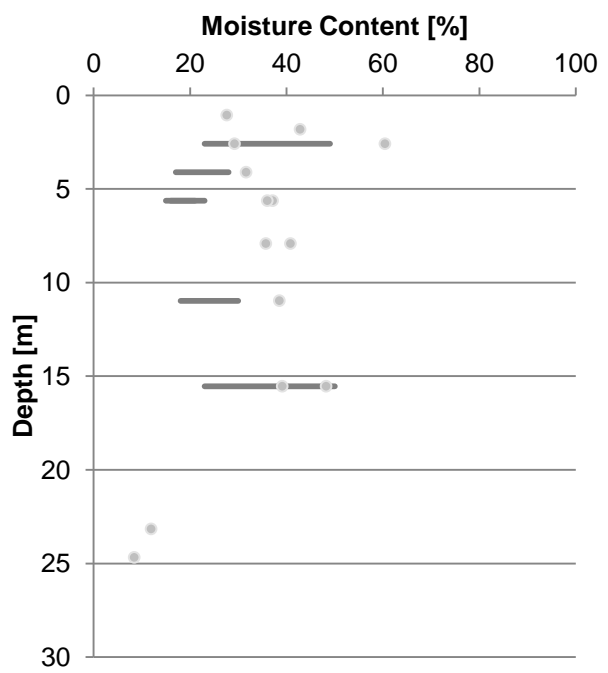
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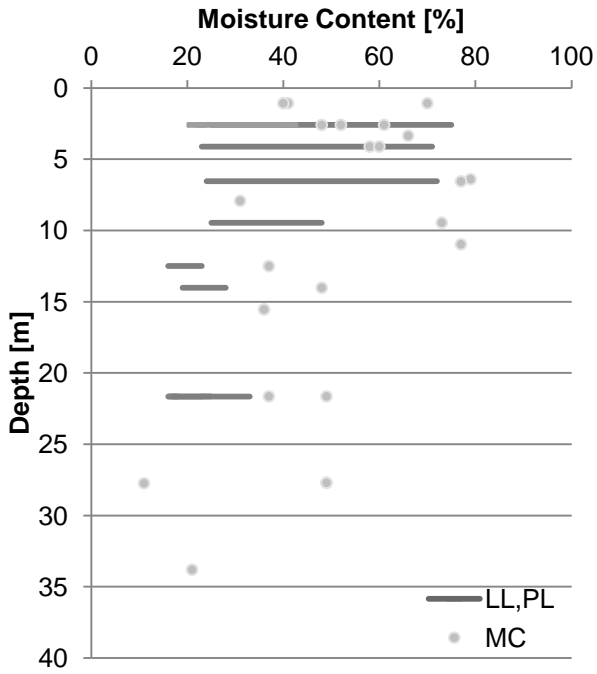
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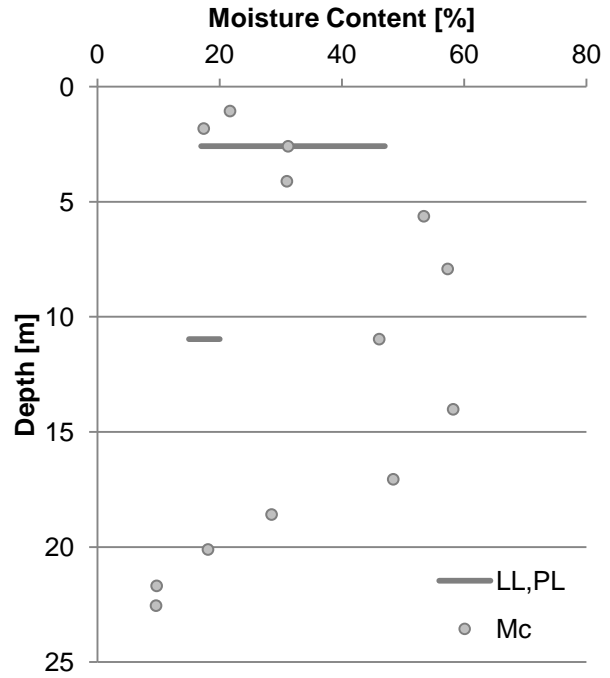
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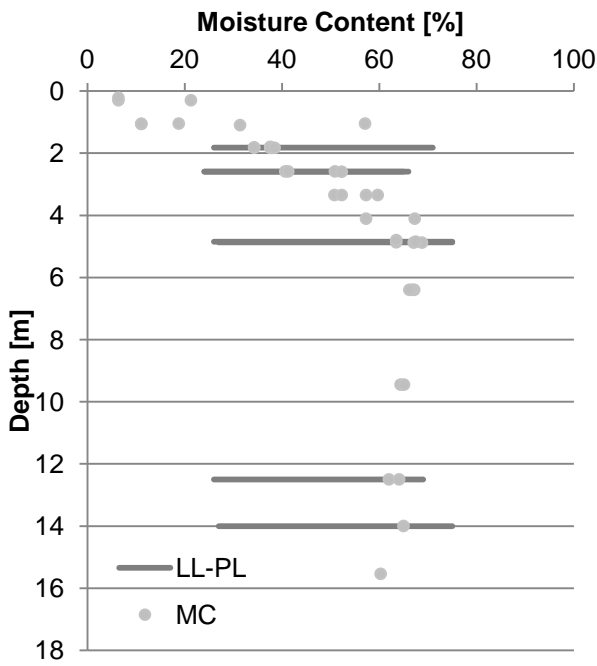
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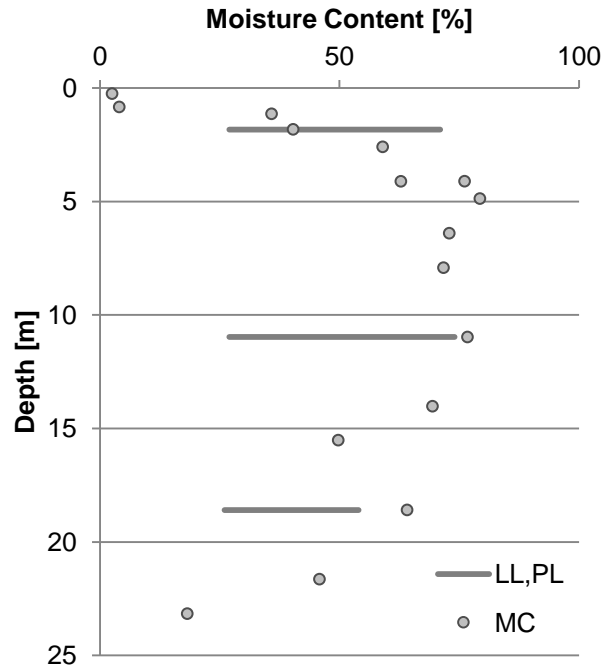
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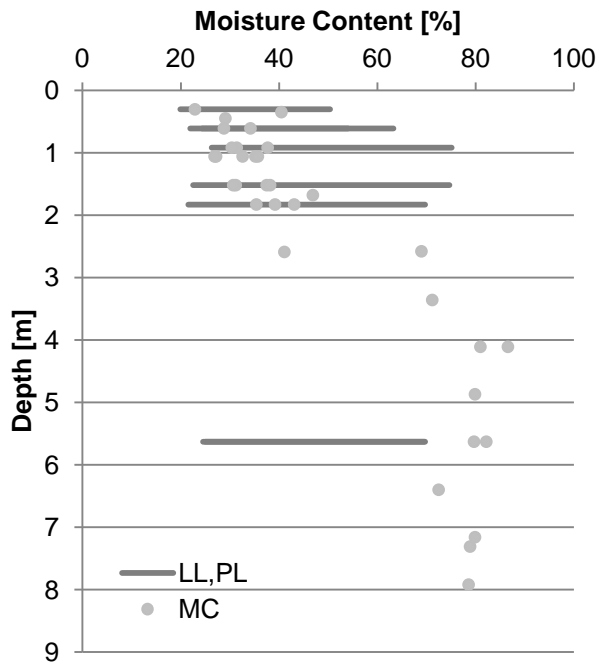
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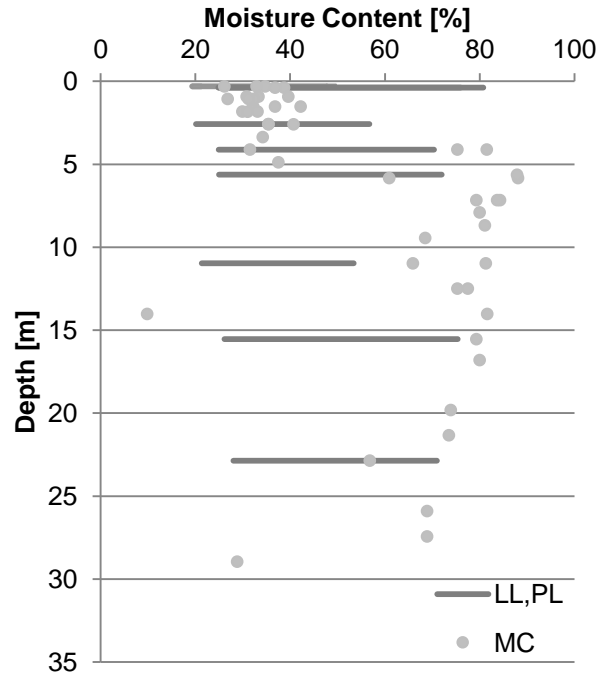
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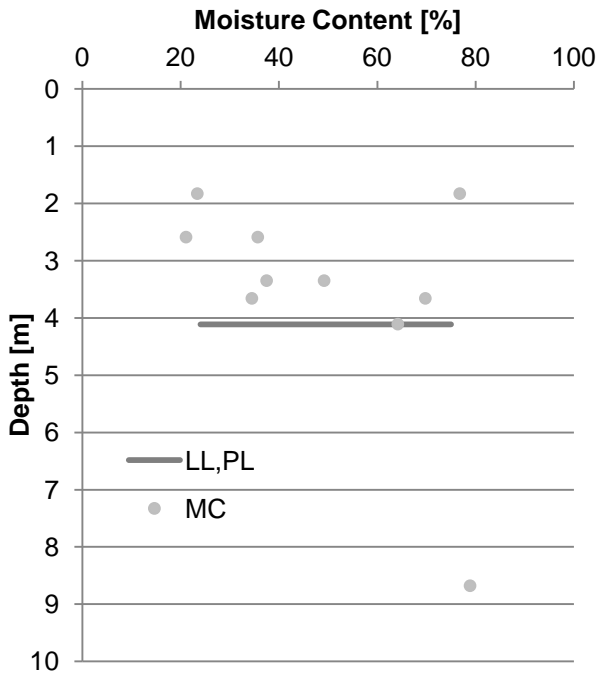
(h)



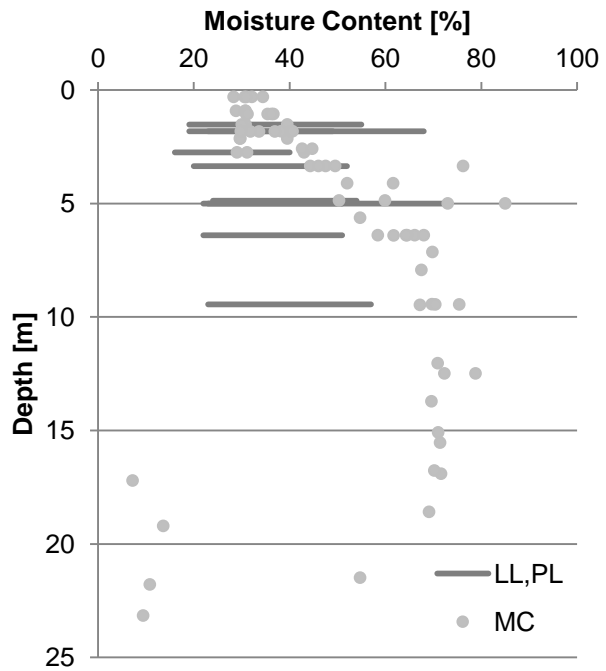
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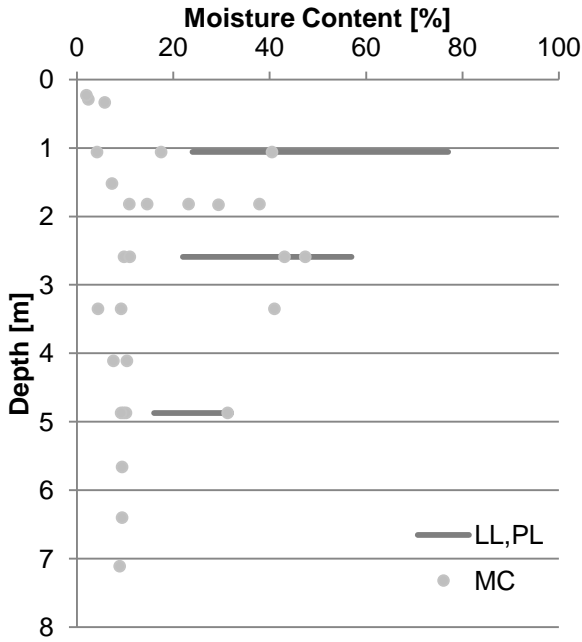
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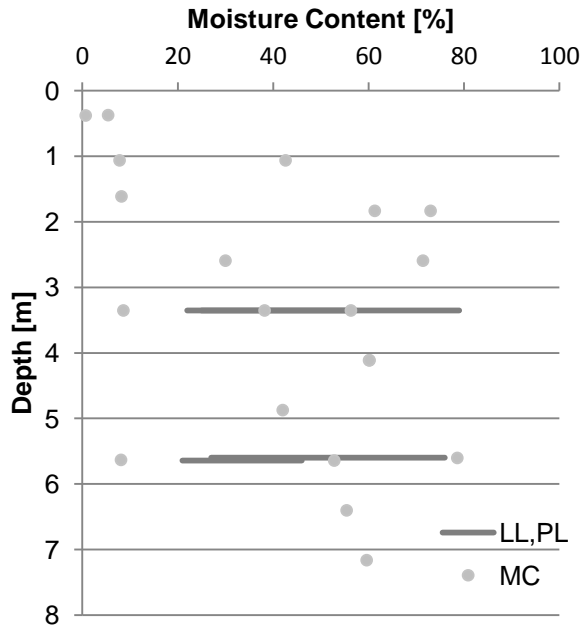
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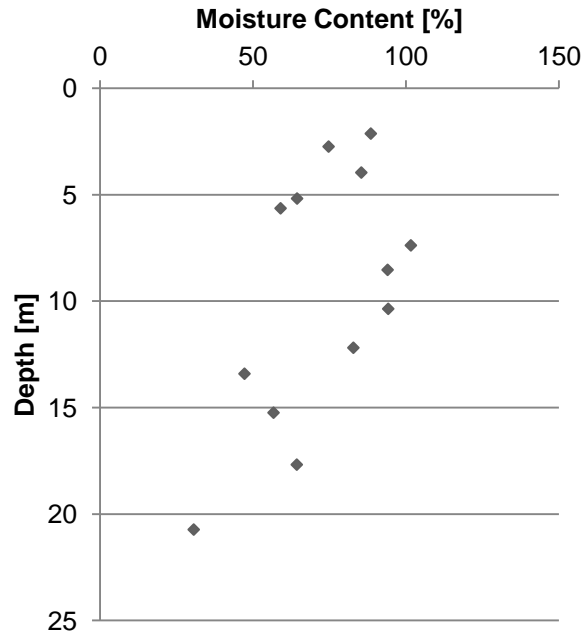
(l)



(m)



(n)



(o)

Figure 3-7 Moisture contents, liquid limits, plastic limits, and plasticity index with depth for (a) Site 1 (b) Site 2 (c) Site 3 (d) Site 4 (e) Site 5 (f) Site 7 (g) Site 8 (h) Site 9 (i) Site 10 (j) Site 11 (k) Site 12 (l) Site 13 (m) Site 14 (n) Site 15 (o) Site 6

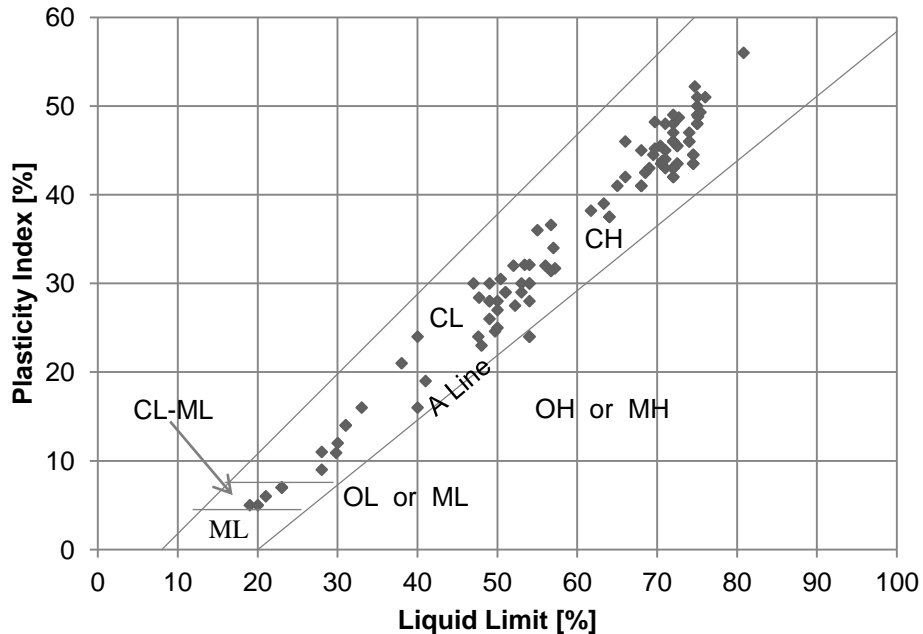


Figure 3-8 Cassagrande's plasticity chart for plasticity properties of all of the sites except for Site

6

3.3.3 Activity

The activity chart for Ottawa sensitive marine clay is shown in Figure 3-9. All of the clay samples fall within a classification of inactive to normal activity except for one sample from Site 1 (Figure 3-9). It can be seen from Figure 3-9 that samples from one site may fall into two activity zones such as Site 9 or even in all three zones such as Site 1. The reason is that the clay layers in one site may behave differently due to the structure or the deposition of that specific layer. The values reported by Leroueil et al. (1983) and Leroueil (1999) are inactive or between an activity of 0.25 and 0.75. Taha et al. (2010) reported an activity of 0.47 for their samples.

Lo et al. (1976) reported a range of activity of 0.42 to 0.28 for Site 6. Therefore, they categorized the site clay as inactive which is the same as the finding in this research. Eden and Crawford (1957) and Skempton (1953) showed that sensitive marine clays in Ottawa have variable activity with location and depth, but all of the samples fall into the inactive zone. Dayal (1970) reported the activity from other researchers, such as Crawford (1961), Crawford (1968), and Crawford and Eden (1965) for sensitive marine clays, which is 0.3 to 0.6.

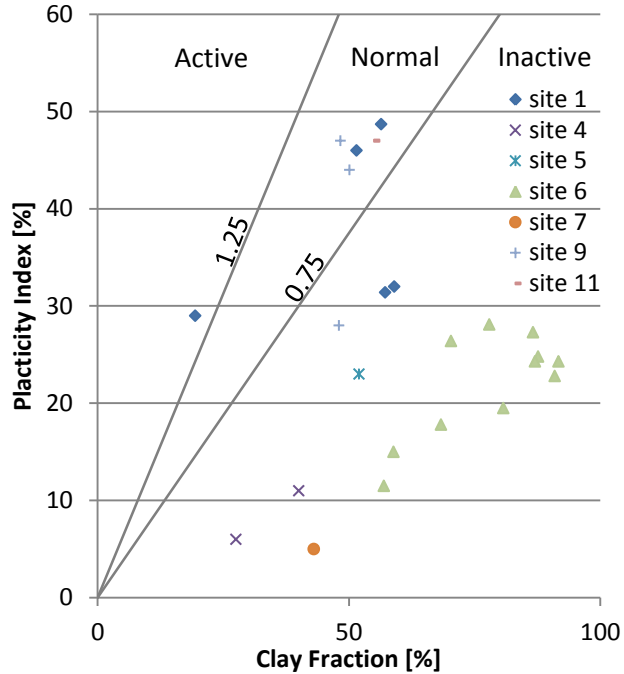


Figure 3-9 Activity chart

3.4 Geotechnical In-Situ Tests on Sensitive Marine Clay in Ottawa

3.4.1 Cone Penetration Tests

The average values of the soil specific cone factors (N_{kt} , N_k , $N_{\Delta u}$) of cone penetration tests (CPTs) obtained from many sites are presented in Table 3-2. The N_{kt} values typically range from 8.5 to 17.5 with outliers as low as 4.5 and 5.5 from Site 7. The N_k ranges from 7 to 15 with an outlier of 2.5 and 3.5 from the same site. The $N_{\Delta u}$ ranges from 6 to 11.5. The $N_{\Delta u}$ seems to have the narrowest range and least affected by outliers. The relative locations of the CPT hole to the reference vane hole are crucial to the accuracy and range of the N_{kt} and N_k values. This effect of the relative location on the soil specific cone factors of CPTs is observed in sites with multiple CPTs carried out. It can be found in the table that different distances to the reference vanes can cause variations in the soil specific cone factors for the same site. The size of the results and information presented are not confirmative for conclusion purposes, but further research should be carried out to establish limits on the distance to the reference vanes.

The typical CPT profiles obtained in this study are shown in Figure 3-10. Dejong et al. (2011) reported values for CPTs at depths of 4.4 and 8.3 m in Site 6. For these two depths, they reported

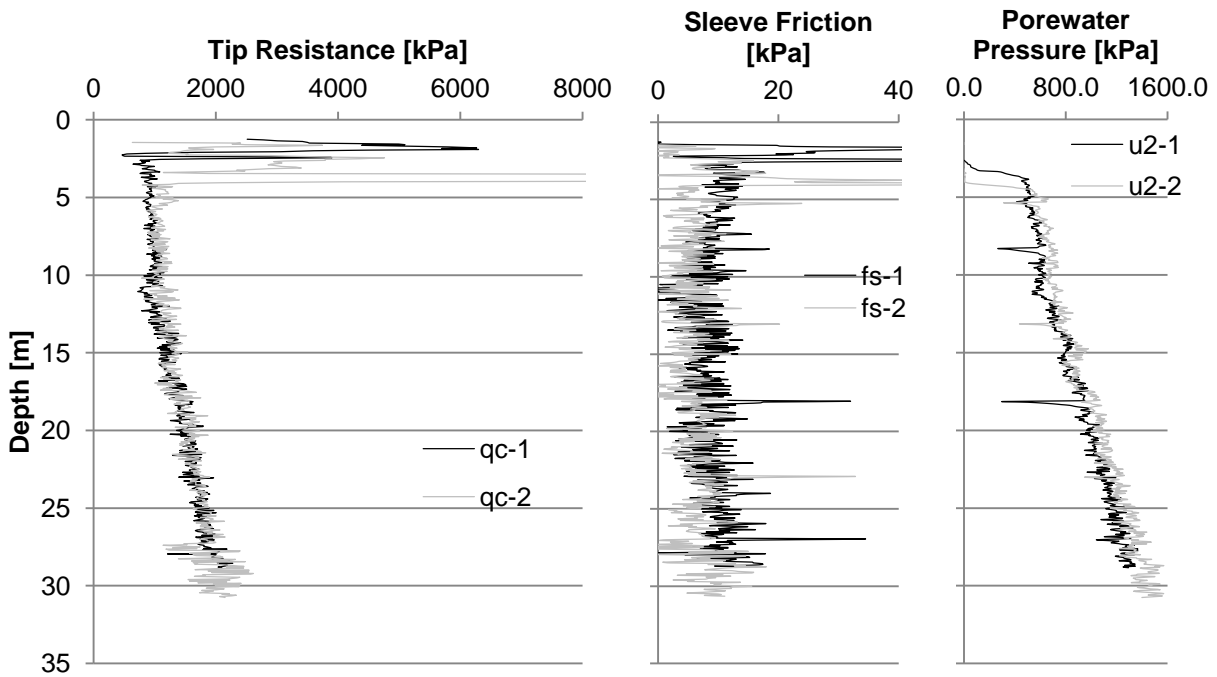
corrected tip resistance (q_t) of 306 and 410 kPa, porewater pressures (u_o) of 35 kPa and 74 kPa, and soil specific cone factors (N_{kt}) of 11.8 and 8.1, respectively. Yafrate et al. (2009) reported the same values of porewater pressure for the same depths. Yafrate and Dejong (2006) reported a CPT profile from the same site. Their values were taken from a depth of approximately 2.5 to 14 m, and they reported values of about 200 to 700 kPa, 1 to 0 kPa, and 200 to 500 kPa for the tip resistance, sleeve friction, and porewater pressure respectively.

Moreover, Konrad and Law (1987) performed CPTs in Site 6 and an Ottawa sewage treatment plant. They mentioned that the profile of Site 6 is relatively smooth and there are no signs of clear stratifications. The tip resistance measured is approximately between 100 and 1000 kPa. The porewater pressure on average is 200 kPa less than the tip resistance throughout the profile as shown in Figure 3-11a. With reference to Figure 3-11b, the Ottawa sewage treatment plant is divided into three layers, which are 3 (crust), 11, and 2.5 m in thickness respectively.

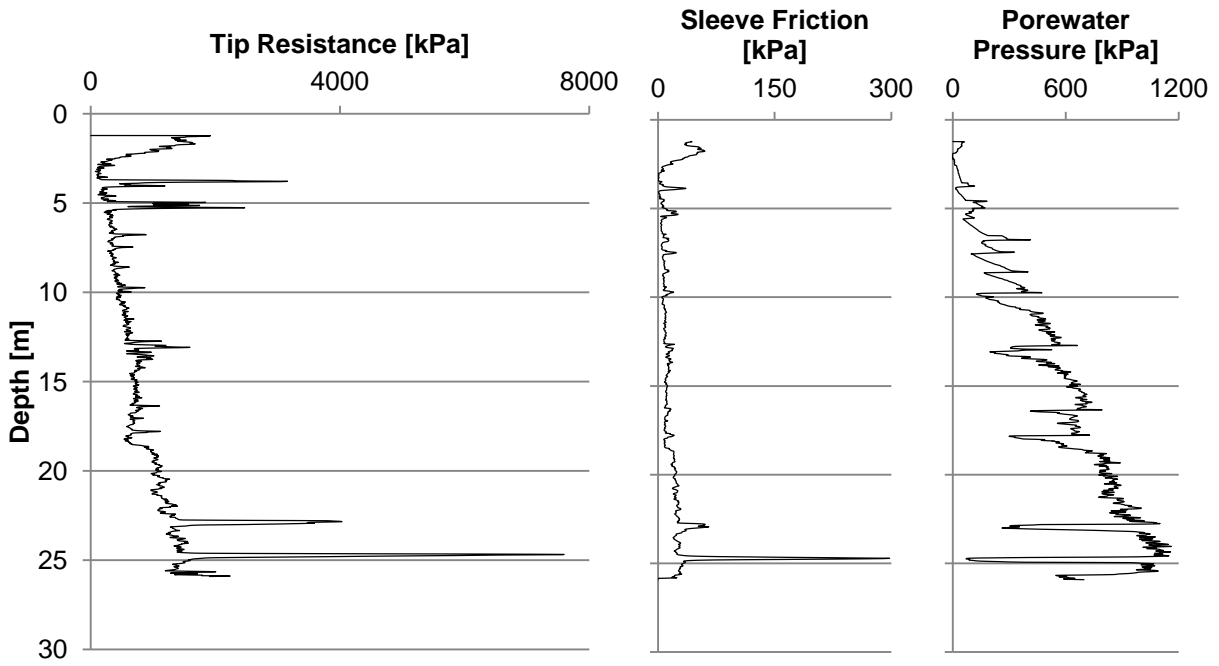
Table 3-2 N_{kt} , N_k , $N_{\Delta u}$, and distance to reference field vanes

Site No.	CPT No.	N_{kt}	N_k	$N_{\Delta u}$	Distance to vanes [m]
Site 1	1	12.3	10.5	7.3	Not Available
	2	13.2	11.4	8.4	Not Available
Site 2	1	13	10.5	7	25.0
Site 3	1	14	12	9	525.0
	2	14	11	10	230.0
	3	11	8	11	200.0
	4	12	11	8	190.0
Site 5	1	8.8	7	9	26.0
	2	10.7	8.8	12	17.0
Site 7	1	5.5	3.5	10.3	5.9
	2	4.5	2.5	8	5.6
	3	8.5	7	9	5.0

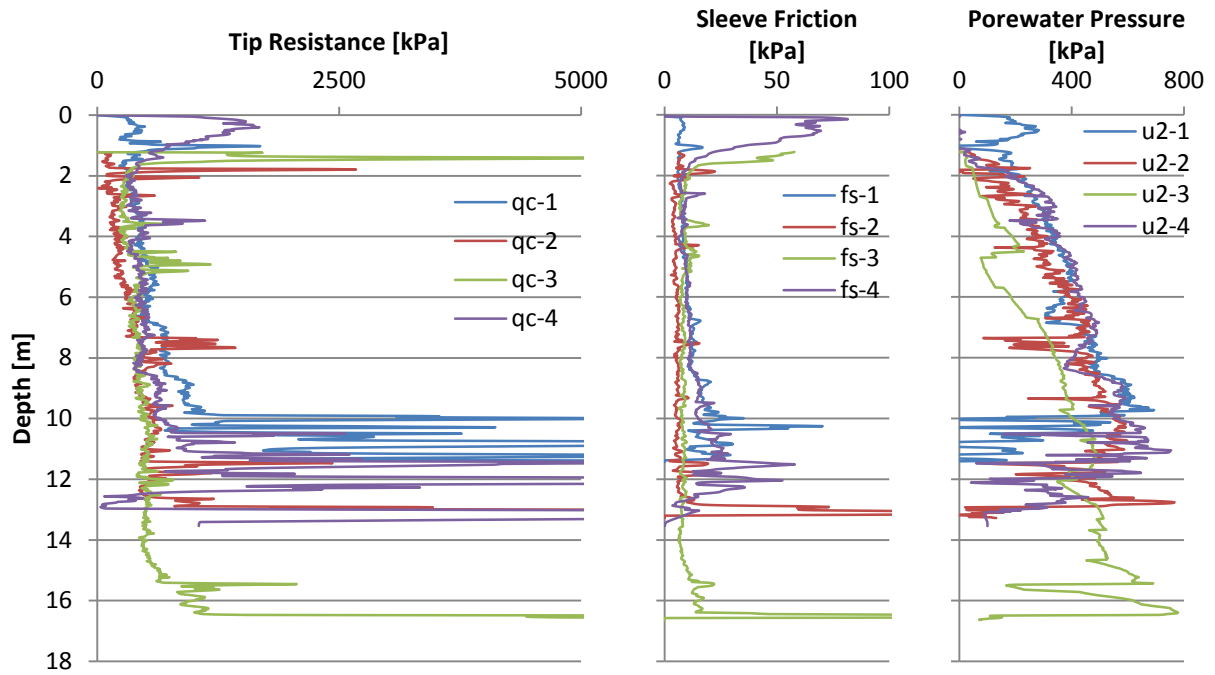
Site 8	1	8	6.5	9.5	19.5
	2	10	9	9	16.5
	3	12	12	8.5	15
Site 9	1	9.5	7	10.1	6
	2	14.5	12	11.5	46.8
	3	12	10.5	6	52.2
	4	10	8.5	8.5	57
	5	17.5	15	9.5	46.8
Site 10	1	9	7	8.5	52
	2	8.5	8.5	7	118
	3	12.5	11	8	84
Site 11	2	10.5	8.5	9.5	54



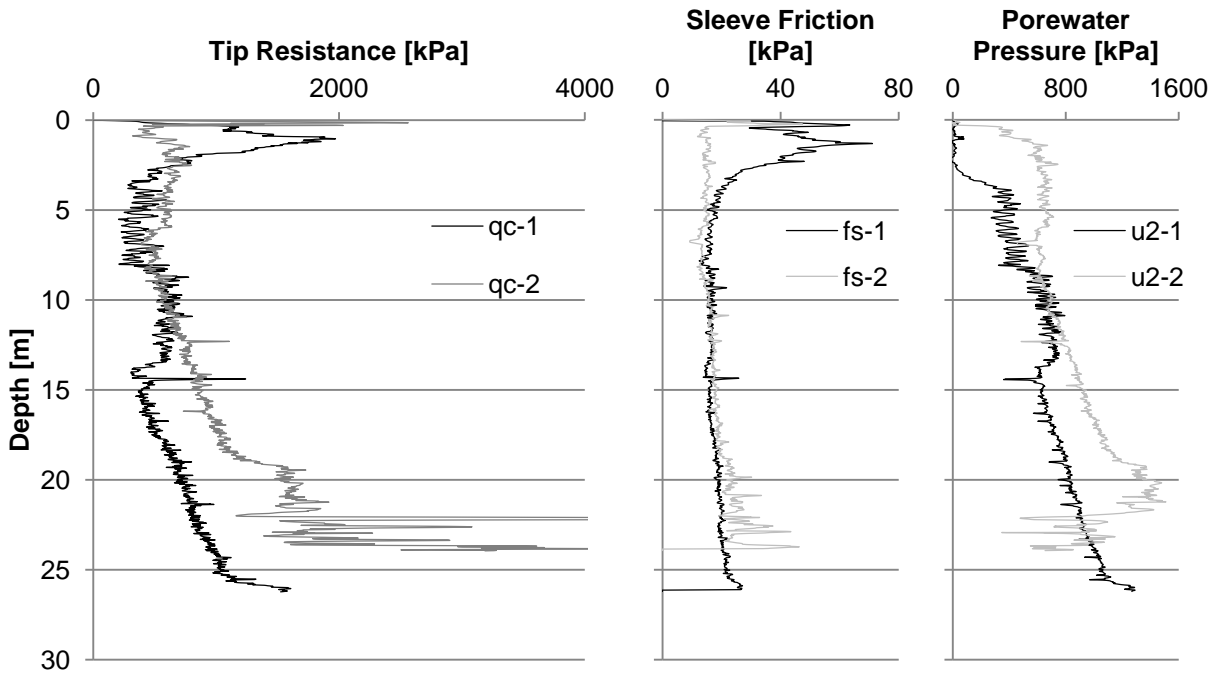
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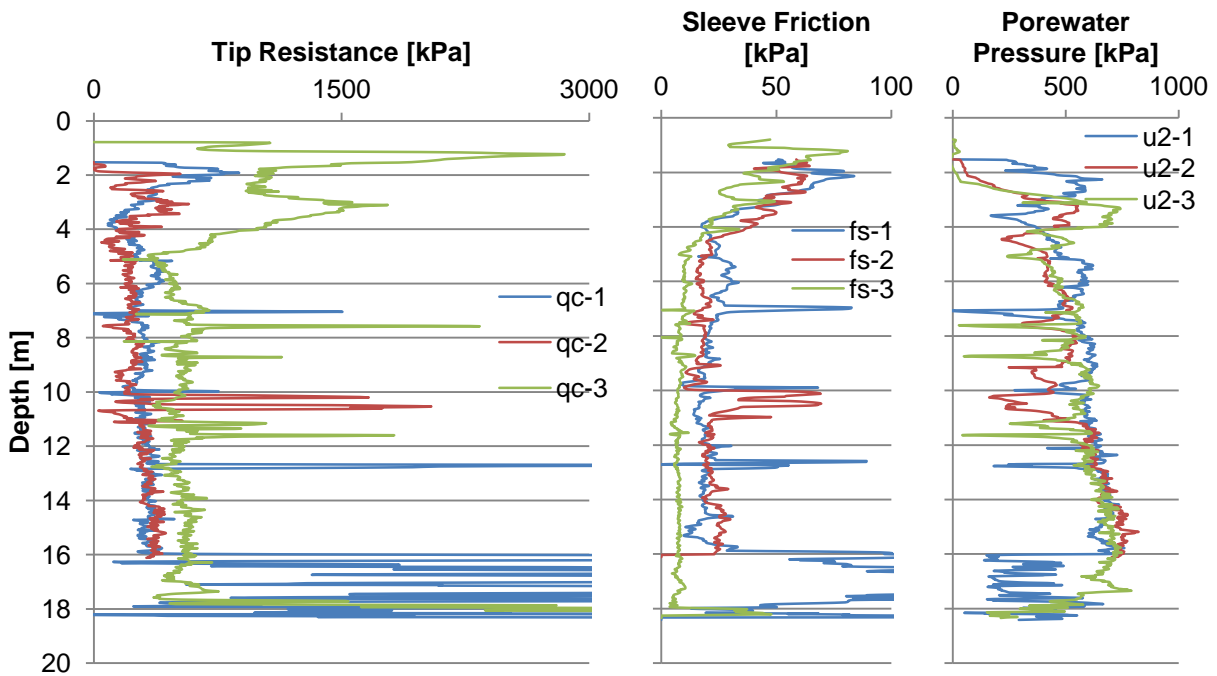
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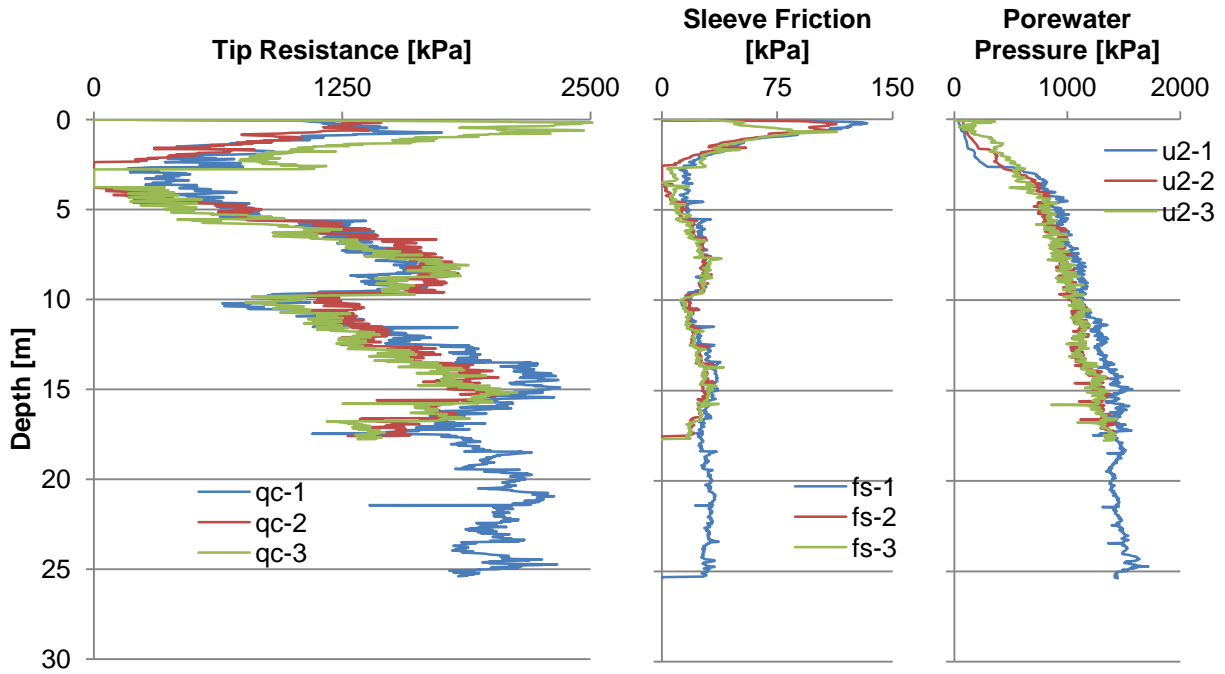
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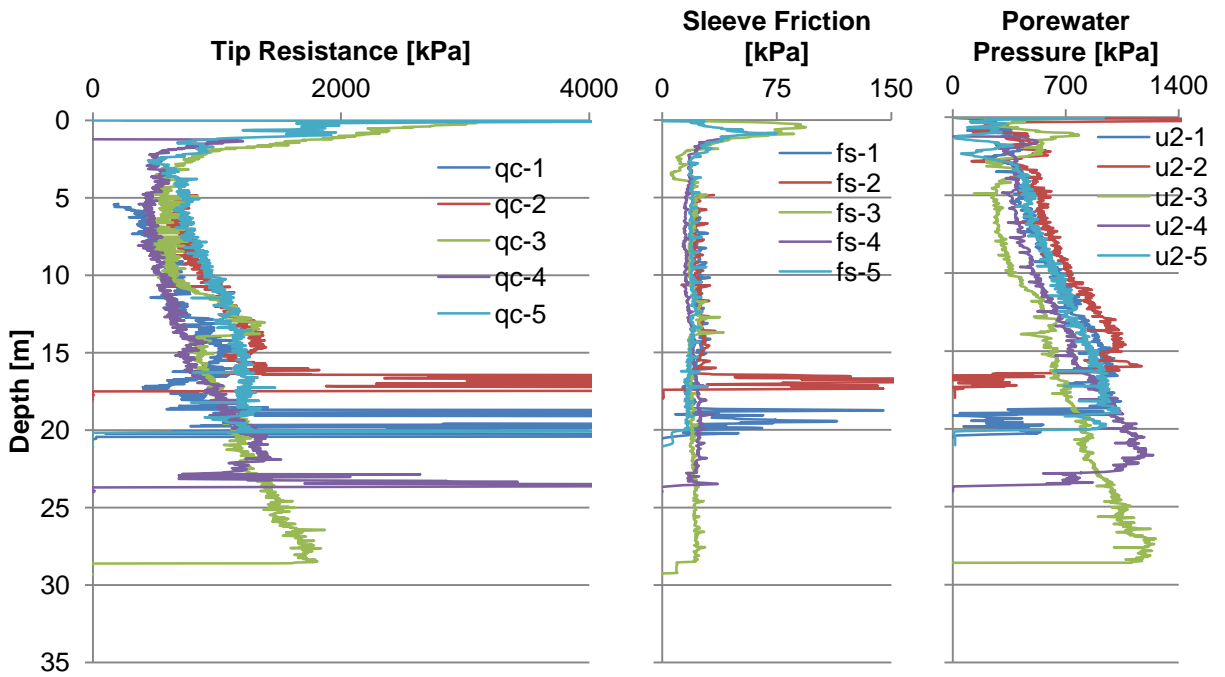
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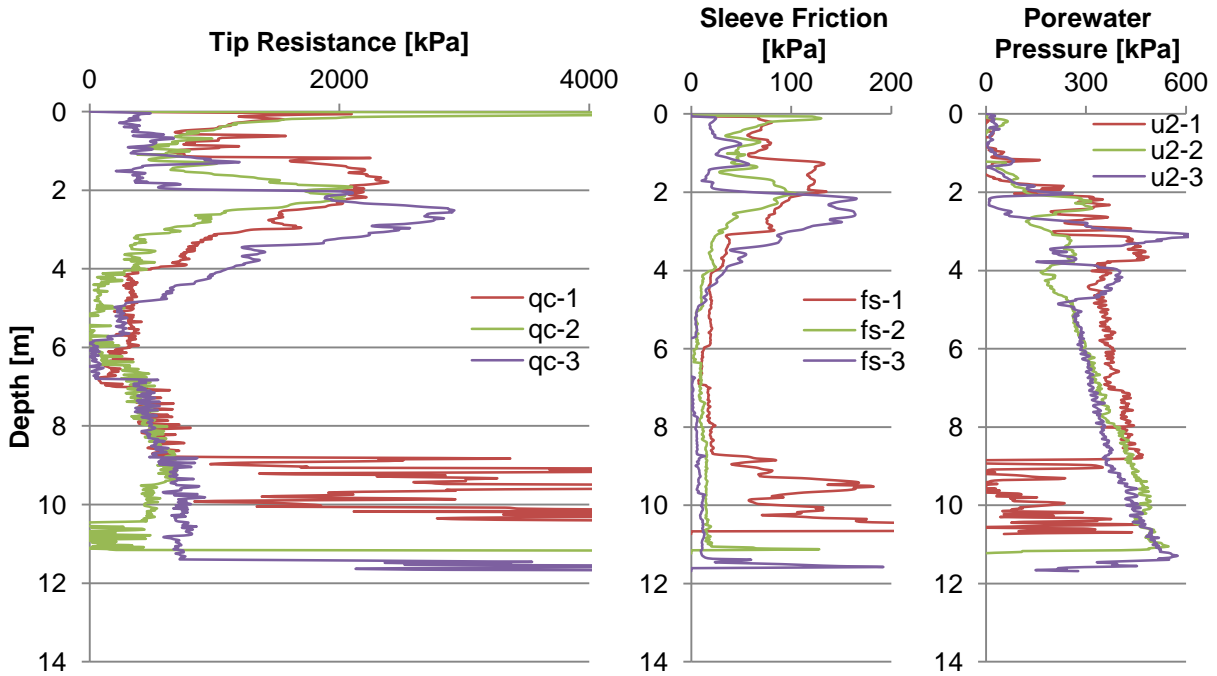
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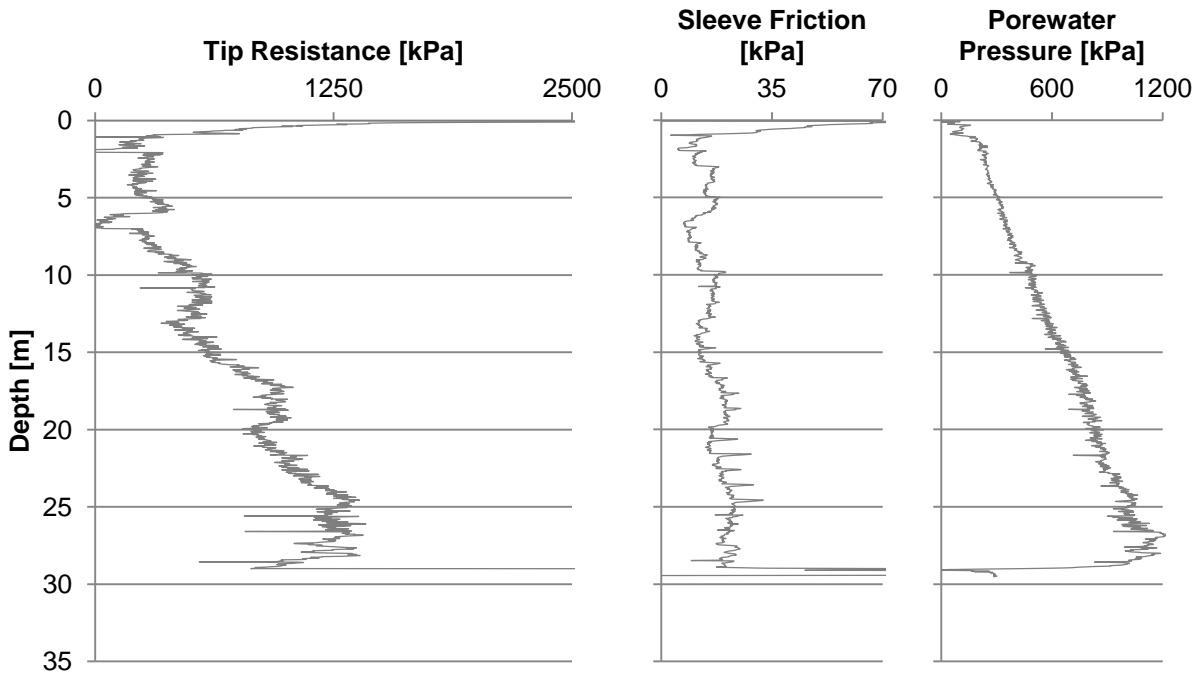
(f)



(g)



(h)



(i)

Figure 3-10 Tip resistance, sleeve friction, and porewater pressure with depth for (a) Site 1 (b) Site 2 (c) Site 3 (d) Site 5 (e) Site 7 (f) Site 8 (g) Site 9 (h) Site 10 (i) Site 11

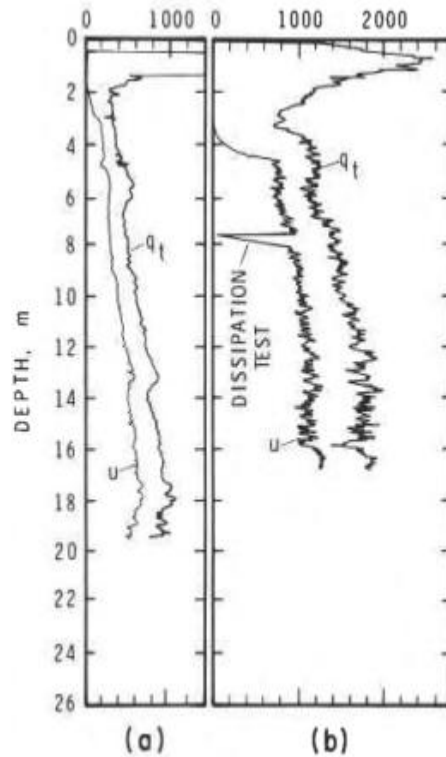
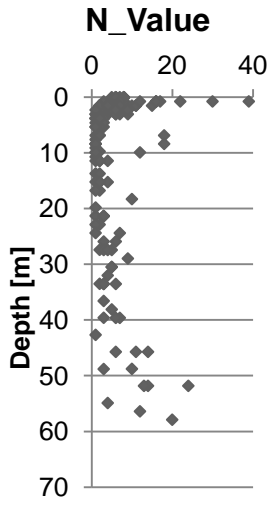


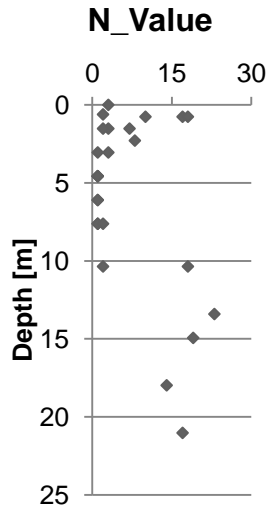
Figure 3-11 Tip resistance and porewater pressure with depth for (a) Site 6 (b) sewage water treatment plant in Ottawa. Taken from Konrad and Law (1987)

3.4.2 Standard Penetration Tests

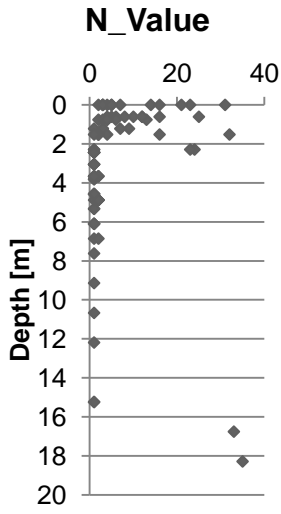
The obtained N_{60} values from standard penetration tests (SPTs) are shown in Figure 3-12. In general, the N_{60} values from SPTs are higher in the top soil and fill layers above the sensitive marine clay layers and in the till layer below than they are in the clay layer. The SPT N values for the clay range between 1 and 5, and for the upper crust layer, between 1 and 16. In Sites 4 and 5, there are layers that intercross and those that are not clearly defined. In the other sites, there may be intercrossing between layers, especially the fill layer and the clay layer beneath it. In other words, the layer thickness is only an estimation.



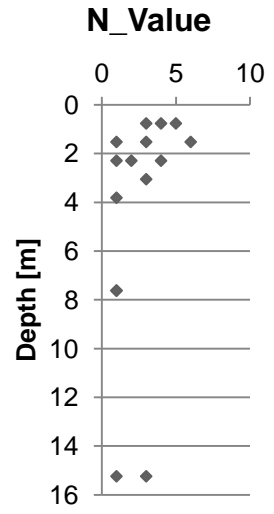
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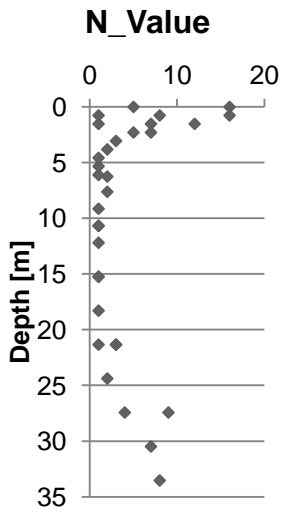
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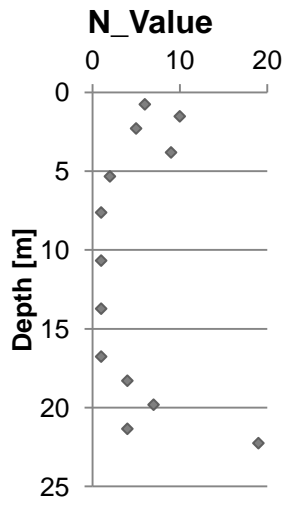
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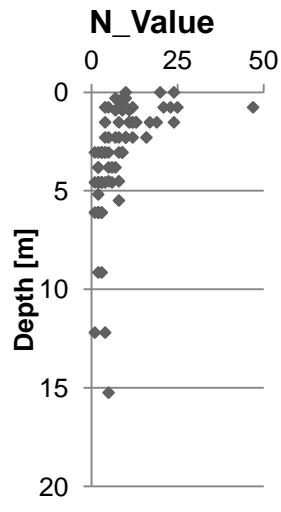
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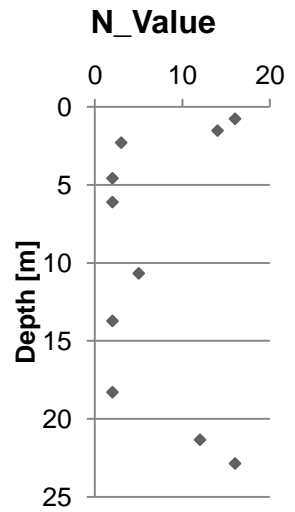
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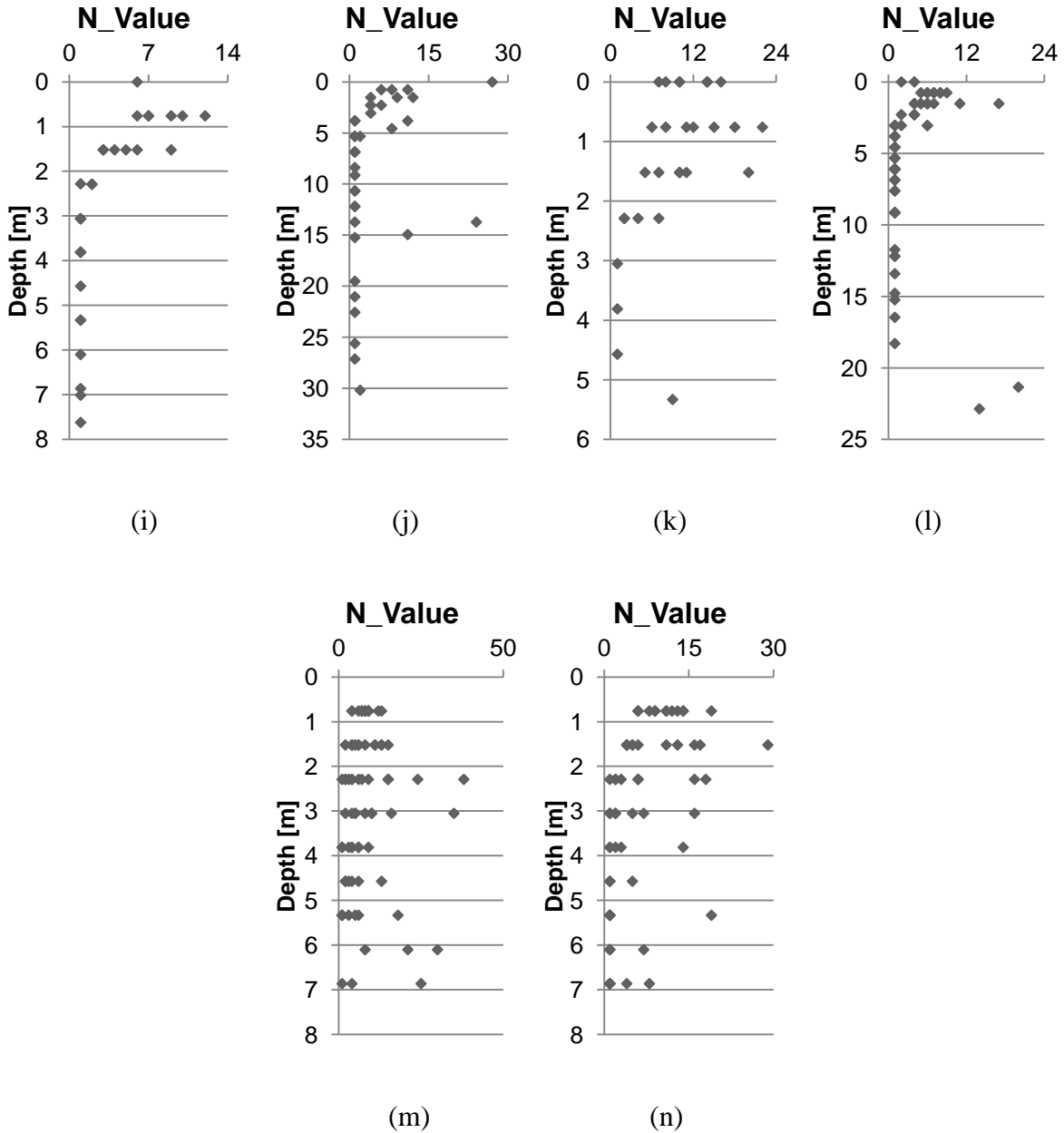


Figure 3-12 N_{60} from SPTs with depth for (a) Site 1 (b) site 2 (c) Site 3 (d) Site 4 (e) Site 5 (f) Site 7 (g) Site 8 (h) Site 9 (i) Site 10 (j) Site 11 (k) Site 12 (l) Site 13 (m) Site 14 (n) Site 15

3.4.3 Vane Shear Tests and Undrained Shear Strength

The undrained shear strength values obtained from the vane tests performed on the sites studied (except for Site 6) are presented in Figure 3-13 along with the shear strength values estimated from the N_{60} values. The Canadian Foundation Engineering Manual (2006) presents a correlation

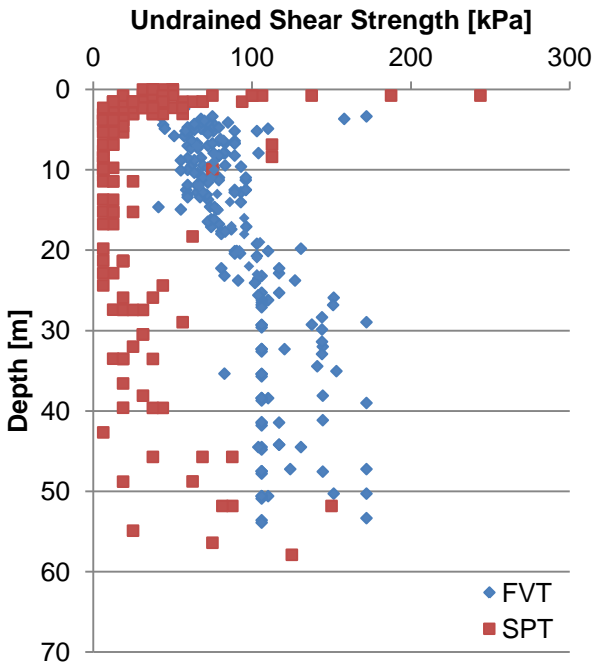
between the undrained shear strength of clay soils and N_{60} from SPTs in Table 3.3 in the manual. However, from Figure 3-13, it can be found that there is an underestimation of the undrained shear strength which was estimated from the SPTs compared to the vane results with all of the sites taken into consideration. Therefore, the results suggest a disagreement of the results between the field SPTs and vanes for the marine clays in Ottawa on the one hand, and the correlation presented in Table 3.3 in the aforementioned manual on the other hand.

Konrad and Law (1987) mentioned that vanes measure the undrained shear strength in the horizontal direction as the vanes turn horizontally in soil. In addition, the horizontal effective stress is lower than the vertical effective stress for normally consolidated and lightly overconsolidated soils such as sensitive marine clays in Ottawa. Therefore, the undrained shear strength measured by using vanes is lower than that measured from CPTs and triaxial tests on vertically sampled specimens.

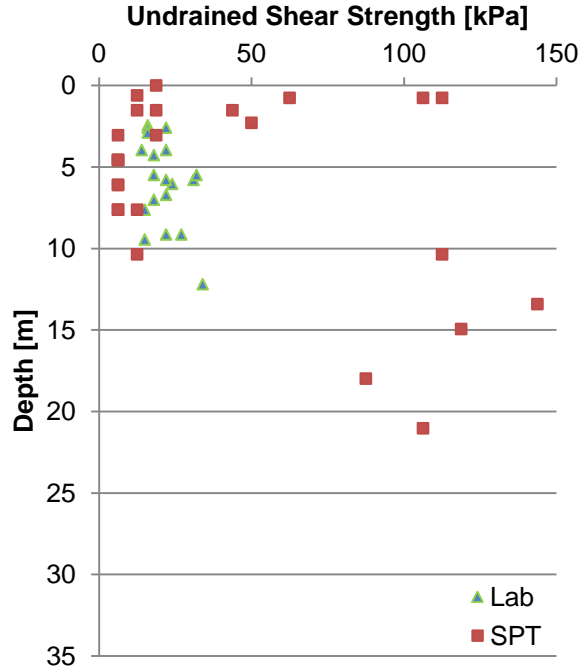
Yafrate and Dejong (2006) reported values of undrained shear strength of 20 and 34 kPa at 4.5 and 8.5 m, respectively, from field vanes at Site 6. Dejong et al. (2011) and Yafrate et al. (2009) reported undrained shear strength values of 20 and 34 kPa from field vanes at depths of 4.4 and 8.3 m, respectively, for the same site.

The results of the undrained shear strength values obtained from the vane tests presented in Figure 3-13 are in agreement with those reported in previous studies. For example, Eden and Crawford (1957) performed tests for five vane profiles in Ottawa. The first profile showed a slight increase in the shear strength with depth at approximately 21 kPa near the surface and 72 kPa around a depth of 18 m. The second profile showed a decrease and then an increase in the shear strength halfway with depth at approximately 70 kPa near the surface, 32 kPa at a depth of 14.6 m, and 110 kPa at a depth of 27 m. The third profile had a general trend of decrease and a shear strength range between 57 and 145 kPa. The fourth profile showed a constant trend in the shear strength with depth between 57 and 91 kPa to a depth of 12 m. The fifth profile showed a slight increase in the shear strength with depth between 57 and 145 kPa to a depth of 15 m. The fissured or crust part of the clay seemed to have approximately the maximum or higher value of shear strength of the soil profile. The crust layer had shear strength of 62 kPa in the first profile and 187 kPa in the second profile. They mentioned that the field vane tests (FVTs) can give results that are many times higher than the laboratory results. Mitchell (1970) reported vane test

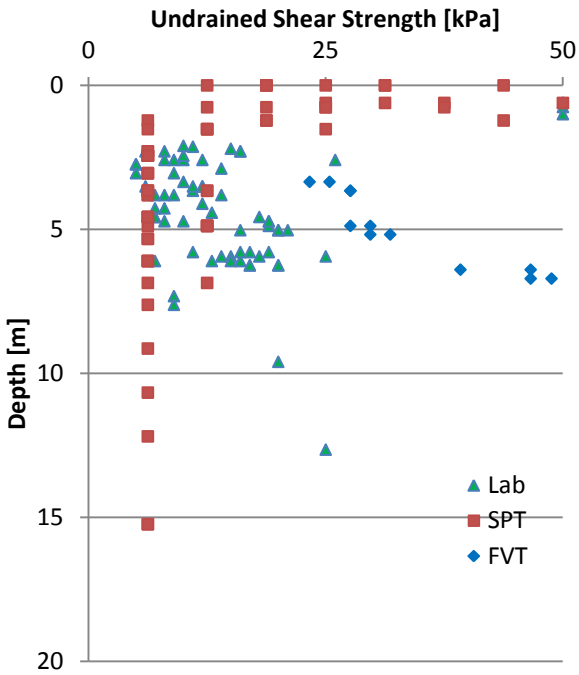
results for Site 6 from block samples as 127.5, 156.9, and 166.7 kPa for depths of 13.7, 16.1 and 19.0 m respectively. Dayal (1970) reported undrained shear strength values from other researchers, such as Crawford (1961), Crawford (1968), and Crawford and Eden (1965), for sensitive marine clays in a range of 9.5 to 383 kPa.



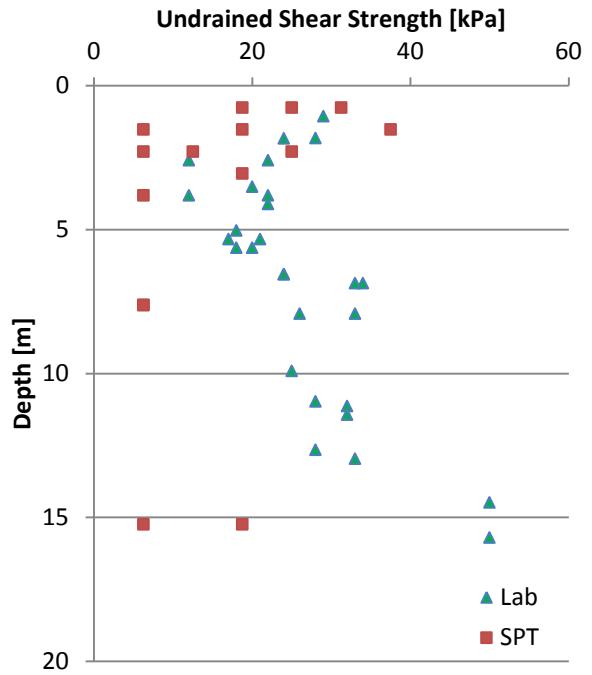
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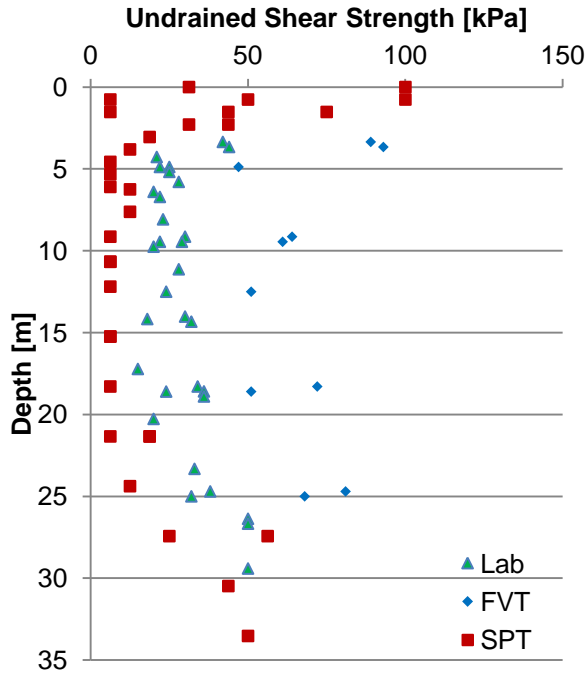
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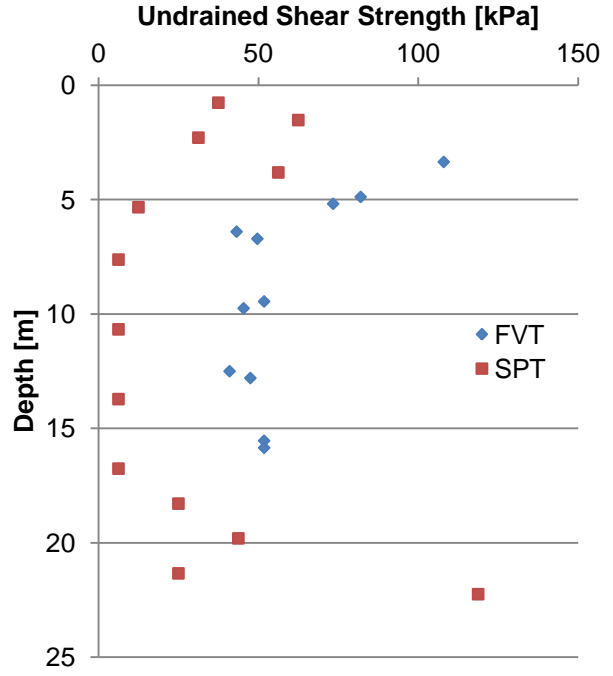
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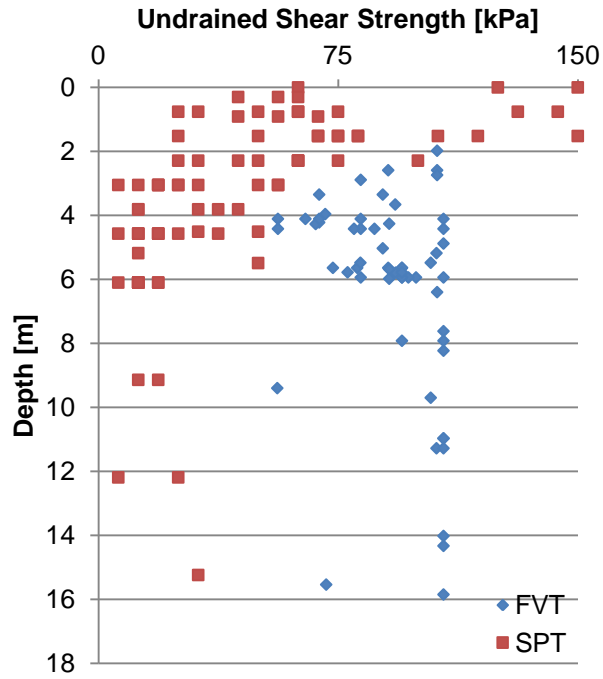
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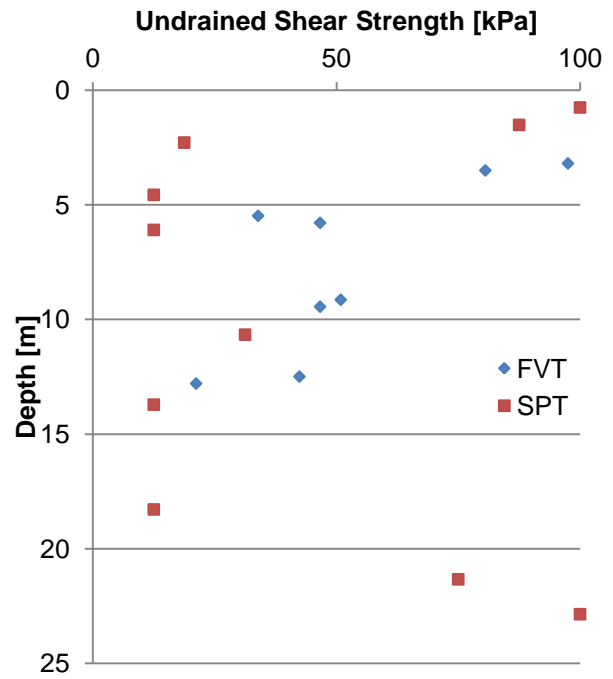
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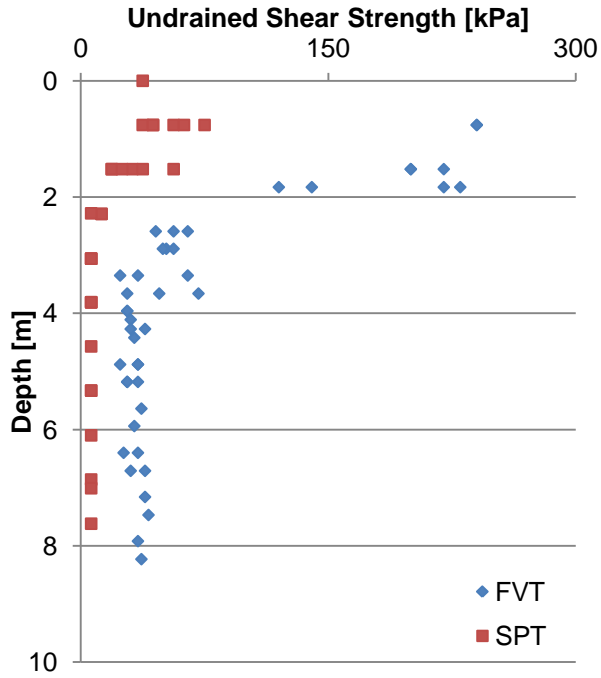
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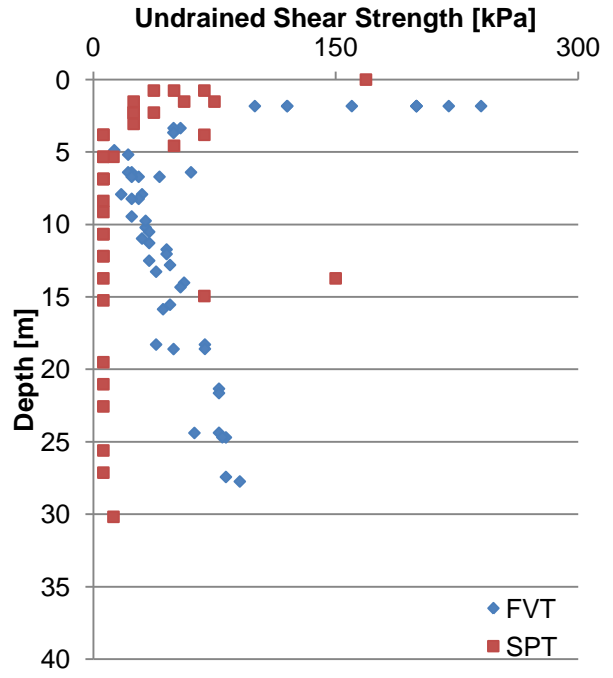
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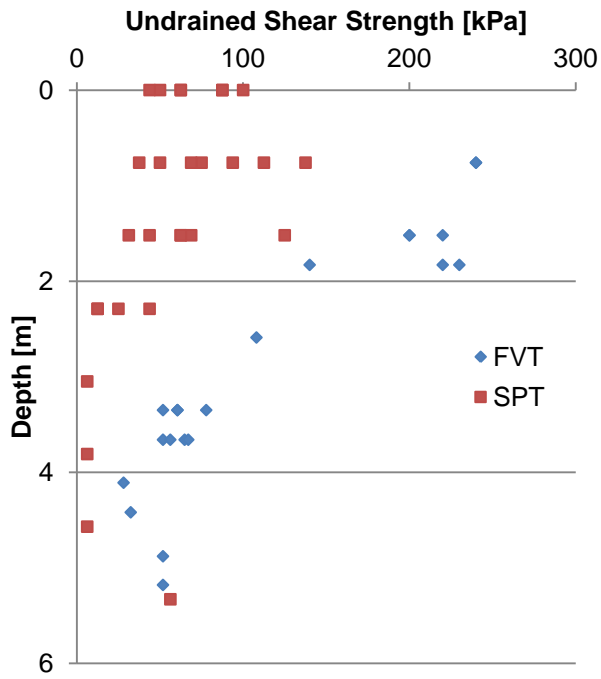
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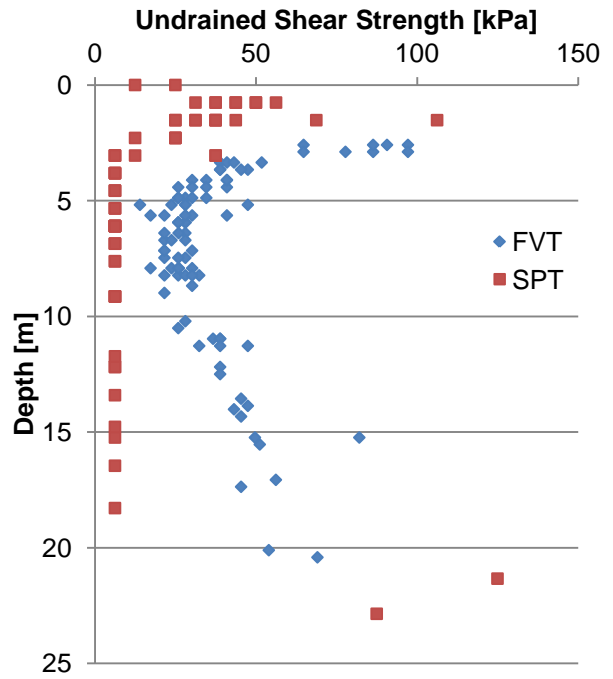
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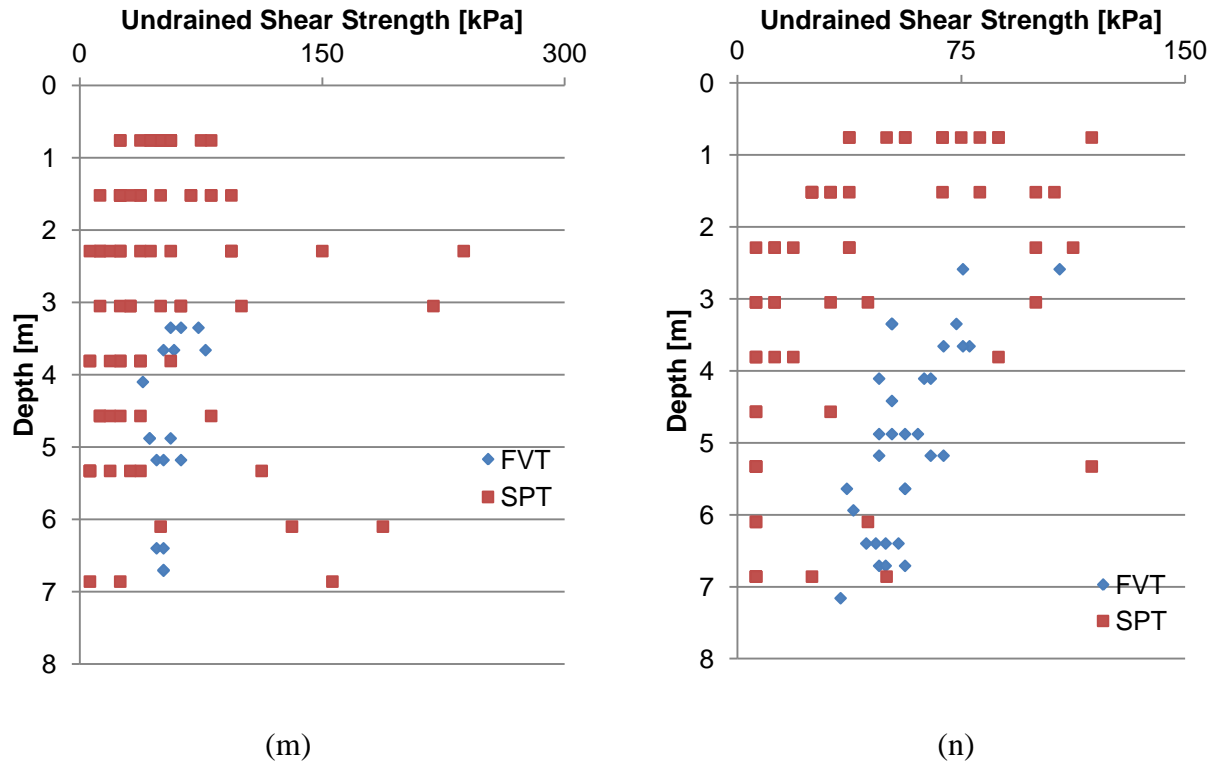


Figure 3-13 Undrained shear strength from field vane tests (FVT), lab vane tests (LVT), and/or SPT correlation for (a) Site 1 (b) Site 2 (c) Site 3 (d) Site 4 (e) Site 5 (f) Site 7 (g) Site 8 (h) Site 9 (i) Site 10 (j) Site 11 (k) Site 12 (l) Site 13 (m) Site 14 (n) Site 15

3.4.4 FVTs and LVTs

The objective of this section is to compare the results obtained from FVTs performed at Site 6 and laboratory vane tests (LVTs) performed on undisturbed samples obtained from the same site. For the former, a rotation rate of 0.1 degree/s was used in accordance with ASTM D2573. The vane was the lower end tapered type with dimensions of 130 x 65 mm. For the latter, a rotation rate of 1 degree/s was used in accordance with ASTM D4648. The vane had a cross section that was 19 x 12.7 mm. Ten rapid rotations were performed before determining the remolded shear strength for both types of tests. Landon-Maynard et al. (2011) mentioned that there should be an adjustment between the field and the lab vanes as a result of the rotation rate. They indicated that the shear strength should be increased by 13% for each order of magnitude of increase in the rate of rotation. Therefore, the LVT results should be multiplied by a factor of 2.3 ($0.13 \cdot 10 + 1$) to

correct for the rotation rate as the lab vanes were turned at 10 times the rate of the field vanes. It can be seen from Figures 3-14a and b that in the undrained and remolded shear strengths from both tests, the use of the lab vanes results in significant underestimation (or overestimation in some cases) compared to the use of field vanes. The difference could be several orders of magnitude. Therefore, this type of correction may not be applicable for sensitive marine clays in Ottawa due to the high dependency on the rate of rotation. However, the sensitivity seems to have a somewhat better agreement between the field and lab vanes than the undrained and remolded shear strength. The adjusting of the lab vanes to the same rate of rotation as the field vanes is an option to consider for future research. The ASTM also recommends the correction indicated by Ladd (1975) and Larsson (1980) by multiplying the result by a factor obtained based on the plasticity index of the soil. This correction has been applied to the results. The ASTM standard recognizes that soft soils may be tested in the field with the maximum rate in the average specified (0.05-0.2 degree/s). Due to the sensitivity of marine clays in Ottawa, strength may be regained during the testing, thus resulting in difficulties, especially in determining the remolded shear strength. At low rates of rotation, the maximum shear strength or well defined failure point may not appear even after turning the vanes to more than 90 degrees. Figures 3-15a and b show that that increases in shear strength may continue as a result of a slow rotational rate which allows sensitive marine clays in Ottawa to regain strength during testing. The gain in strength may be due to the gain of strength at the vanes or on the turn rods in the form of rod friction. Figure 3-15a shows a gain of strength that is dominated by the soil on the vane as it is at a shallower depth than that of Figure 3-15b. Figure 3-15b on the other hand, shows the regain of strength dominated by the soil which acts on the turn rods in the form of rod friction as it is deeper than that in Figure 3-15a and contains up and down waves as a result of failure envelopes.

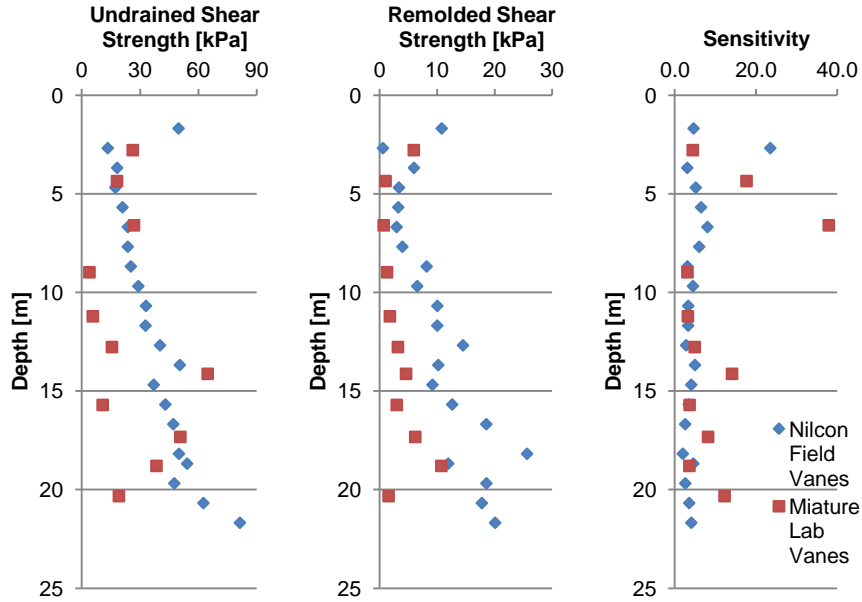


Figure 3-14 Results of Nilcon field and miniature lab vanes in Site 6 (a) undrained shear strength with depth (b) remolded shear strength with depth (c) sensitivity with depth

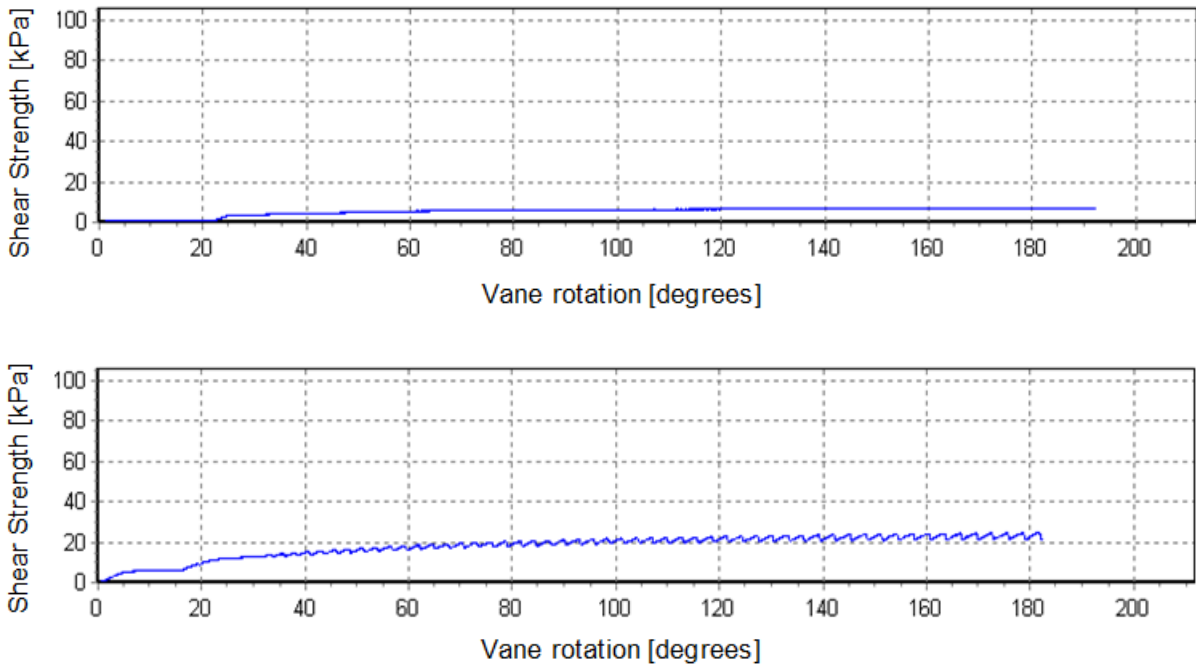
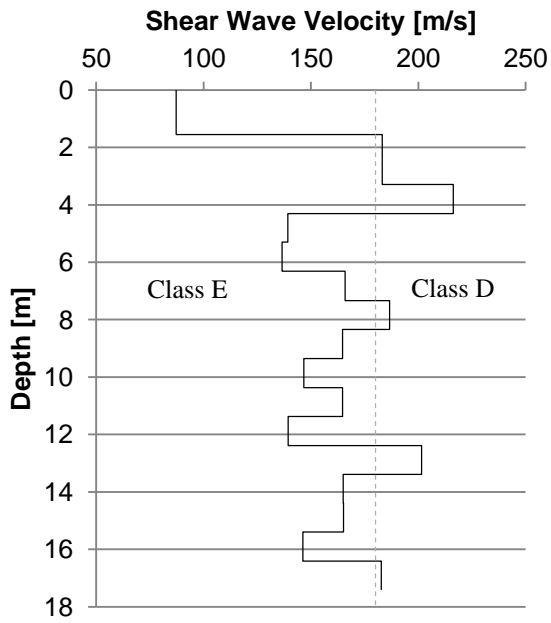


Figure 3-15 Remolded shear strength with rotation angle from Nilcon field vanes for rotational rate of 0.1 degree/s in Site 6 at a depth of (a) 3.68 m, and (b) 16.68 m

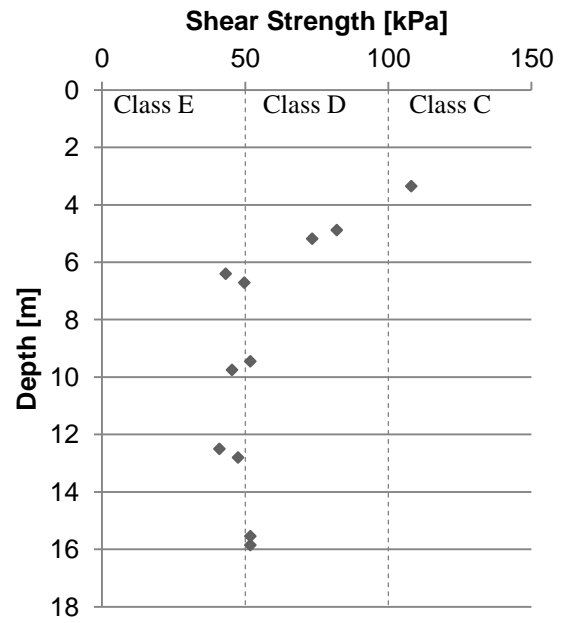
3.4.5 Shear Wave Velocity Measurements

The Canadian Foundation Engineering Manual (2006) presents a correlation between the site class on the one hand and the soil shear wave velocity or the undrained shear strength on the other hand in Table 6.1A in the manual. There are seven site classes from A to F that are used to find site coefficients for seismic design purposes. The range of the values of the shear wave velocities (over 180 m/s and under 1500 m/s) and undrained shear strength (over 50 kPa and under 100 kPa) are used as the limits to define site classes.

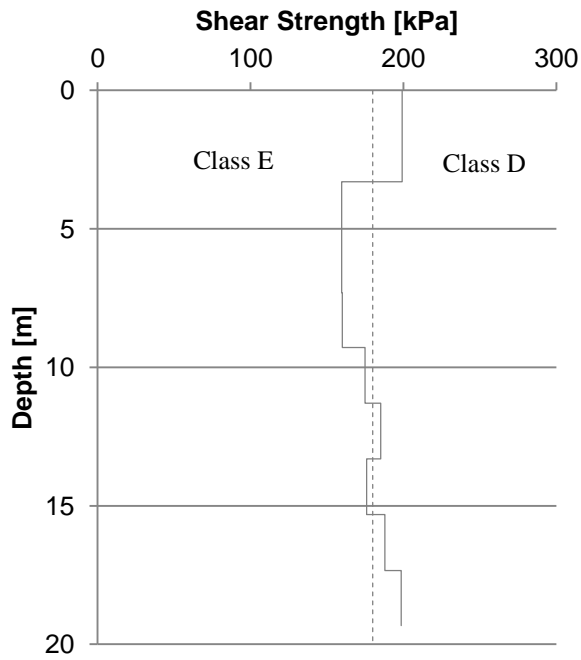
For sites with available shear wave velocity measurements and vanes, comparisons between the site class defined by using the shear wave velocity measurements and undrained shear strength are presented in Figure 3-16. For ease of explanation, Method 1 represents the site class that uses the shear wave velocity measurement, and Method 2 represents the site class that uses undrained shear strength. For Site 7, Method 1 renders that the site class ranges between D and E while Method 2 renders the site class that fluctuates between C, D and E. At a depth of approximately 3 to 4 m, Method 1 peaks in Class D while Method 2 peaks in Class C. It is observed that the general trend of the two methods is in agreement, but in different site classes, which indicates a site class limits problem. In Site 8, there is a misestimation of a whole class letter. In other words, Method 1 results in Classes D and E while Method 2 results in Classes C and D. Site 9 has five shear wave velocity profiles and one vane profile. Method 2 ranges between Classes D and E, and approaches Class C near a depth of approximately 3 m. Method 2 results in five different trends of site classifications from five profiles. All of the profiles from Method 1 range between Classes D and E, except for Figure 18(c) which extends to Class C near the surface. In Fig. 18(g), the site class extends into Class D near the bottom of the profile while Method 2 profiles fluctuate between Classes D and E throughout and near Class C near the surface. As there may be underestimations in the estimation of undrained shear strength in the Canadian Foundation Engineering Manual for sensitive marine clays in Ottawa, there is disagreement in the estimation of site class from the undrained shear strength profiles. The number of data sets is not conclusive but raises the issue for future research. A point to mention is that according to the seismic site class map of Ottawa in the National Building Code of Canada (NBCC), all five classes from A through F are found in Ottawa.



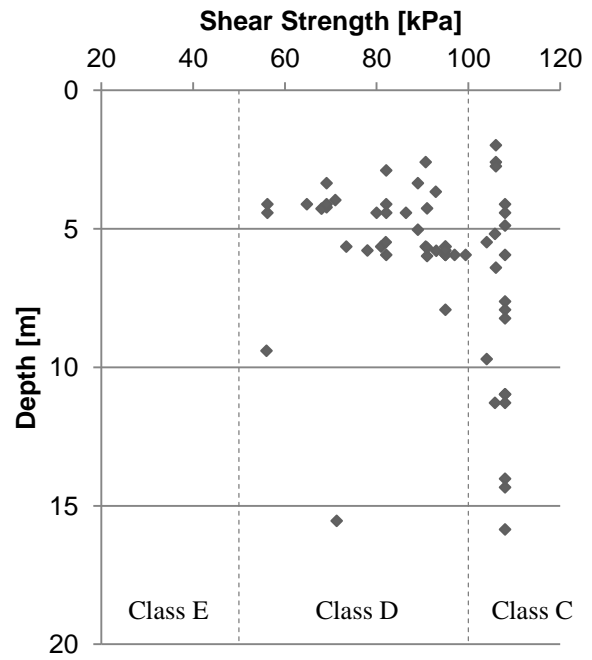
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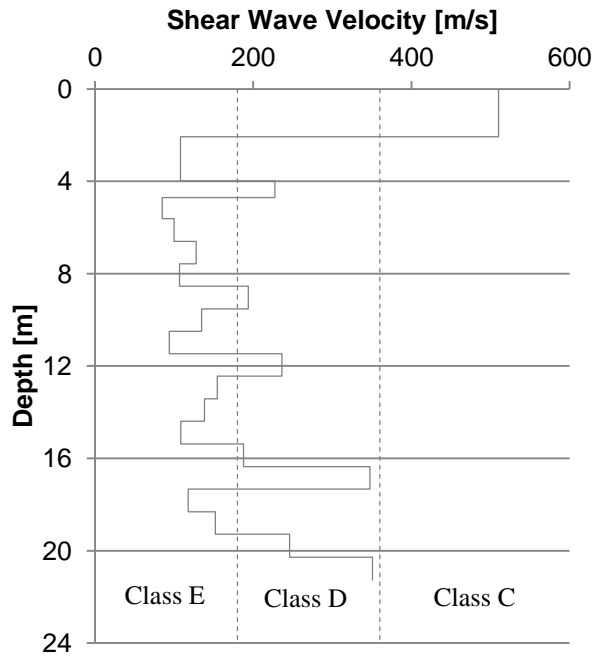
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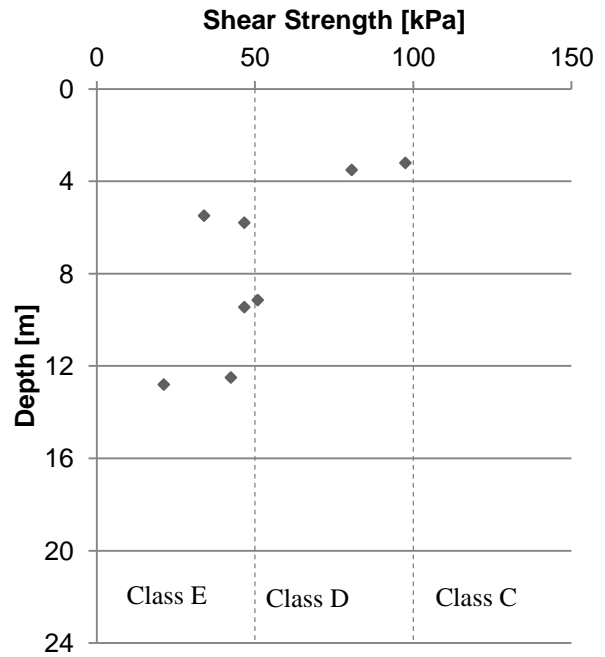
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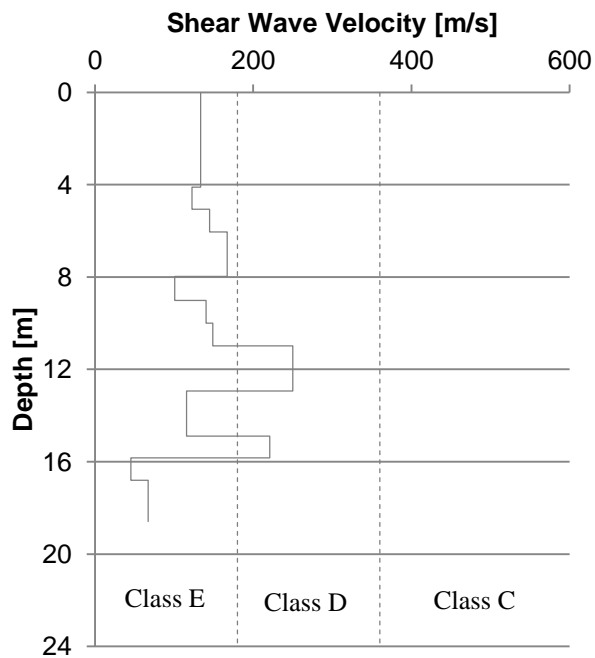
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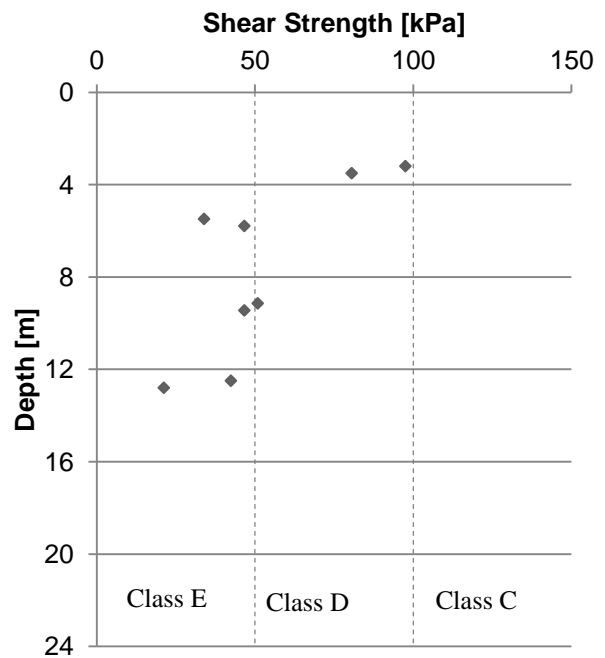
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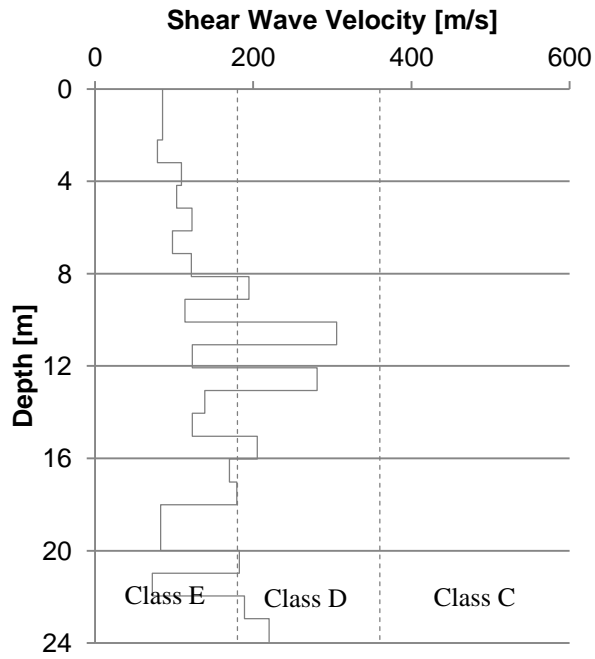
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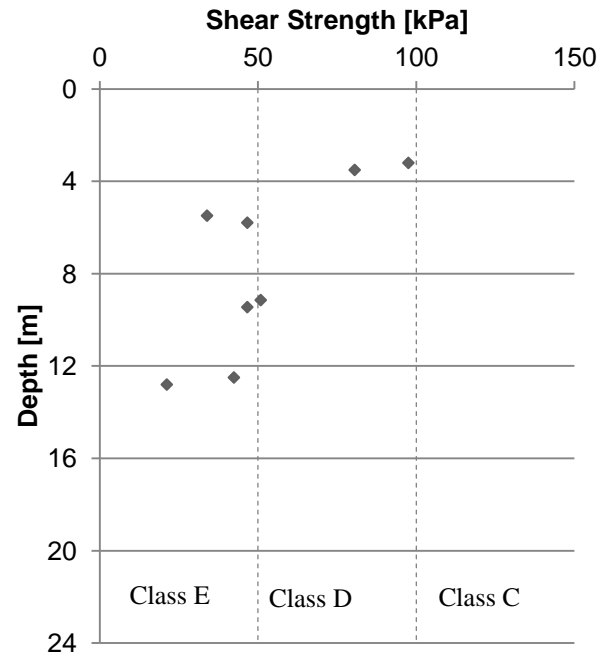
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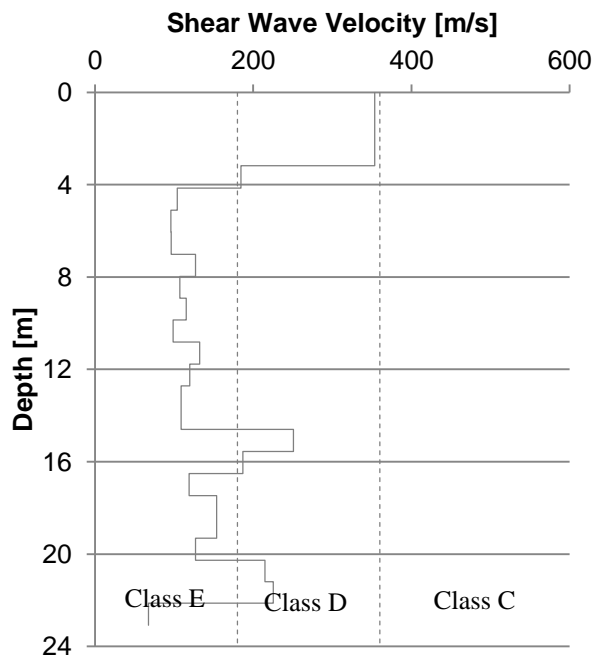
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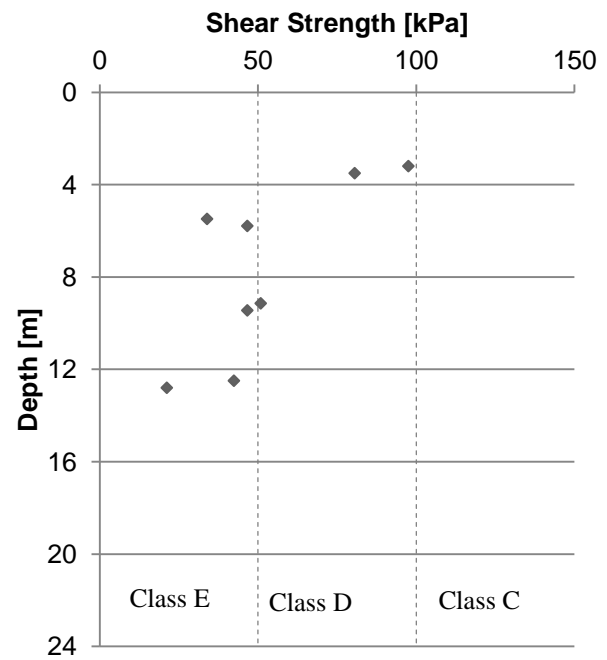
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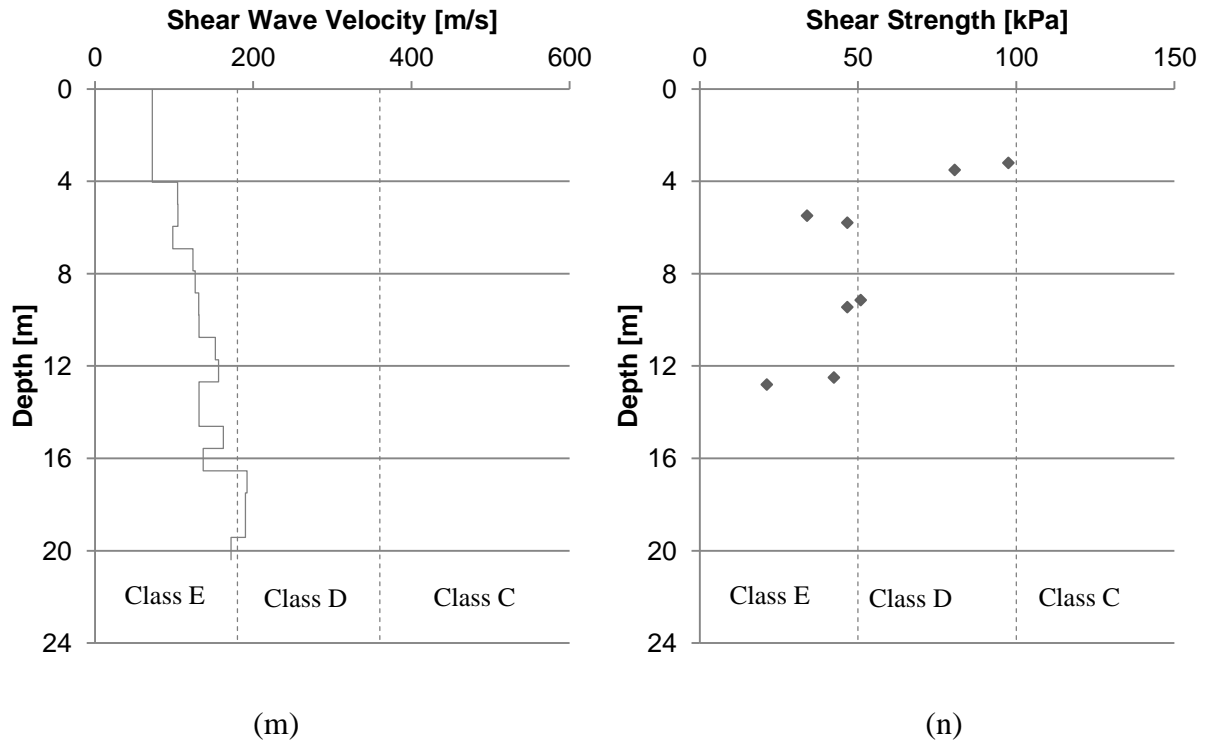


Figure 3-16 Shear wave velocity and shear strength with depth and site classification according to the Canadian Engineering Foundation Manual for (a) Site 7 (b) Site 7 (c) Site 8 (d) Site 8 (e) Site 9 (f) Site 9 (g) Site 9 (h) Site 9 (i) Site 9 (j) Site 9 (k) Site 9 (l) Site 9 (m) Site 9 (n) Site 9

3.5 Interface Shear Strength and Behavior

An understanding of the interface shear behavior between sensitive marine clays and structures is significantly important for the safe and cost effective designs of several geotechnical structures (e.g., friction piles, retaining walls, and anchors). Taha (2010) and Taha and Fall (2010) performed a series of interface shear testing on marine clay samples from Ottawa to examine the shear behaviour and strength at the interfaces of marine clay - steel and marine clay - concrete. A series of consolidated drained and unconsolidated undrained shear tests were performed to examine the interface with steel. The samples had different overconsolidation ratios (OCRs) and saturation values. They noticed that by applying a normal stress of 250 kPa, the clay alone exhibits higher shear resistance than in the interface of the marine clay - steel. However, at normal stresses higher than 250 kPa, shear resistance of the clay alone is the same as that of the interface zone. They attributed this behavior to the tendency of marine clays to remold at higher

normal stresses. They also found that an increase in the steel surface roughness can increase the resistance shear strength as a result of the increase of the interlocking forces between the clay and steel. This is similar to the findings of Tsubakihara and Kisheda (1993). Moreover, they found that denser Ottawa marine clay has higher interlocking forces between the clay and steel. They showed that increasing the surface roughness increases the vertical deformation or displacement of the marine clay, and small changes in the surface roughness (1 to 5 μm) does not induce significant deformation differences. When the surface roughness increases, the internal friction angle increases. Therefore, when the internal friction angle approaches the internal friction coefficient of the soil, shear failure would occur in the soil rather than in the interface (Hamid and Miller 2009; Taha, 2010; Taha and Fall, 2010). The effect of the OCR decreases with increasing the normal stress on the soil. However, the shear strength of marine clay increases with increasing the OCR as a result of higher density associated with a higher OCR. Also, the angle of internal friction is higher when there is increase in the OCR (DeJong and Westgate 2005; Taha, 2010; Taha and Fall, 2010). As the degree of saturation decreases or the suction increases, the interface shear strength increases (Fleming et al. 2006; Hamid and Miller 2009; Taha, 2010; Taha and Fall 2010). Similar to the OCR effect, as the density increases, the interface dry density increases. This is attributed to the increased density and higher interlocking forces between the clay and steel. Therefore, Taha (2010) suggested that the soil be compacted in order to obtain higher interface shear strength.

For the concrete-clay interface, it was observed that the shear resistance is in the sensitive clay layer above the interface layer up to a deformation of 2%. After a displacement of 2%, the shear strength increased as the roughness increased. It was found that the difference in the shear strength between a roughness of 6.5 and 10 μm is not as significant as that between 10 and 20 μm . The reason is attributed to the interlocking forces between the soil and concrete and the accuracy of the surface roughness measurements. Also, there is higher shear strength because there is higher normal stress. It is worth noting that the surface roughness is not significant when there is low normal stress (250 kPa) as the interlocking forces between the soil and concrete require high stress to break. The angle of the interface friction increased as the surface roughness increased, but was not sensitive to small changes, such as 6.5 to 10 μm (Taha and Fall 2010). Clay at the interface deformed less than pure clay, especially at high shear increments. The reason is that concrete roughness can cause remolding and reorientation of the clay particles. The

shear stress and displacement were found to be independent of the consolidation state of the clay (Taha et al. 2010; Tsubakihara and Kisheda 1993).

The interface shear strength increased as the salt or sodium chloride concentration increased (Taha et al. 2010). Torrance (1975) attributed this behavior to the positive charges of the clay particles which exhibit negative charges on their surfaces. Higher ion concentrations reduce the effect of the charges on the edges which make the clay particles less repulsive to each other. It also facilitates the formation bonds between the positively charged edges and negatively charged surface of the clay particles. The interface shear strength increases as the degree of saturation decreases and the suction increases. This can be explained by the increase in the interlocking forces between the clay and concrete and between the clay particles as the suction increases. It was found that the peak interface shear strength can be achieved within a range of 1% to 3% of displacement (Taha and Fall, 2010). The interface shear strength increased as the density increased. This is because there are higher interlocking forces between the clay and concrete in denser soils and there is a larger interaction area between the clay and concrete (DeJong and Westgate 2005; DeJonge and Westgate 2009).

3.6 Consolidation Behavior

3.6.1 Preconsolidation Pressure and Over Consolidation Ratio

The preconsolidation pressure is one of the most important characteristics of clays: it is the vertical effective stress beyond which large strains take place, particularly in sensitive clays (Leroueil et al. 1983). The preconsolidation pressures and OCRs of the sensitive marine clays in the studied sites are shown in Figure 3-17. It is evident that the preconsolidation pressures generally increase with depth, while the OCRs slightly decrease with depth. The preconsolidation pressure ranges between 40 and 660 kPa for all of the sites. The OCR ranges between 1.4 and 7.1 for all of the sites.

Sensitive marine clay in Ottawa is known to have a so called "card-house" structure. This type of structure does not experience significant settlement before reaching the preconsolidation pressure. After reaching the preconsolidation stress, the soil experiences a sudden high settlement and clay plates orient into a form parallel to each other and perpendicular to the direction of the consolidation (Taha, 2010). Therefore, the void ratio is reduced, and bonds

between the soil particles break. As a result, remolded or disturbed samples may have flatter consolidation curves than intact samples and this may lead to underestimating of the preconsolidation pressure (Quigley and Thompson 1966; Taha, 2010).

The results presented in Figure 3-17 are in agreement with those reported by other researchers. For instance, preconsolidation pressure and OCR were determined to be 150 kPa and 1.1 respectively for samples obtained by Taha (2010). Leroueil (1999) reported that the OCR for eastern Canada clays ranges between 1.2 and 5. Dejong et al. (2011) reported an OCR between 1.8 and 1.5 for depths of 4.4 and 8.3 m respectively in Site 6. Rasmussen (2012) reported preconsolidation pressure, overburden stress, and OCR ranges of 100 to 200 kN/m², 39 to 172 kN/m², and 1.0 to 3.2 respectively for the Ottawa area. Eden and Crawford (1957) reported a preconsolidation pressure for sensitive marine clays in Ottawa of 71.8 kPa. Mitchell (1970) reported a preconsolidation value of 441.3 kPa at Site 6 for block samples obtained at depths of 13.7, 16.1, and 19.0 m. He also reported overburden stress values of 127.5, 147, and 176.5 kPa for the same depths.

The preconsolidation pressure profile for Site 6 was supplied by the National Research Council of Canada and published by Hinchberger and Rowe (1998). This profile has a pattern of increases with depth from approximately 48 to 171 kPa at a depth of 1 to 17 m, respectively.

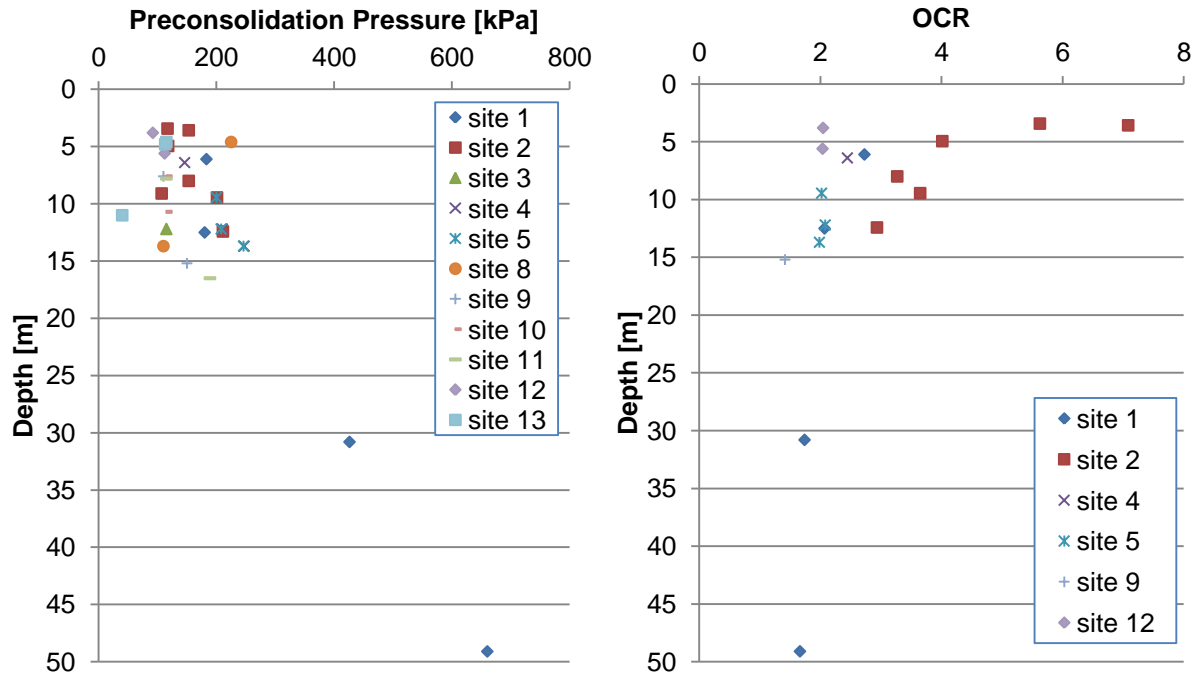


Figure 3-17 (a) Preconsolidation pressure with depth (b) OCR with depth for all of the sites

3.6.2 Compression Index

The compression indexes of the marine clays of the studied sites are shown in Figure 3-18. The compression indexes were determined from the slope of the virgin compression line from the oedometer tests. There does not seem to be a clear relationship with depth. The C_c ranges between 0.44 and 2.01 for Site 6 and between 0.4 and 2.24 for all of the other sites.

The compression index (C_c) was determined to be 2.44 for the samples tested by Taha (2010). They mentioned that the calculated C_c is 2.34 verifies the formula proposed by Leroueil (1999):

$$C_c = 0.65e_o^2 + 0.15e_o - 0.5$$

Holtz and Kovacs (1981) indicated that the C_c of Canadian marine clays including those in the Ottawa region is between 1 and 4. The C_c profile for Site 6 was supplied by the National Research Council of Canada, and published by Hinchberger and Rowe (1998). The C_c values range between approximately 0.25 and 1.9. The values generally show a pattern of increase, from a depth of approximately 1 to 10 m, then a pattern of decrease, from a depth of 10 to 19 m. An average value of 0.65 was designated by Hinchberger and Rowe (1998).

Sensitive marine clays in Ottawa do not have a constant compression index, but the compression index decreases as the stress increases past the yield pressure (Mitchell 1970; Rasmussen 2012). Sensitive marine clays in Ottawa exhibit high initial values in the compression index as a result of the sudden break of the cementation bonds between the clay particles of the clay (Jarrett 1967; Rasmussen 2012; Walker and Raymond 1968).

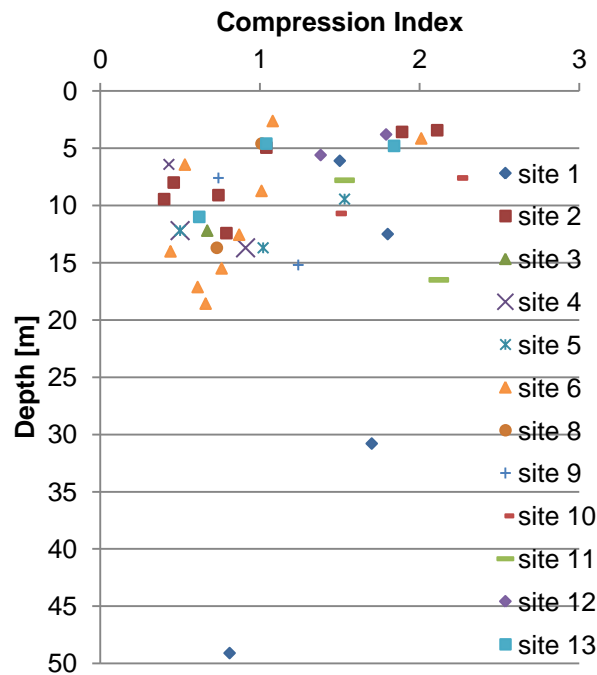


Figure 3-18 Compression index with depth

3.6.2.1 Example of Consolidation Curve

Figure 3-19 is a typical example of an oedometer test for sensitive marine clay in Ottawa. It can be seen that the slope of the curve beyond the preconsolidation pressure decreases with increasing the stress. Therefore, there could be more than one compression index for the same soil. From general engineering practices, it is recommended that the compression index and other indices and coefficients from the loading cycle that is closest to the field condition be determined. Sometimes engineers use the stage with the highest compression index for a higher factor of safety in designs, which is usually the stage directly after the preconsolidation pressure.

Figure 3-19 also shows a sudden break of the soil at the preconsolidation pressure. The reason behind this behaviour is the cardhouse structure of the soil. The porewater and the soil skeleton take the stress and resist consolidation. Eventually, they fail, and cause a high degree of settlement at the break point (Taha,. 2010). Therefore, for determining the preconsolidation pressure, it is necessary to avoid the loading of the soil with double the stress near the break point (ASTM D 2435). The ASTM D 2435 standard recommends the use of a double scheme for loading. However, this can cause a reduction in accuracy when determining the preconsolidation pressure.

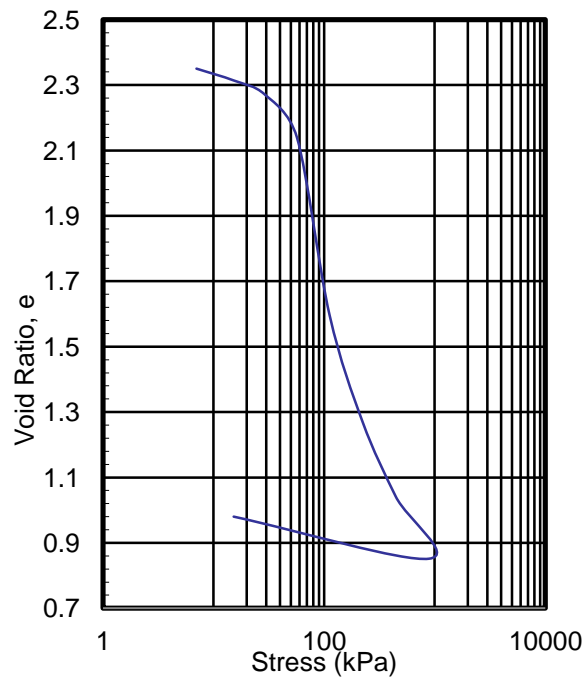


Figure 3-19 Consolidation curve for a depth of 3.8 m at Site 6

3.6.2 Recompression Index

Figures 3-20 show recompression indexes with depth for the studied sites. It can be observed that there is no clear relationship between the recompression indexes and depth. The recompression index for Site 6 ranges between 0.03 and 0.09 and between 0.02 and 0.09 for all of the other sites. It can be seen from Table 3-3 that the ratio of the compression index to the recompression index ranges between 3.4 and 76.5 with an average of 36.1 with two outliers of 140.7 at Site 2 and 106 at Site 11.

Sensitive marine clays in Ottawa in general have low recompression indexes (Crawford 1968). Rasmussen (2012) reported recompression indexes between 0.011 and 0.072 for stresses of 5 to 50 kPa. She also reported swell indexes that range between 0.028 and 0.12 for stresses between the end of her oedometer tests and 50 kPa. She attributed the low values of the swell indexes to the high stress that she had subjected her samples. On average, she had subjected her samples to 1500 kPa while the overburden stresses of the samples were between 39 and 172 kPa. Therefore, there is a high degree of destruction to the internal structure of the clay. A value of 0.025 for C_r was estimated by Hinchberger and Rowe (1998) based on a study by Leroueil et al. (1983) on Site 6.

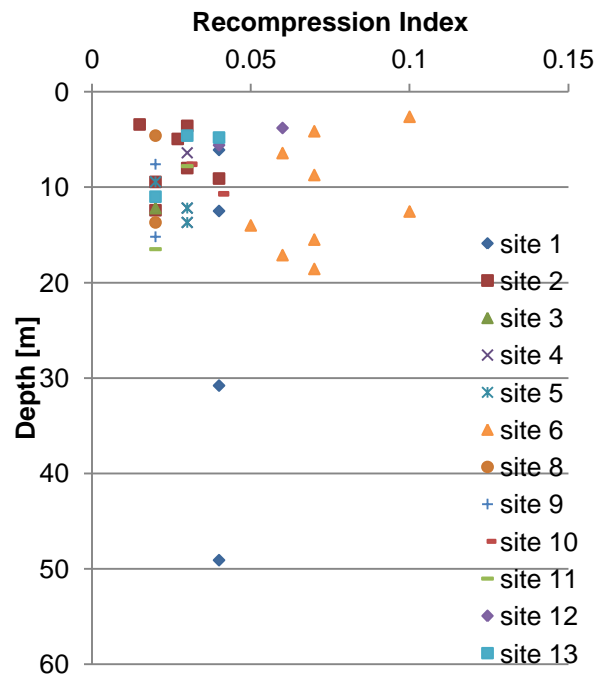


Figure 3-20 Recompression index with depth from oedometer tests

Table 3-3 Compression index to recompression index ratios

Site	Depth [m]	Cc/Cr	Site	Depth [m]	Cc/Cr
1	6.1	37.5	6	2.6	10.8
1	30.8	42.5	6	4.1	28.7
1	12.5	45.0	6	6.4	8.8
1	49.1	20.3	6	8.7	14.4
2	9.1	18.5	6	12.6	8.7
2	3.4	140.7	6	14.0	8.8
2	8.0	15.3	6	15.5	10.9
2	5.0	38.5	6	17.1	10.2
2	9.5	20.0	6	18.6	9.4
2	3.6	63.0	9	7.6	37.0
2	12.4	39.5	9	15.2	62.0
3	12.2	33.5	10	7.6	74.7
4	12.2	16.7	10	10.7	37.0
4	13.7	30.3	11	7.8	51.0
4	6.4	14.3	11	16.5	106.0
5	13.7	34.0	12	3.8	29.8
5	12.2	16.7	12	5.6	34.5
5	9.5	76.5	13	11.0	31.0
8	4.6	50.5	13	4.6	34.7
8	13.7	36.5	13	4.8	46.0

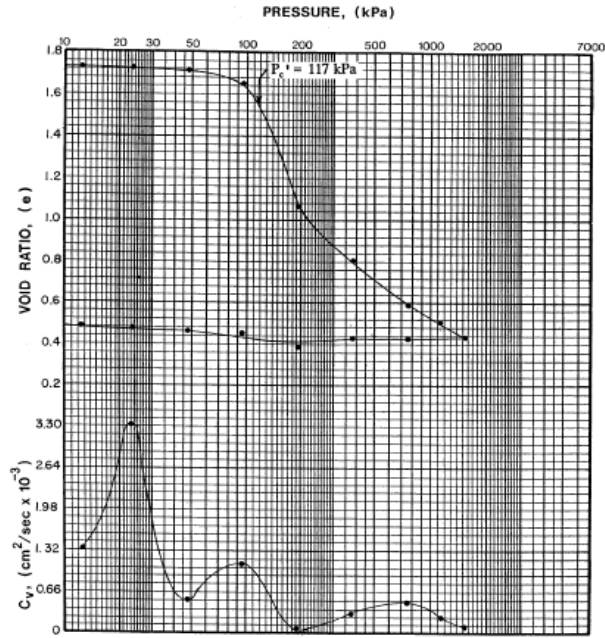
3.6.3 Coefficient of Consolidation

Figure 3-21 shows the variation of the coefficient of consolidation with pressure in the same graph with the consolidation curves. The testing results are from Sites 2 and 3. There is no clear pattern between the pressure and the coefficient of consolidation. Also, there is no clear pattern of increase or decrease in the coefficient of consolidation with pressure in the recompression or virgin compression portion of the consolidation curve.

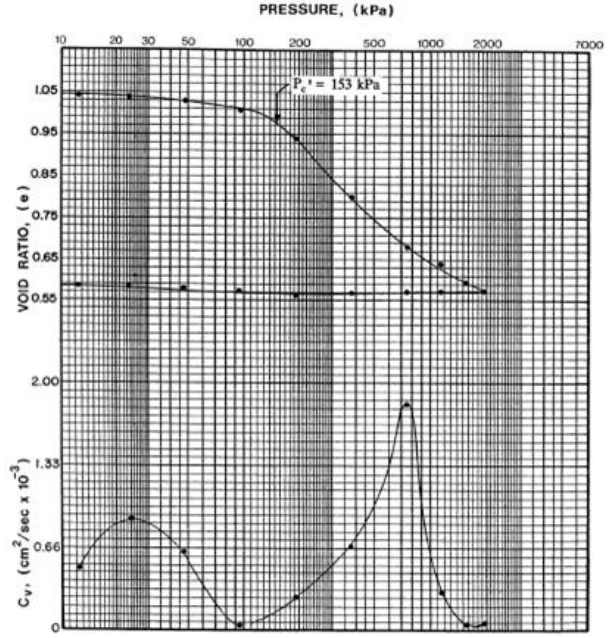
Figure 3-22 shows the variation of the coefficient of consolidation with stress from the oedometer testing for the sites under study. The coefficient of consolidation was not determined for all of the stress intervals due to the lack of the availability of the data. The coefficient of consolidation of all the sites was determined by using Taylor's square root of time fitting method (Holtz and Kovacs 1981). Due to the variability of the coefficient of consolidation, it is recommended that the value of the stress closest to the field condition is used. For the other sites in the study, the t_{90} ranged between 4 and 139 minutes which is a relatively large range. On the other hand, the coefficient of consolidation ranged between 8.99×10^{-5} and $3.3 \times 10^{-3} \text{ cm}^2/\text{s}$.

The coefficient of consolidation generally decreases with increases in the stress. However, it can increase then decrease with increasing stress. As the stress increases, the soil structure and the porewater take the load and resist compression. At this point, there is low consolidation in the soil. Eventually, this structure breaks and causes the rearrangement of the soil structure and increases the coefficient of consolidation. Table 3-4 is a summary of t_{90} and the coefficient of consolidation relative to the stress interval, depth, and preconsolidation pressure from oedometer tests for the sites studied.

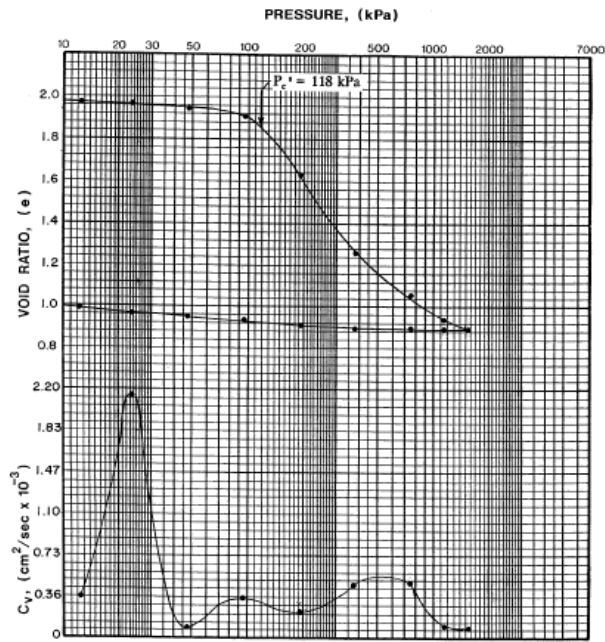
Lo et al. (1976) performed consolidation tests by using large (11.28 cm x 5.08 cm) samples and small (5.08 cm x 1.27 cm) samples, and they found that the smaller samples yielded values of C_v that are an order of one or two magnitudes less than the larger samples (Taylor 1948). They attributed this behavior to higher disturbance associated with the trimming and preparation of smaller samples (Bozozuk 1971; Zelst 1948), higher hydraulic gradients because of a shorter drainage distance, and larger strain rates in the early stages of consolidation. They also attributed this behavior to temperature and mechanical effects from sampling. Lo et al. (1976) reported values of C_v for different depths and load increments. For small samples, the values of C_v are 0.027 and 0.045 cm^2/s for depths of 2.4, 2.5, and 3.4 m, and pressure increments of 38.3 to 88.4 kPa, respectively. For 15.24 cm x 5.08 cm samples of the same increment at a depth of 2.4 m, the C_v was 2.75 cm^2/s from the consolidation testing and 4.74 cm^2/s from pore pressure dissipation testing. In large samples at depths of 4.3 and 11.1 m and pressure increments of 43.1 to 82.4 kPa and 72.8 to 99.6 kPa respectively, the C_v values are 2.67 and 2.59 cm^2/s . Dayal (1970) reported coefficients of consolidation from other researchers, such as Crawford (1961), Crawford (1968), and Crawford and Eden (1965), for sensitive marine clays, which are 1×10^{-2} to $1 \times 10^{-5} \text{ cm}^2/\text{s}$.



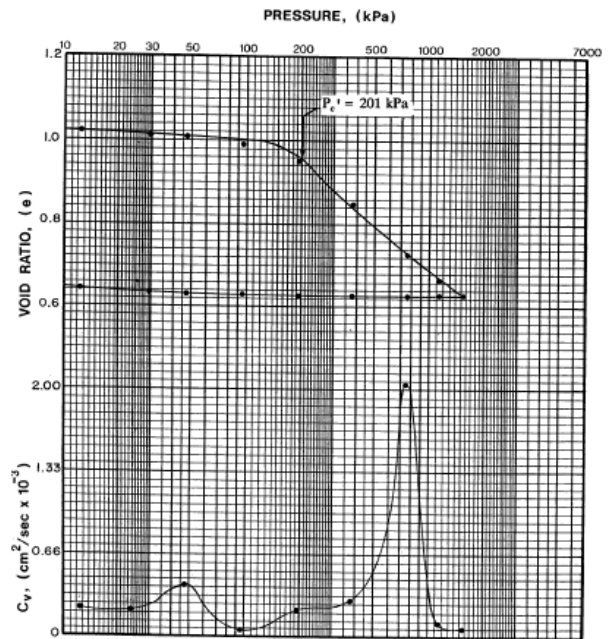
(a)



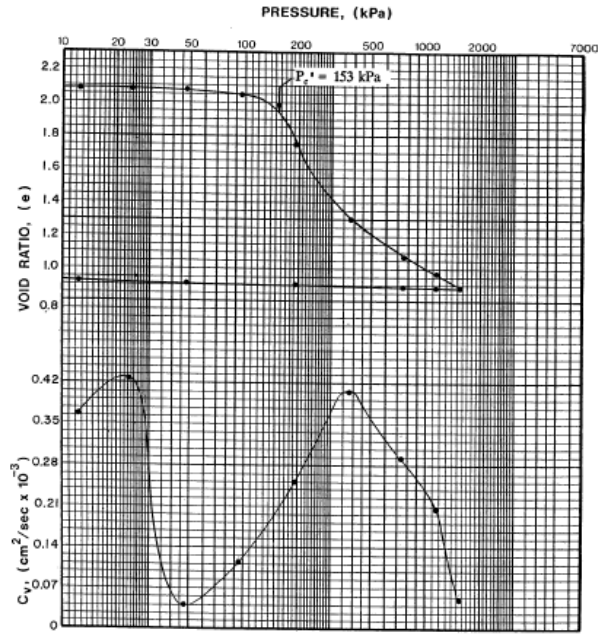
(b)



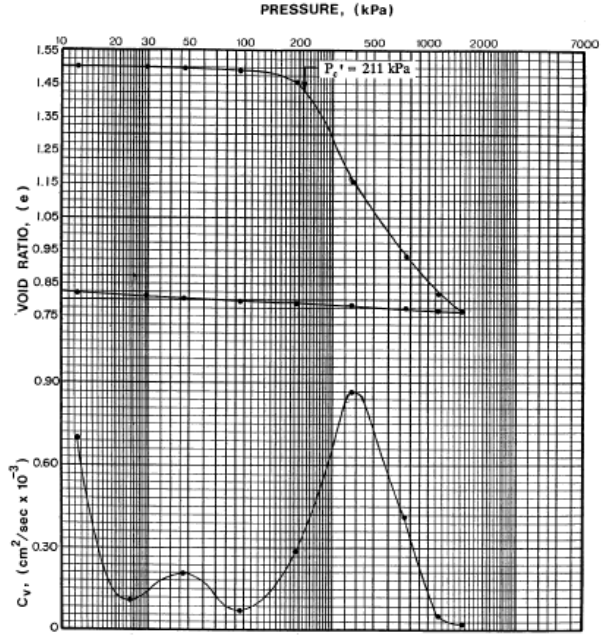
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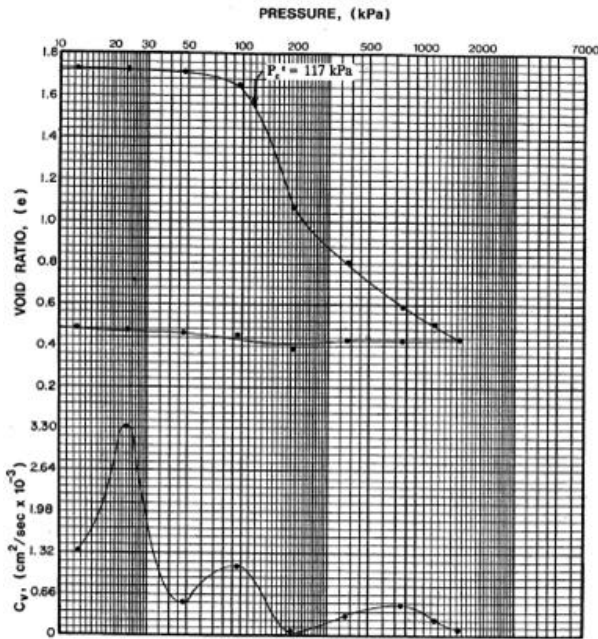
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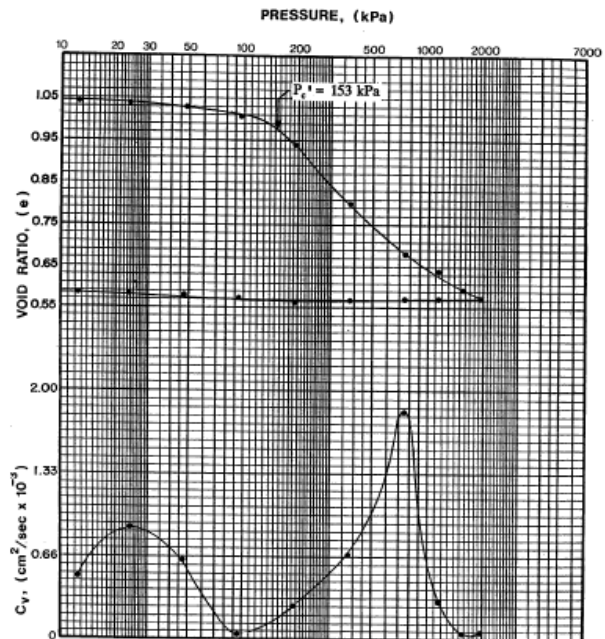
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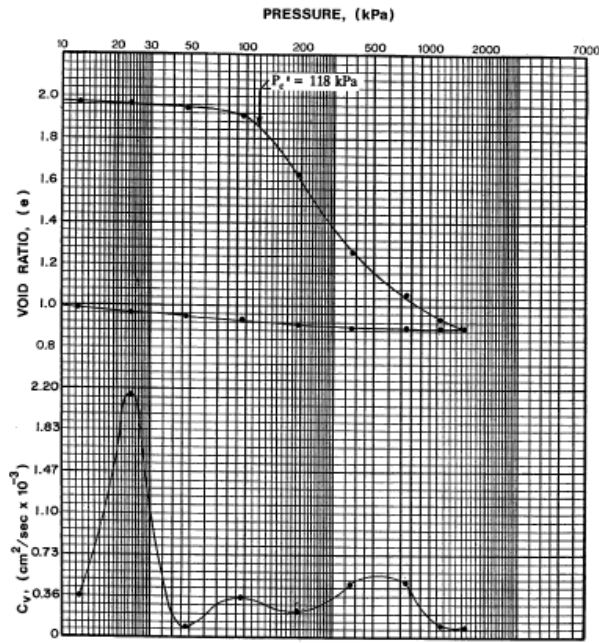
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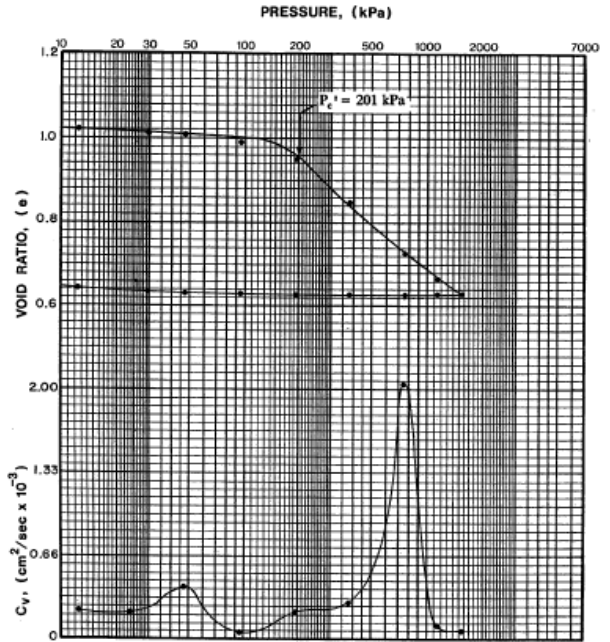
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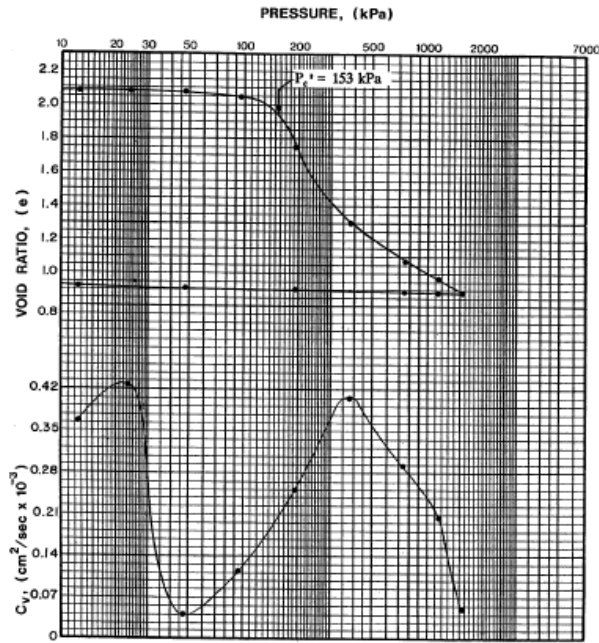
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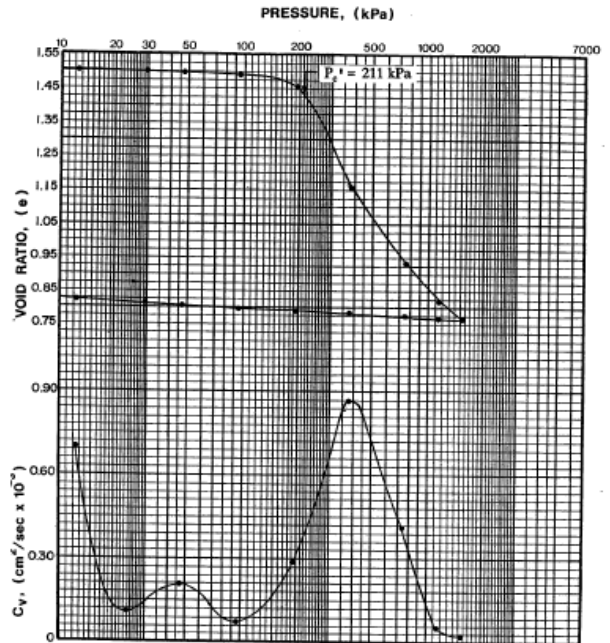
(i)



(j)

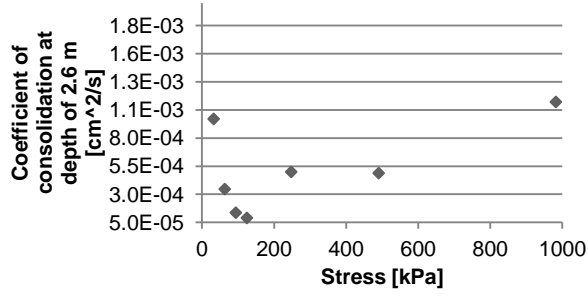


(k)

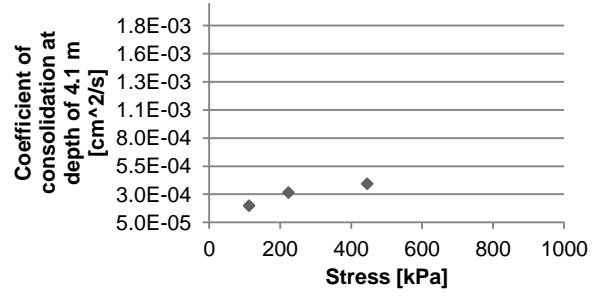


(l)

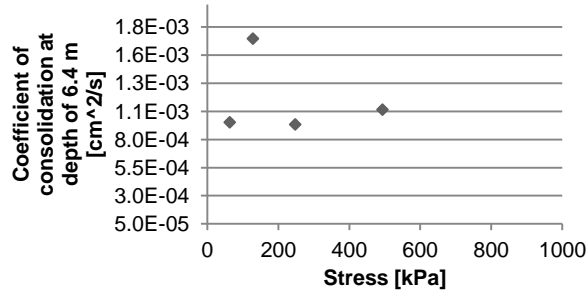
Figure 3-21 Consolidation curve and the coefficient of consolidation with pressure from oedometer tests for Site 2 at depths of (a) 3.43 m (b) 8.0 m (c) 4.95 m (d) 9.45 (e) 3.58 m (f) 12.42 m and for Site 3 at depths of (g) 3.43 m (h) 8.0 m (i) 4.95 m (j) 9.45 (k) 3.58 m (l) 12.42 m



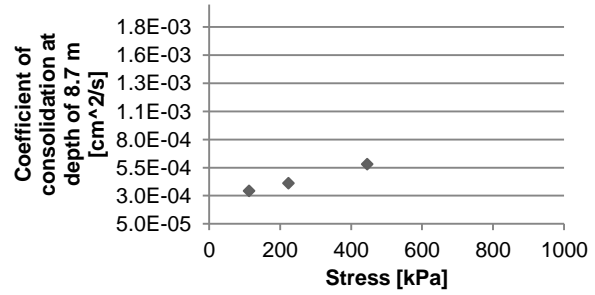
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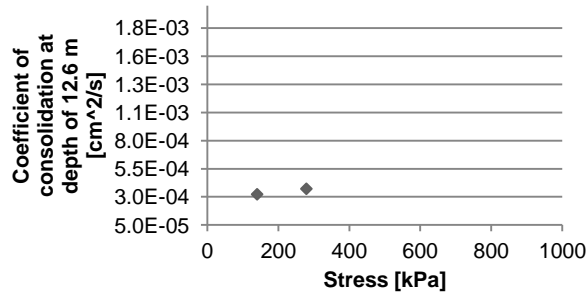
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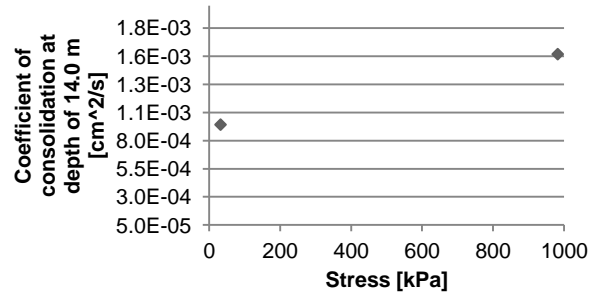
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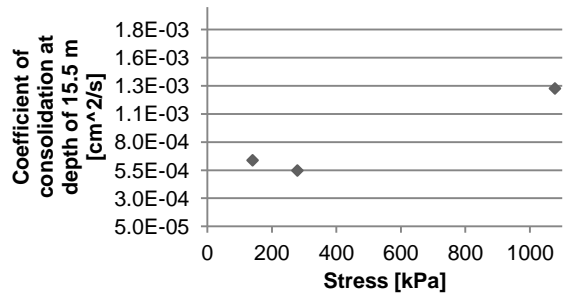
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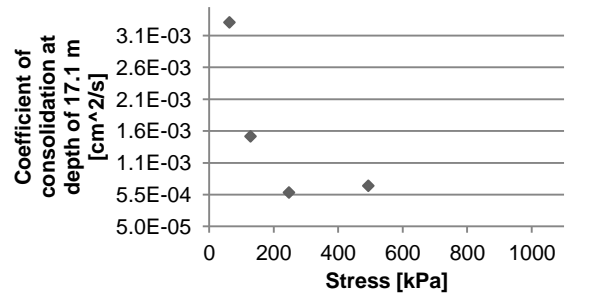
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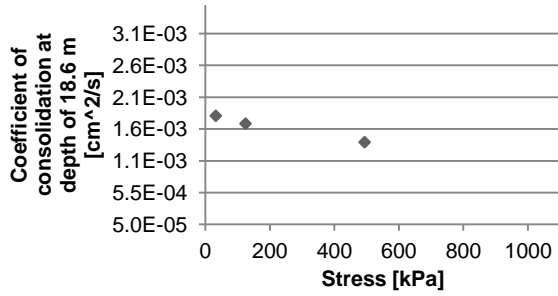
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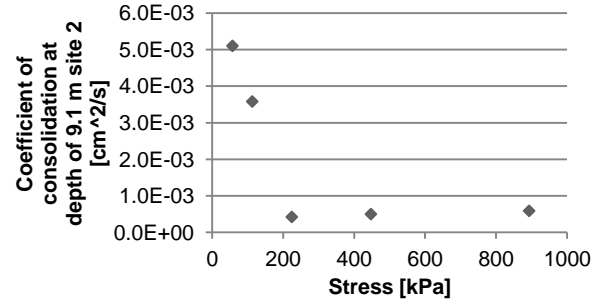
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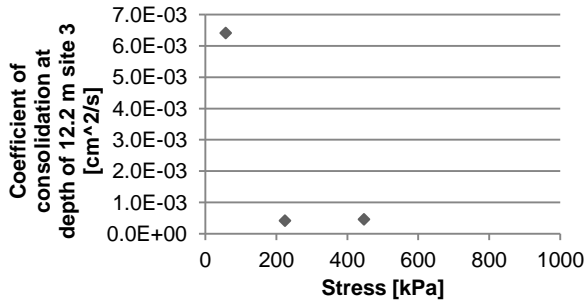
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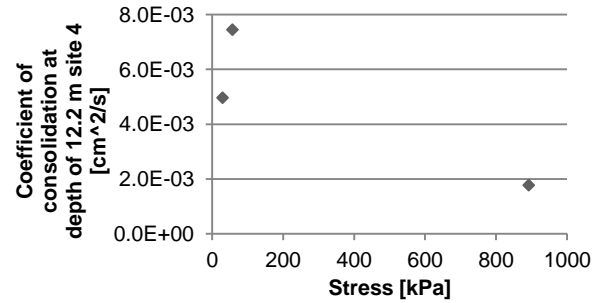
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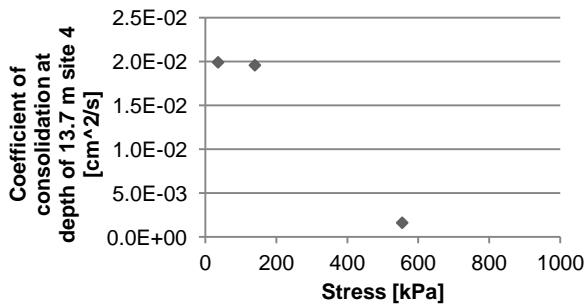
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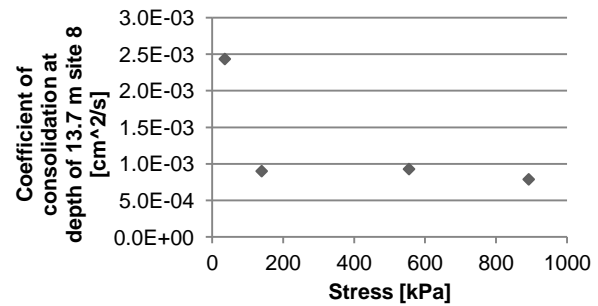
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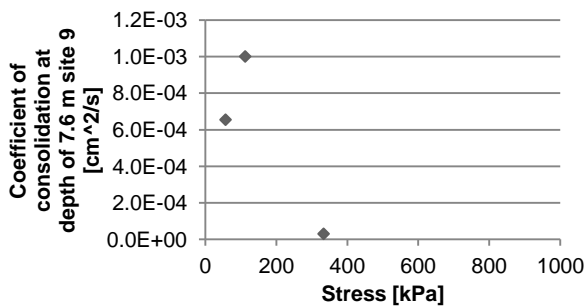
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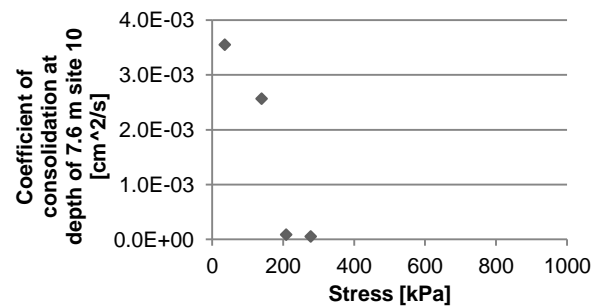
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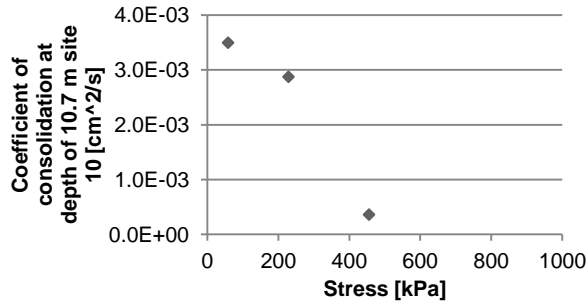
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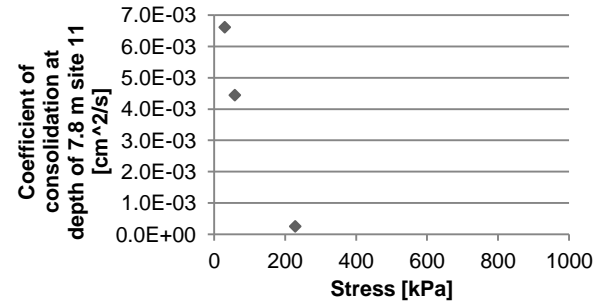
(o)



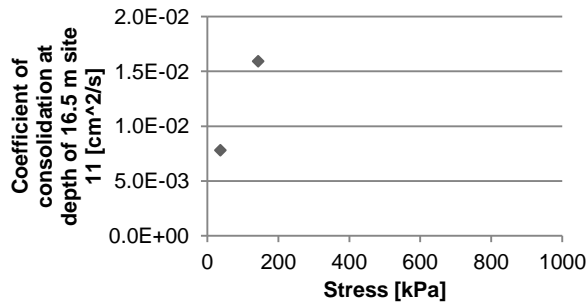
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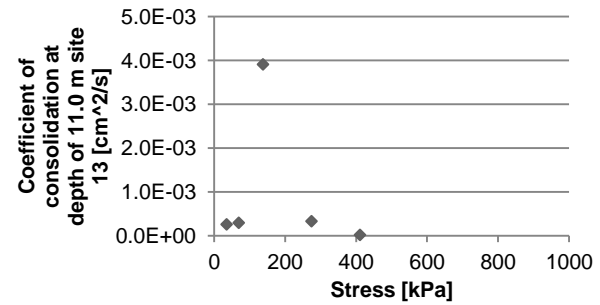
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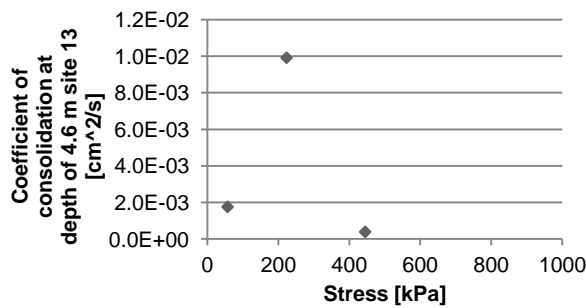
(r)



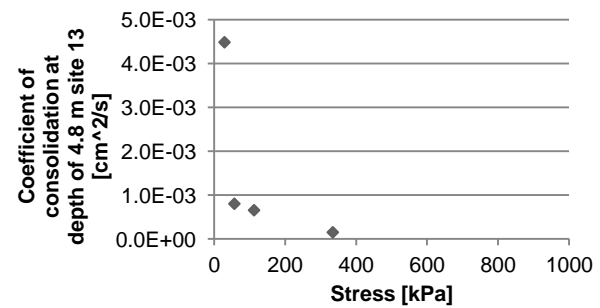
(s)



(t)



(u)



(v)

Figure 3-22 Coefficient of consolidation with stress from oedometer tests for a depth of (a) 2.6 m Site 6 (b) 4.1 m Site 6 (c) 6.4 m Site 6 (d) 8.7 m Site 6 (e) 12.6 m Site 6 (f) 14.0 m Site 6 (g) 15.5 m Site 6 (h) 17.1 m Site 6 (i) 18.6 m Site 6 (j) 9.1 m Site 2 (k) 12.2 m Site 3 (l) 12.2 m Site 4 (m) 13.7 m Site 4 (n) 13.7 m Site 8 (o) 7.6 m Site 9 (p) 7.6 m Site 10 (q) 10.7 m Site 10 (r) 7.8 m Site 11 (s) 16.5 m Site 11 (t) 11.0 m Site 13 (u) 4.6 m Site 13 (v) 4.8 m Site 13

Table 3-4 Summary of t_{90} and the coefficient of consolidation relative to the stress interval, depth, and preconsolidation pressure from oedometer tests

Site	Depth [m]	Stress [kPa]	t_{90}	Coefficient of consolidation [cm ² /sec]	Preconsolidation pressure [kPa]
2	9.1	57.1	2.6	5.10E-03	107
2	9.1	112.9	3.6	3.58E-03	107
2	9.1	224.3	30.3	4.23E-04	107
2	9.1	447.3	25.0	5.01E-04	107
2	9.1	893.2	18.5	5.90E-04	107
3	12.2	57.0	2.0	6.40E-03	115
3	12.2	224.1	29.2	4.17E-04	115
3	12.2	446.8	23.0	4.59E-04	115
4	12.2	29.2	2.6	4.96E-03	210
4	12.2	57.0	1.7	7.44E-03	210
4	12.2	892.1	6.3	1.77E-03	210
4	13.7	35.5	0.6	1.99E-02	247
4	13.7	139.3	0.6	1.96E-02	247
4	13.7	554.6	7.3	1.62E-03	247
6	2.6	32.0	14.4	9.70E-04	53
6	2.6	62.8	39.7	3.46E-04	53
6	2.6	93.5	98.0	1.36E-04	53
6	2.6	124.3	139.2	8.99E-05	53
6	2.6	247.3	22.1	4.99E-04	53
6	2.6	490.2	15.2	4.88E-04	53
6	2.6	982.3	6.3	1.12E-03	53
6	4.1	112.4	57.8	1.99E-04	58
6	4.1	223.3	29.2	3.15E-04	58
6	4.1	445.3	16.8	3.93E-04	58
6	6.4	62.8	13.7	9.51E-04	57
6	6.4	128.1	7.3	1.69E-03	57

6	6.4	247.3	12.3	9.32E-04	57
6	6.4	493.3	9.6	1.06E-03	57
6	8.7	112.4	32.5	3.42E-04	52.5
6	8.7	223.3	24.0	4.10E-04	52.5
6	8.7	445.3	13.7	5.79E-04	52.5
6	12.6	139.9	31.4	3.23E-04	52
6	12.6	278.8	24.0	3.72E-04	52
6	14.0	32.0	14.4	9.42E-04	61
6	14.0	982.3	5.8	1.57E-03	61
6	15.5	139.9	18.5	6.37E-04	71
6	15.5	278.8	19.4	5.46E-04	71
6	15.5	1077.3	5.8	1.27E-03	71
6	17.1	62.8	4.0	3.26E-03	81
6	17.1	128.1	8.4	1.46E-03	81
6	17.1	247.3	19.4	5.81E-04	81
6	17.1	493.3	14.4	6.87E-04	81
6	18.6	32.0	7.8	1.76E-03	90
6	18.6	124.3	7.8	1.63E-03	90
6	18.6	493.3	7.8	1.34E-03	90
8	4.6	892.1	13.7	7.88E-04	225
8	13.7	35.6	5.3	2.43E-03	110
8	13.7	139.5	13.7	9.01E-04	110
8	13.7	554.7	11.6	9.29E-04	110
9	7.6	56.6	19.4	6.54E-04	110
9	7.6	111.9	12.3	1.00E-03	110
9	7.6	333.0	342.3	2.99E-05	110
10	7.6	35.5	3.6	3.55E-03	115
10	7.6	139.3	4.8	2.56E-03	115
10	7.6	208.5	125.4	8.47E-05	115
10	7.6	277.7	174.2	5.15E-05	115
10	10.7	58.1	3.6	3.49E-03	115
10	10.7	228.4	4.0	2.87E-03	115
10	10.7	455.3	22.1	3.61E-04	115

11	7.8	29.8	2.0	6.61E-03	115
11	7.8	58.1	2.9	4.44E-03	115
11	7.8	228.4	47.6	2.57E-04	115
11	16.5	36.4	1.7	7.79E-03	189
11	16.5	142.9	0.8	1.59E-02	189
13	11.0	35.0	49.0	2.58E-04	40
13	11.0	69.2	41.0	2.95E-04	40
13	11.0	137.5	2.9	3.90E-03	40
13	11.0	274.2	29.2	3.30E-04	40
13	11.0	410.9	420.3	1.88E-05	40
13	4.6	56.7	7.3	1.76E-03	115
13	4.6	222.9	1.2	9.91E-03	115
13	4.6	444.4	23.0	3.90E-04	115
13	4.8	29.1	2.9	4.48E-03	113
13	4.8	56.9	16.0	8.03E-04	113
13	4.8	112.4	19.4	6.56E-04	113
13	4.8	334.3	51.8	1.53E-04	113

3.6.4 Secondary Compression Index

The secondary compression index was determined from the slope of the final straight line portion of the dial reading versus the log of time for one cycle of oedometer loading. Figure 3-23 shows the secondary compression index with depth for all of the sites. For Site 6, there is a trend of a general decrease with depth. The secondary compression index ranges between 0.15 and 0.45 for Site 6, and between 0.13 and 0.61 for all of the other sites.

Holtz and Kovacs (1981) mentioned that the ratio of the secondary compression index to the compression index ($C\alpha/Cc$) for Canadian marine clays range between 0.03 and 0.06. From this study, it was found that this ratio for sensitive marine clays in Ottawa ranges between 0.159 and 0.619. The ratio found in this study on average is at least 8 orders of magnitude higher than the average ratio reported by Holtz and Kovacs (1981). The reason is that the values proposed are

general values for all Canadian clays while the values suggested in this study are only for local clays in Ottawa. Tables 3-5 and 3-6 show this ratio for all of the sites.

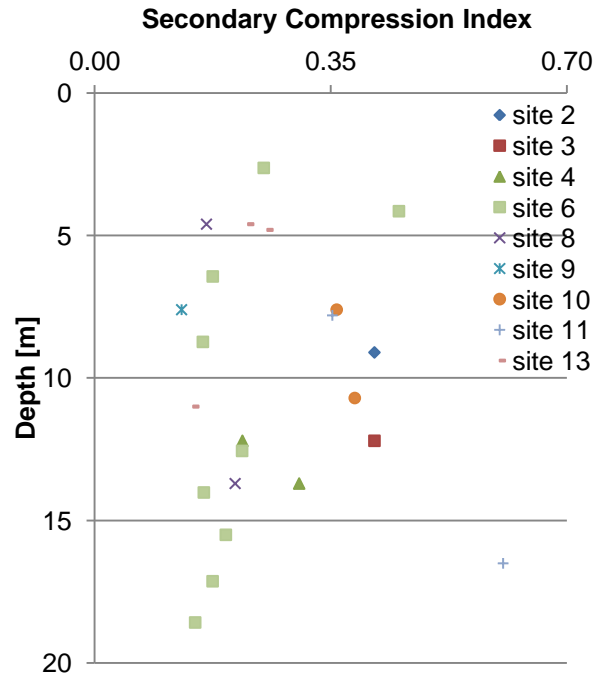


Figure 3-23 Secondary compression index with depth from oedometer tests

Table 3-5 Compression and secondary compression indexes from oedometer tests for Site 6

Depth [m]	Cc	Ca	Ca/Cc
2.626	1.08	0.25	0.232
4.145	2.01	0.45	0.224
6.434	0.53	0.17	0.330
8.73	1.01	0.16	0.159
12.5555	0.87	0.22	0.251
14.0135	0.44	0.16	0.368
15.499	0.76	0.19	0.256
17.128	0.61	0.17	0.286
18.575	0.66	0.15	0.226

Table 3-6 Compression and secondary compression index ratios from oedometer tests

Site	Depth [m]	Cc	Ca	Ca/Cc
2	9.1	0.74	0.41	0.560
3	12.2	0.67	0.41	0.619
4	12.2	0.5	0.22	0.438
4	13.7	0.91	0.30	0.333
8	4.6	1.01	0.17	0.164
8	13.7	0.73	0.21	0.285
9	7.6	0.74	0.13	0.174
10	7.6	2.24	0.36	0.160
10	10.7	1.48	0.39	0.260
11	7.8	1.53	0.35	0.230
11	16.5	2.12	0.61	0.286
13	11	0.62	0.15	0.235
13	4.6	1.04	0.23	0.219
13	4.8	1.84	0.26	0.139

As shown earlier, sensitive marine clays in Ottawa have relatively high secondary compression indexes. According to ASTM D2435, each loading cycle in an oedometer test may take up to 24 hours. It is observed that the secondary compression index does not change much if the loading cycle is extended to more than 24 hours. Figure 3-24 shows an example of the dial reading versus log of time for a loading cycle that was extended to approximately three days.

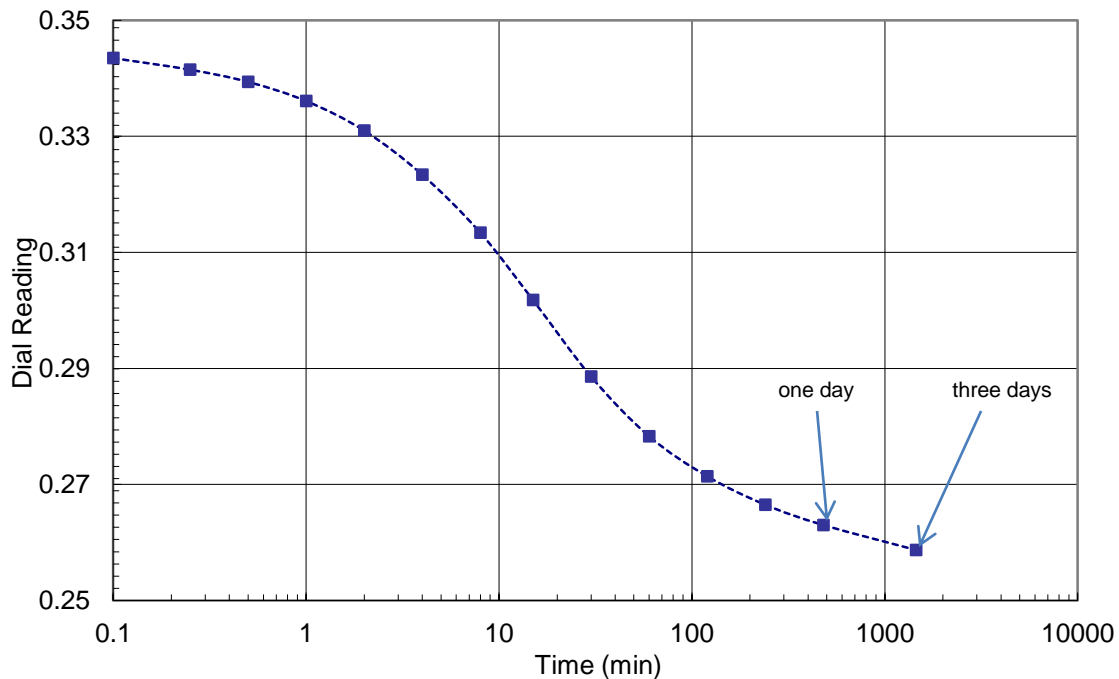


Figure 3-23 Example of dial reading with time from oedometer tests for samples from Ottawa

3.7 Sensitivity of Marine Clay in Ottawa

According to the Canadian Engineering Foundation Manual (2006), sensitivity that is less than 2 is considered as low sensitivity, 2 to 4 as medium sensitivity, 4 to 8 as high sensitivity, and higher than 16 as quick clay. Figure 3-25 shows the sensitivity with depth for all of the sites under study. The blue line separates the low and medium sensitivity regions. Likewise, the green, black, and red lines separate medium and high sensitivity, high sensitivity to high-quick, and high-quick to quick regions respectively. In general, sensitive marine clays in Ottawa occupy all of the sensitivity regions. Most sites (8 sites) occupy 3 sensitivity regions, 4 sites occupy 4 sensitivity regions, and only 1 site occupies 5 sensitivity regions. All of the sites are extended to beyond the high sensitivity limit while Site 1 is extended from medium to significantly above the quick limit.

Yong et al. (1979) studied the effect of amorphous material on the sensitivity of Champlain clays in St. Alban and Gatineau. Even though the study was performed on clays outside the Ottawa area, the clay is still a Champlain Sea clay and shares similar characteristics with sensitive marine clays in Ottawa. Amorphous material may hold water in its pores. Amorphous material is

held by quartz grains through electrostatic attraction bonds when mixed with artificially synthesized silicon-iron hydroxides (Rousseaux and Warkentin 1976). Another study also showed that amorphous material is mobile during shearing (McKeyes et al. 1974). Therefore, bonds are likely the reason for high sensitivity in Champlain Sea clays.

Chemical reactions that affect the Gouy double layer in the clay particles reduce the remolded strength of the soil and increase the sensitivity (Quigley 1980). It has been determined that there is a correlation between Ottawa clay sensitivity and the electrokinetic potential (Penner 1965). Clay minerals affect the sensitivity of clay soils. However, as the clay portion in the soil decreases, the soil sensitivity becomes rather controlled by the cementation bonds (Moum et al. 1971). Also, higher adsorption of Na and K than Ca and Mg are associated with high sensitivity. The exchange of Ca, Mg, K with Na can reduce the sensitivity of the clay (Moum et al. 1971). Amorphous material affects the initial and remolded shear strengths of the soil, and therefore, the sensitivity (Yong and Sethi 1979). Sensitivity is shown to have a direct correlation with carbonate content and inverse correlation with salinity (Haynes and Quigley 1978; Quigley and Bohdanowicz 1979). However, there does not seem to be a correlation between sensitivity and amorphous material. The salinity of marine clays is between 2 and 4 g/L at the surface layer and 15 g/L at deeper layers with higher values of Na^+ than Ca^{2+} , Mg^{2+} , and K. These salinity values are considered to be high; therefore the effects of organic dispersants and bicarbonates are neutralized. In summary, Quigley (1980) and Taha (2010) indicated that high sensitivity depends on high electrokinetic potential, sediment concentration, divalent cation adsorption, slow rate of overburden pressure increase, cementation bonds, high water content or higher than the liquid limit, and low specific surface of soil particles, amorphous content and smectite content.

The sensitivity value reported by Taha (2010) for a depth of 8 to 12 m is 6 from laboratory testing. The sensitivity is affected by the salinity concentration. Torrance (1975) found that increases in the salinity decreases the sensitivity and increases the undrained shear strength of marine clays. He explained that this behavior is attributed to negative surfaces and positive charged edges of clay particles. A higher ion concentration from salinity reduces the effect of repulsion between the edges. This also makes it easier for the formation of bonds between positively charged edges and the negatively charged surface of clay particles. Taha and Fall

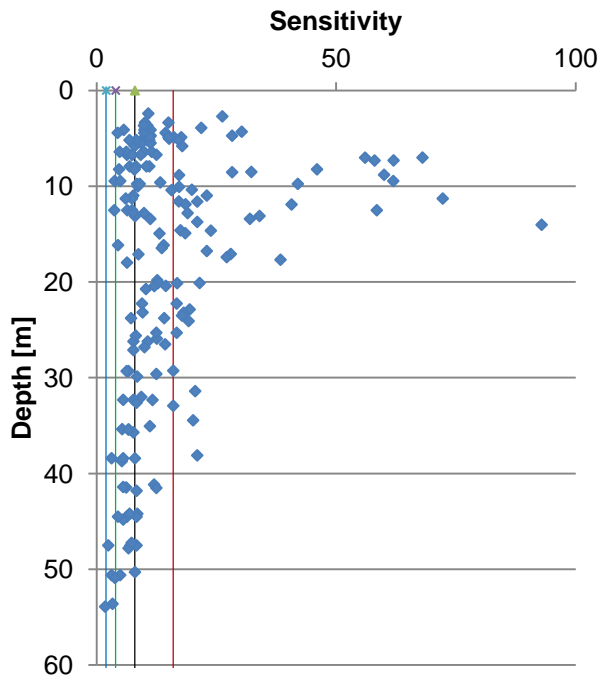
(2010) found that when the salinity was increased from 10 to 20 g/l, the sensitivity decreased by 50%.

Penner (1965) studied the sensitivity of sensitive marine clays in Ottawa. He observed an increase in the sensitivity as a result of the addition of sodium metaphosphate. He attributed this to an increase in the repulsive forces between the clay particles and expected an increase in the electrokinetic potential. He found that by washing a remolded sample, this can increase the electrokinetic potential from 19.7 to 23.2 mV and the sensitivity from 34 to 91. On the other hand, after washing and adding sodium metaphosphate to a remolded sample, this can increase the electrokinetic potential from 19.7 to 64.6 mV and the sensitivity from 34 to 2150.

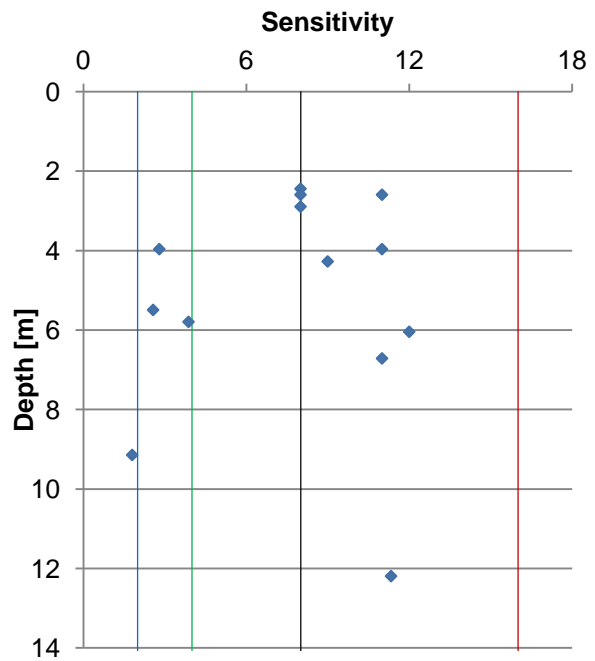
Penner (1965) explained that the increase in sensitivity is due to the remolding which breaks the van der Waal forces between the clay particles, and the adsorbed sodium metaphosphate increases the repulsion forces between the particles. As a result, the porewater redistributes itself by forming a separating layer between the particles. When the soil is left for a few days, the sodium metaphosphate tends to settle and water collects on the surface of the samples even though the samples have insitu moisture content. The soil structure becomes denser and the particles form in a face to face rather than cardhouse arrangement. At low salt concentrations, the sensitivity is highly variable and can be up to 1000, in pore water with a specific conductivity of less than $4 \text{ ohm}^{-1} \text{ cm}^{-1}$. However, at higher specific conductivity, the sensitivity does not exceed 75 as a result of the high flocculation. Penner (1965) also established that the sensitivity consistently increases with electrokinetic potential. From his published results, it can be seen that in general, there is an increase in sensitivity with a decrease in the clay percentage and the surface area. There is also an increase in the sensitivity as the concentrations of Na^+ and K^+ increase and concentrations of Ca^{2+} and Mg^{2+} decrease (Penner 1965).

Dejong et al. (2011) and Yafrate et al. (2009) reported sensitivity values of 33.3 and 68 for depths of 4.4 and 8.3 m respectively at Site 6. Lo et al. (1976) divided Site 6 into three principle layers of 6, 6, and 5 m. They assigned an average sensitivity value for each layer in the range of 20 to 100, 50 to 100, and 20 to 50, respectively. According to Eden and Crawford (1957), the sensitivity of the Ottawa area is between 20 and 60 which was obtained from five profiles. Mitchell (1970) reported high sensitivity values from block samples at Site 6 at 100, 500, and 500 for depths of 13.7, 16.1, and 19.0 m respectively. The block sample obtained from sensitive

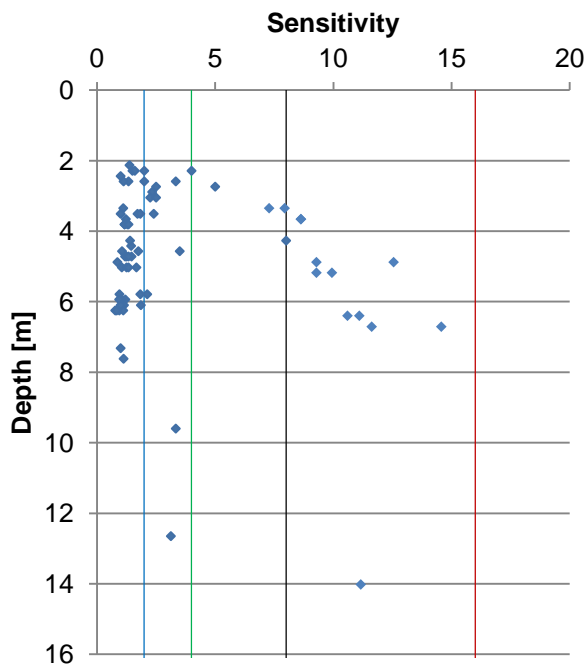
marine clay in Ottawa during a sewer installation by Quigley and Thompson (1966) had a sensitivity between 10 and 12.



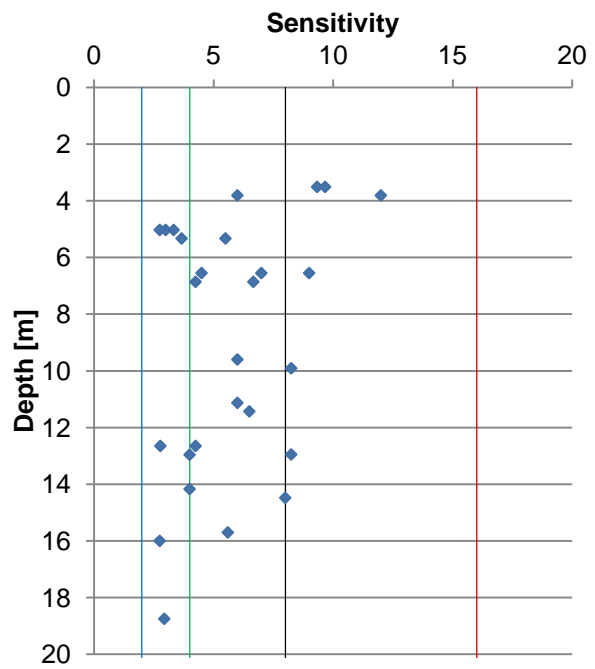
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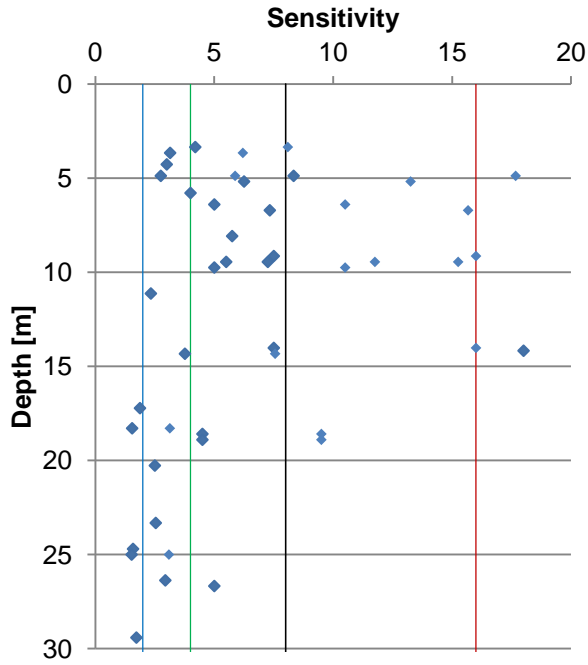
(b)



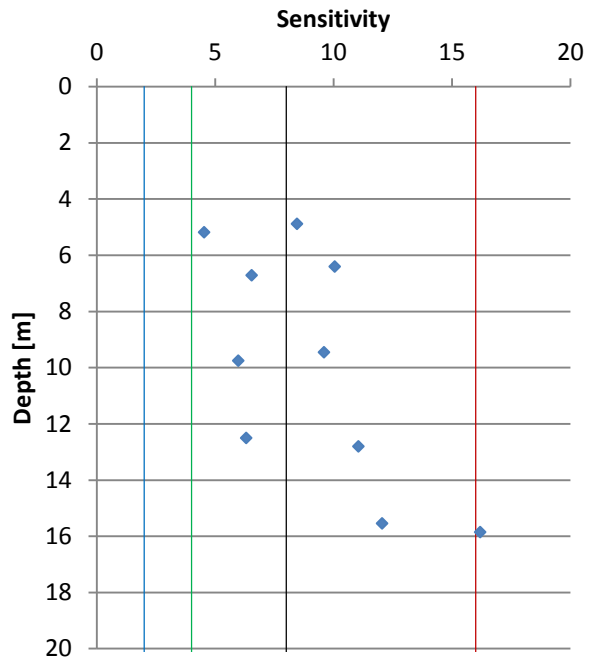
(c)



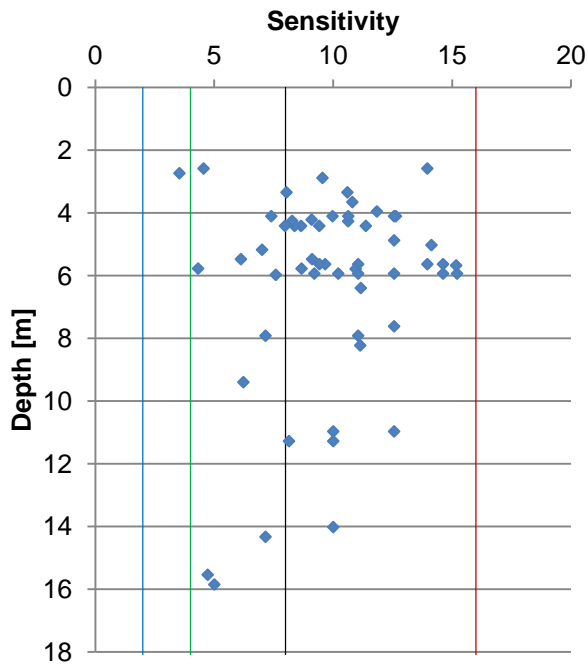
(d)



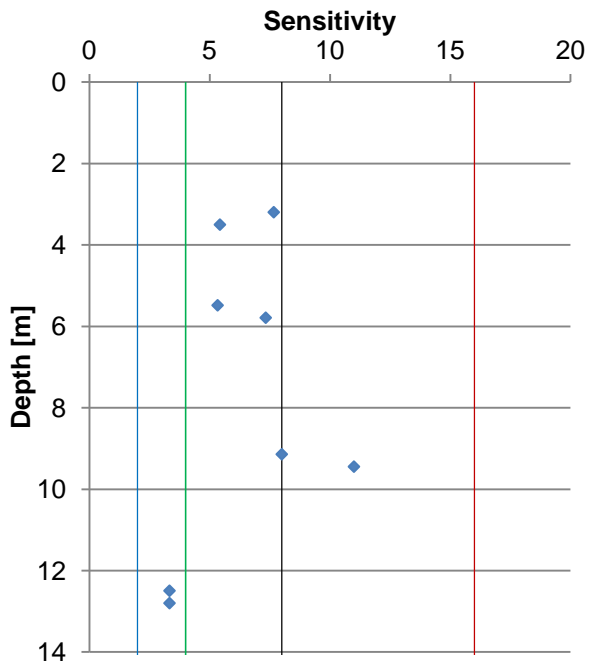
(e)



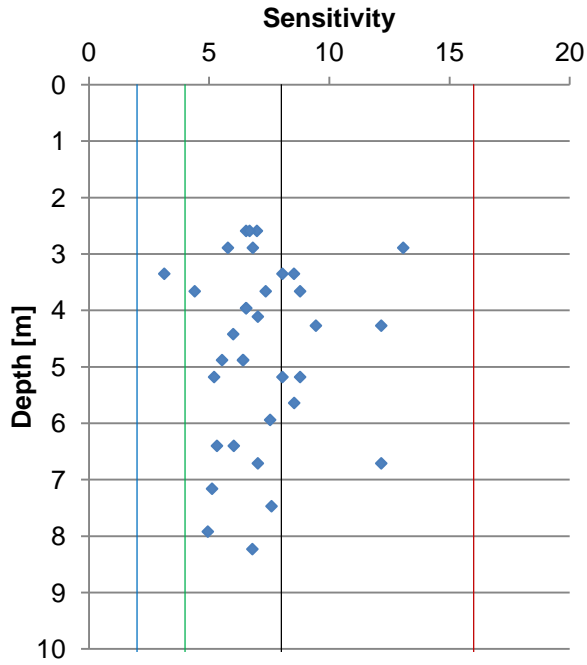
(f)



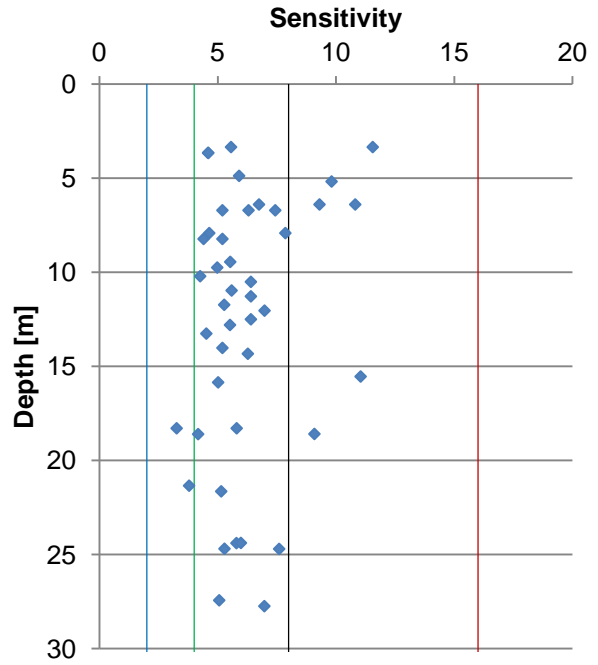
(g)



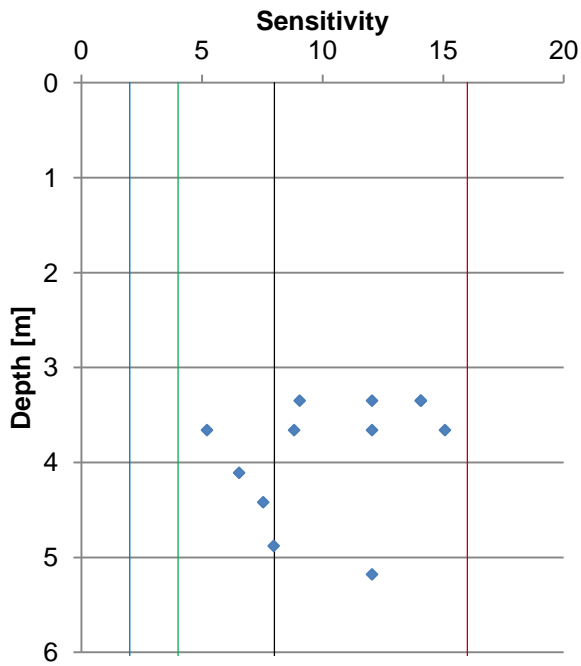
(h)



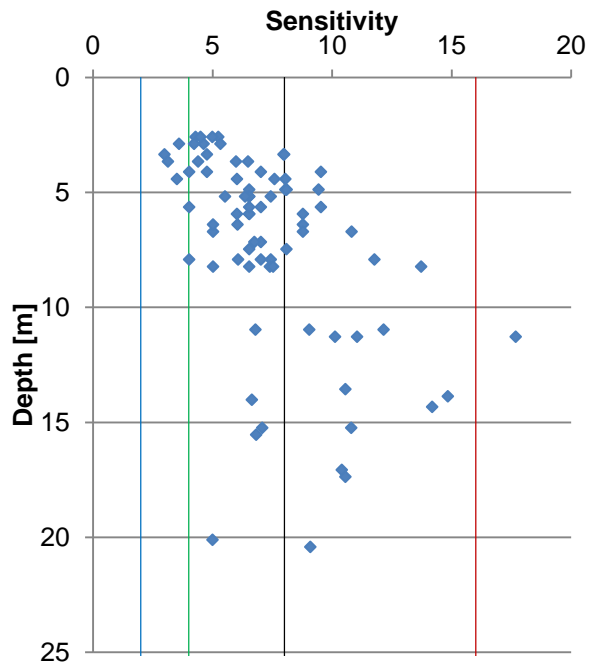
(i)



(j)



(k)



(l)

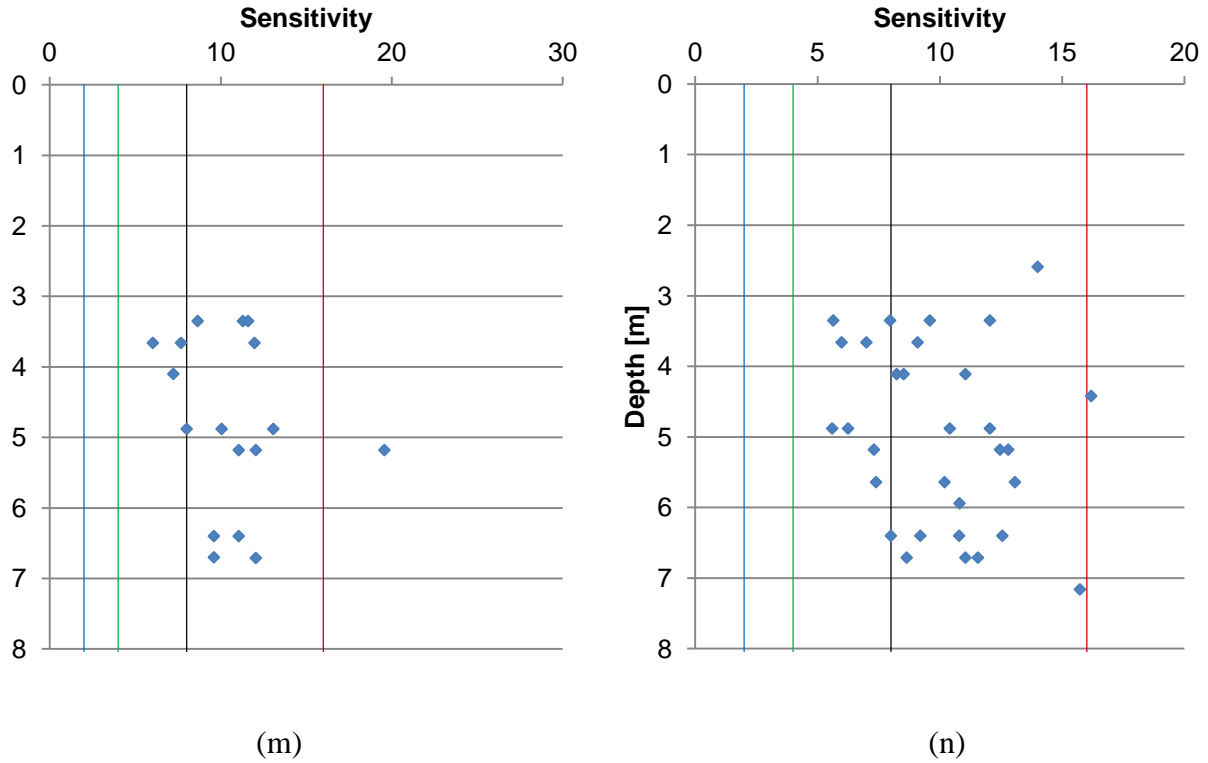


Figure 3-24 Sensitivity with depth for (a) Site 1 (b) Site 2 (c) Site 3 (d) Site 4 (e) Site 5 (f) Site 7 (g) Site 8 (h) Site 9 (i) Site 10 (j) Site 11 (k) Site 12 (l) Site 13 (m) Site 14 (n) Site 15

3.8 Hydraulic Conductivity

The coefficient of permeability is a key geotechnical parameter of soil and significantly influences or controls several geotechnical engineering problems or mechanisms, such as the groundwater regime in stratified deposits or near natural and excavated slopes, consolidation of clay foundations, migration of pollutants from waste disposal facilities, and flow of water through or around engineered structures (Tavenas et al. 1983). Hence, the coefficients of the permeability of the sensitive marine clays in the sites studied have been assessed.

The coefficient of permeability was determined from one dimensional consolidation tests in accordance with ASTM D 2435 by using the following equation (Holtz and Kovacs 1981):

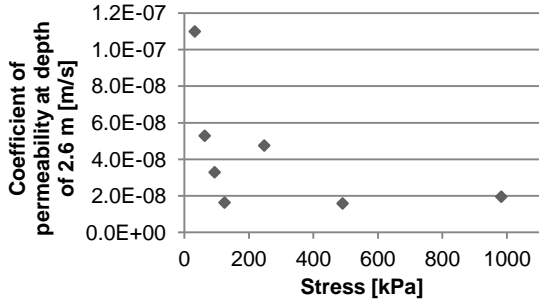
$$k = \frac{c_v \rho_w g}{1 + e_1} * \frac{e_1 - e_2}{\sigma_2 - \sigma_1}$$

where k is the coefficient of permeability, C_v is the coefficient of consolidation determined from oedometer testing, ρ_w is the density of water, e_1 is the void ratio at the start of the loading stage,

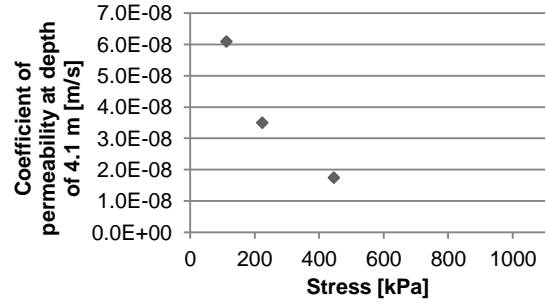
e_2 is the void ratio at the end of the loading stage, and $\sigma_2' - \sigma_1'$ is the difference in the loading between the previous and current loading stages.

Figure 3-26 shows the variation of the coefficient of permeability with stress for Site 6 and all of the other sites. The coefficient of permeability generally decreases with increases in the stress. However, it can increase then decrease with increases with stress, and the reason is the same as that of the coefficient of consolidation behaviour with stress. As the stress increases, the soil structure and the porewater take the load and resist compression. At this point, there is high hydraulic conductivity as the pores are maintained at the same volume. Eventually, the structure breaks and causes the rearrangement of the soil structure and reduces the hydraulic conductivity. The coefficient of permeability ranges between 1.58×10^{-8} and 3.5×10^{-7} cm/s for Site 6 and between 5.52×10^{-10} and 1.09×10^{-6} cm/s for all of the other sites. Table 3-7 is a summary of all the coefficients of consolidation with depth, load increment, and preconsolidation pressure of the test samples.

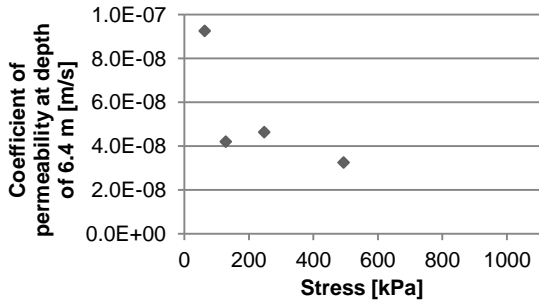
A profile of the hydraulic conductivity was supplied by the National Research Council Canada and reported by Hinchberger and Rowe (1998) for the first stage of the embankment of Site 6. The profile is graphed as shown in Figure 3-27. The hydraulic conductivity ranges between 0.6×10^9 and 2.7×10^9 m/s at a depth of 2.13 to 20.73 m. The hydraulic conductivity does not follow a specific pattern with depth. However, it is higher at the desiccated surface layer and after a depth of 20 m as there is a layer of coarser silty clay or sandy clay as reported by Yafrate and Dejong (2005).



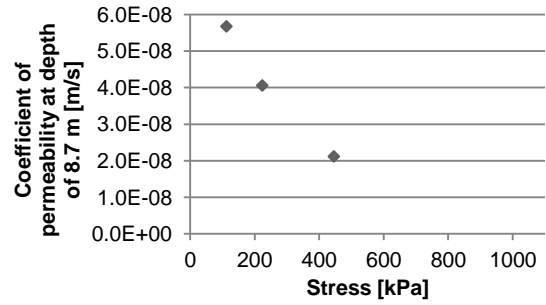
(a)



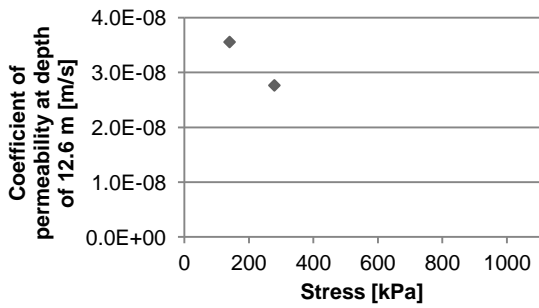
(b)



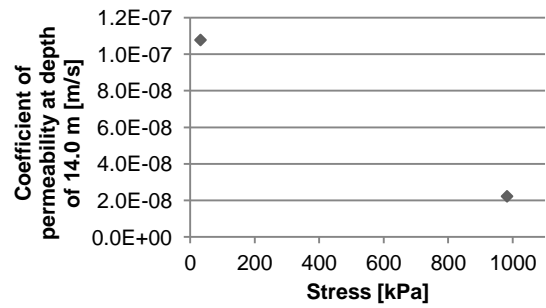
(c)



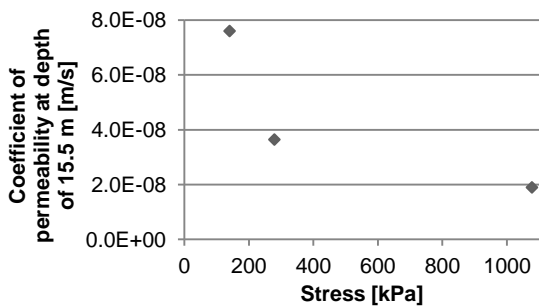
(d)



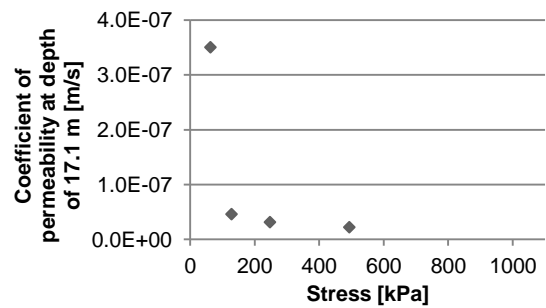
(e)



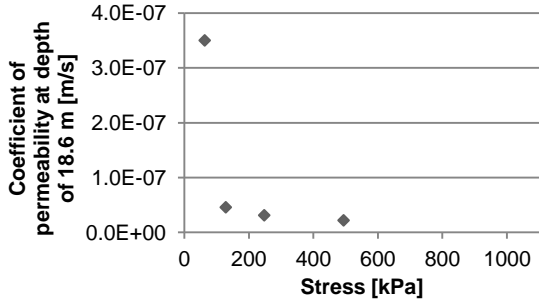
(f)



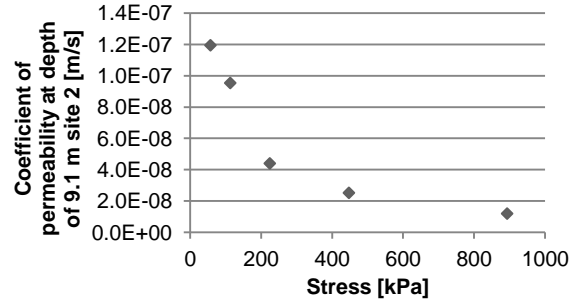
(g)



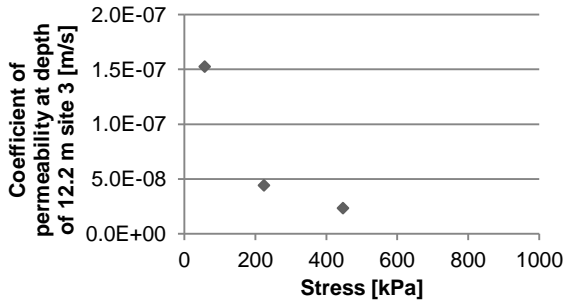
(h)



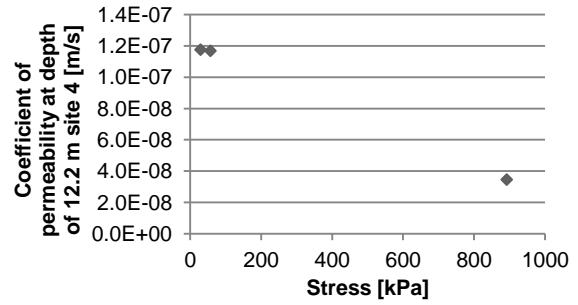
(i)



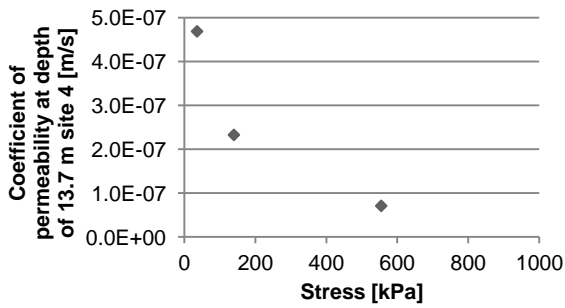
(j)



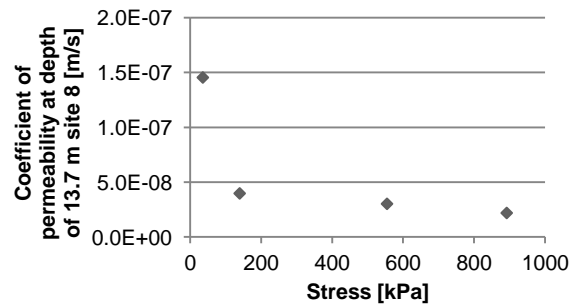
(k)



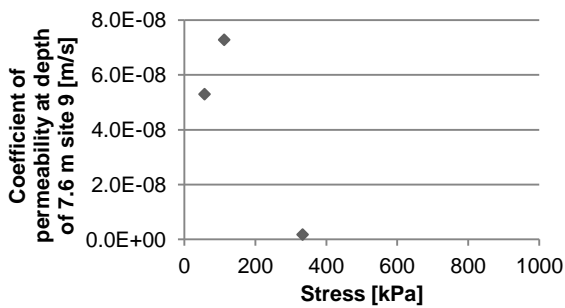
(l)



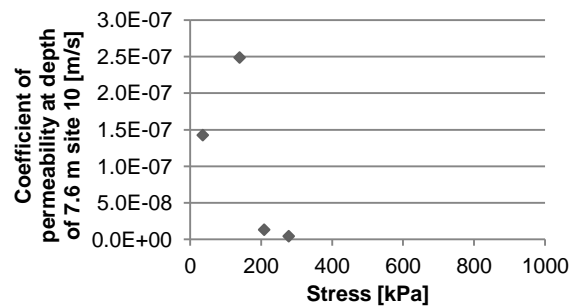
(m)



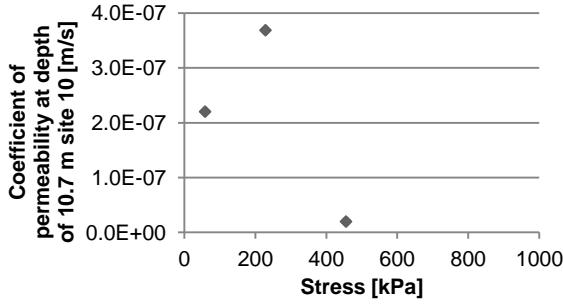
(n)



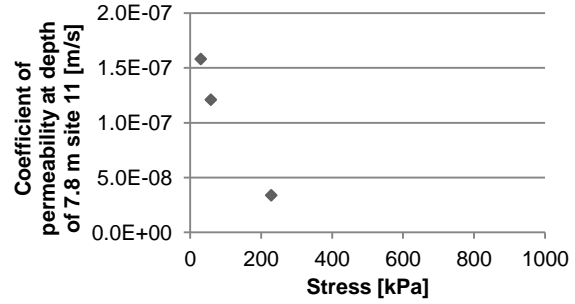
(o)



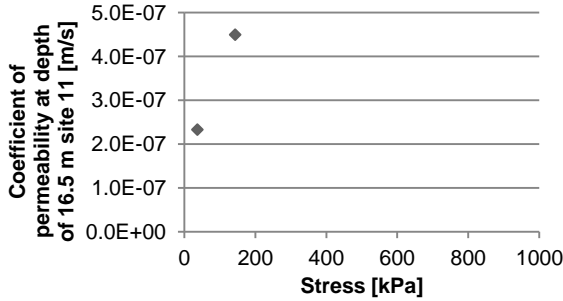
(p)



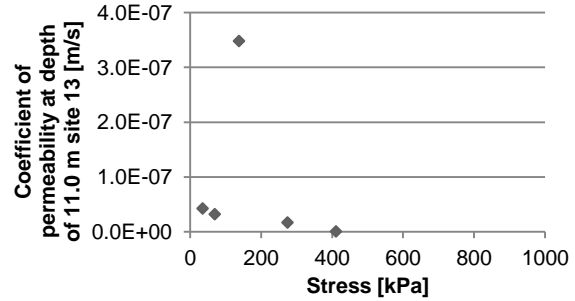
(q)



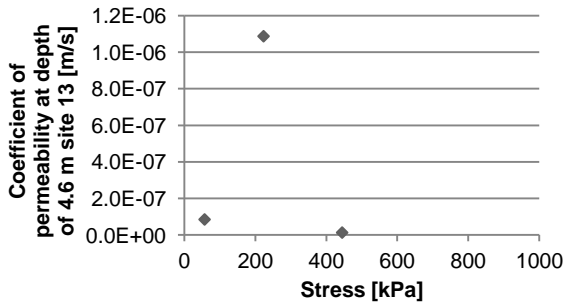
(r)



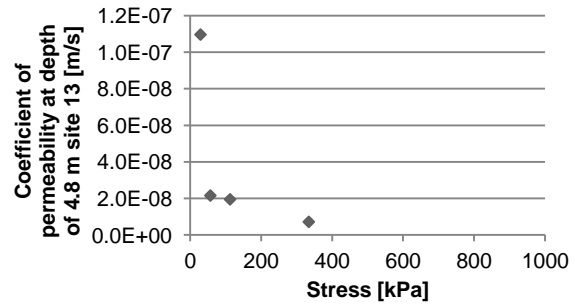
(s)



(t)



(u)



(v)

Figure 3-25 Coefficient of permeability with stress from oedometer tests for a depth of (a) 2.6 m Site 6 (b) 4.1 m Site 6 (c) 6.4 m Site 6 (d) 8.7 m Site 6 (e) 12.6 m Site 6 (f) 14.0 m Site 6 (g) 15.5 m Site 6 (h) 17.1 m Site 6 (i) 18.6 m Site 6 (j) 9.1 m Site 2 (k) 12.2 m Site 3 (l) 12.2 m Site 4 (m) 13.7 m Site 4 (n) 13.7 m Site 8 (o) 7.6 m Site 9 (p) 7.6 m Site 10 (q) 10.7 m Site 10 (r) 7.8 m Site 11 (s) 16.5 m Site 11 (t) 11.0 m Site 13 (u) 4.6 m Site 13 (v) 4.8 m Site 13

Table 3-7 Summary of the coefficient of permeability relative to the stress interval, depth, and preconsolidation pressure from oedometer tests

Site	Depth [m]	Stress [kPa]	Coefficient of permeability [m/sec]	Preconsolidation pressure [kPa]
2	9.1	57.1	1.19E-07	107
2	9.1	112.9	9.54E-08	107
2	9.1	224.3	4.40E-08	107
2	9.1	447.3	2.52E-08	107
2	9.1	893.2	1.20E-08	107
3	12.2	57.0	1.52E-07	115
3	12.2	224.1	4.41E-08	115
3	12.2	446.8	2.33E-08	115
4	12.2	29.2	1.17E-07	210
4	12.2	57.0	1.17E-07	210
4	12.2	892.1	3.45E-08	210
4	13.7	35.5	4.69E-07	247
4	13.7	139.3	2.33E-07	247
4	13.7	554.6	7.10E-08	247
6	2.6	32.0	1.10E-07	53
6	2.6	62.8	5.29E-08	53
6	2.6	93.5	3.29E-08	53
6	2.6	124.3	1.63E-08	53
6	2.6	247.3	4.75E-08	53
6	2.6	490.2	1.58E-08	53
6	2.6	982.3	1.95E-08	53
6	4.1	112.4	6.09E-08	58
6	4.1	223.3	3.50E-08	58
6	4.1	445.3	1.75E-08	58
6	6.4	62.8	9.24E-08	57
6	6.4	128.1	4.20E-08	57
6	6.4	247.3	4.63E-08	57
6	6.4	493.3	3.25E-08	57

6	8.7	112.4	5.67E-08	52.5
6	8.7	223.3	4.05E-08	52.5
6	8.7	445.3	2.11E-08	52.5
6	12.6	139.9	3.55E-08	52
6	12.6	278.8	2.76E-08	52
6	14.0	32.0	1.08E-07	61
6	14.0	982.3	2.22E-08	61
6	15.5	139.9	7.59E-08	71
6	15.5	278.8	3.64E-08	71
6	15.5	1077.3	1.89E-08	71
6	17.1	62.8	3.50E-07	81
6	17.1	128.1	4.59E-08	81
6	17.1	247.3	3.13E-08	81
6	17.1	493.3	2.20E-08	81
6	18.6	32.0	2.05E-07	90
6	18.6	124.3	1.28E-07	90
6	18.6	493.3	4.53E-08	90
8	4.6	892.1	2.19E-08	225
8	13.7	35.6	1.45E-07	110
8	13.7	139.5	3.97E-08	110
8	13.7	554.7	3.02E-08	110
9	7.6	56.6	5.29E-08	110
9	7.6	111.9	7.27E-08	110
9	7.6	333.0	1.72E-09	110
10	7.6	35.5	1.42E-07	115
10	7.6	139.3	2.49E-07	115
10	7.6	208.5	1.31E-08	115
10	7.6	277.7	4.24E-09	115
10	10.7	58.1	2.20E-07	115
10	10.7	228.4	3.69E-07	115
10	10.7	455.3	1.98E-08	115
11	7.8	29.8	1.58E-07	115
11	7.8	58.1	1.21E-07	115

11	7.8	228.4	3.39E-08	115
11	16.5	36.4	2.33E-07	189
11	16.5	142.9	4.49E-07	189
13	11.0	35.0	4.24E-08	40
13	11.0	69.2	3.20E-08	40
13	11.0	137.5	3.48E-07	40
13	11.0	274.2	1.68E-08	40
13	11.0	410.9	5.52E-10	40
13	4.6	56.7	8.50E-08	115
13	4.6	222.9	1.09E-06	115
13	4.6	444.4	1.32E-08	115
13	4.8	29.1	1.10E-07	113
13	4.8	56.9	2.16E-08	113
13	4.8	112.4	1.95E-08	113
13	4.8	334.3	7.19E-09	113

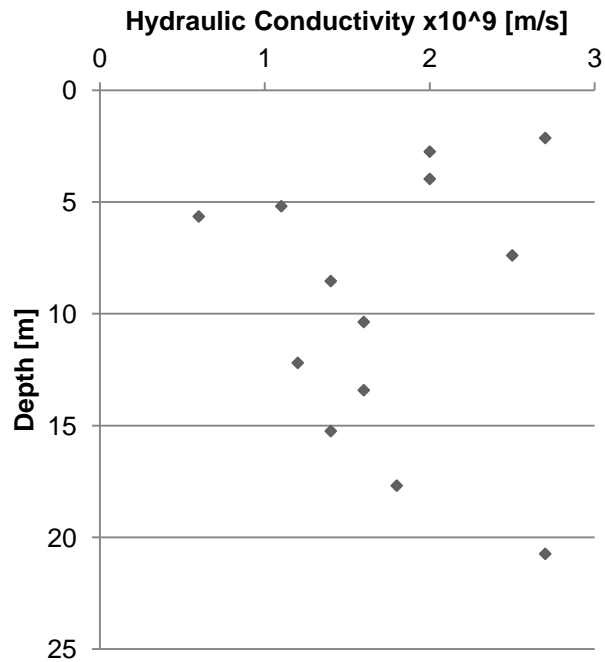


Figure 3-26 Hydraulic conductivity with depth for Site 6 supplied by the National Research Council Canada and reported by Hinchberger and Rowe (1998)

3.9 Summary and Conclusion

A compilation and review of the engineering geotechnical properties of sensitive marine clays in Ottawa have been made. Fifteen site locations well spread out in the urban areas of Ottawa have been studied to characterize sensitive marine clays. Location maps and site descriptions, including stratifications, geodetic elevations, and water table depths, are provided. Engineering characteristics such as index properties, grain size distribution, activity, undrained and remolded shear strengths, sensitivity, shear wave velocity, preconsolidation pressure, overconsolidation ratio, compression and recompression indexes, coefficient of consolidation, secondary compression index, and hydraulic conductivity are studied. The work and results of previous researchers have been discussed and the value ranges for the said engineering properties have been suggested. Also, a review and summary on the origin and geological settings, porewater chemistry (Cl, Su, pH, and salinity), clay structure, laboratory shear strength and deformation, and field and interface shear strengths and behaviour are presented. The characterization of marine clays in Ottawa with "sensitivity" is supported by the fact that all of the sites under study extend to beyond the high sensitivity limit or the quick limit.

It is suggested that the Canadian Foundation Engineering Manual has underestimated the shear strength of sensitive marine clays in Ottawa. The underestimation is presented as the correlation between the N_{60} from the SPTs and the undrained shear strength. The underestimation is also present in the correlation between site classification and the undrained shear strength. The data sets available are not conclusive as they are limited in number. Therefore, future research should consider this issue.

Due to the high sensitivity of marine clays in Ottawa, issues are present in the determination of the remolded shear strength from field and Nilcon vanes. The rate of rotation is slow for sensitive marine clays in Ottawa in that the soil is capable of gaining strength during testing on the vane tip or as rod friction. Future research should consider the studying of the rotation rate of field vanes in sensitive marine clays in Ottawa. In addition, laboratory vane results are compared to field vane results for the same site. The former shows misestimation, even after correcting for the rate of rotation or overburden pressure. Future research should consider the adjusting of the rate of vane rotation so it is as similar as possible with the field vane rotation rate.

It has been observed from this research that a double loading scheme for consolidation testing increases the accuracy of the determination of the coefficient of consolidation. A double loading scheme renders the curve closer to the theoretical curves expected by interpretation methods. It is also observed that pore pressure dissipation tests may render an exaggerated coefficient of consolidation compared to the values obtained from oedometer testing.

The suggested range for the ratio of the secondary compression index to the compression index for sensitive marine clays in Ottawa has been underestimated by previous researchers. It is found from this research that the ratio is higher than the suggested range. A ratio of the recompression index to the compression index has also been suggested in this research.

It is found that the moisture contents of sensitive marine clays in Ottawa are close to or exceeds the liquid limits which may contribute to the sensitive behaviour of sensitive marine clays in Ottawa. These clays fall between the A-line and U-line in the plasticity chart which means they are somewhere in between lean and fat clays. The activities of these clays fall in general in the normal and inactive areas of the activity chart. However, one site can have a variety of soil activities that can fall in all of the active, normal, and inactive zones.

It is also found that there is a wide range of soil cone specific factors for sensitive marine clays in Ottawa. These factors are affected by the distance from the cone and the vane tests. Therefore, future research should be conducted to verify the ideal distance or determine the limits on the distance from the penetrometer location and the reference vane location. Also, these clays exhibit low penetration resistance and sleeve frictions. For this reason, the penetrometers used on sensitive marine clays in Ottawa are prone to shifts in readings due to temperature effects.

Acknowledgement

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Chapter 4: Technical Paper II - Characterization of Sensitive Marine Clays by Using Cone and Ball Penetrometers – Examples of Clays in Eastern Canada

N. Athir, M. Fall, R. Hache

Abstract

Soundings made by using the cone penetration test in sensitive marine clay in Ottawa (Canada) have been problematic. Negative readings have been recorded. The incorrect estimation of the engineering characteristics of Ottawa clays has been noted. The effect of temperature on the output of penetrometer load cell is suspected as the reason for the negative drift in soundings and erroneous estimations of engineering characteristics. Also, focused research is needed to verify existing correlations and empirical factors between the cone penetration test and engineering characteristics of Ottawa clays. These are the reasons for carrying out this research work. Temperature effects have become a significant factor in countries with radical temperature changes, such as Canada. Three types of penetrometer tips have been used at the Canadian Geotechnical Research Site No. 1. The tips used are a 36 mm cone, and 40 mm and 113 mm balls. Nilcon field vane testing has also been carried out in the site to determine the undrained shear strength and sensitivity (the ratio of the initial undrained shear strength to the remolded undrained shear strength) from the penetrometer soundings. Undisturbed samples are collected to obtain the engineering characteristics by laboratory testing. Laboratory testing on penetrometer equipment is performed to verify the effects of temperature on the soundings. It is found that temperature changes affect the penetrometer soundings. Temperature effects can cause drifting as indicated by negative tip stress or sleeve friction in soft or low resistance clays. Therefore, temperature effects may increase the errors in the estimation of the engineering characteristics of Ottawa clays. It has also been found that correlations may or may not apply well to Ottawa sensitive marine clay. Temperature corrections could be applied to the penetrometer soundings. From this research, correlations and empirical factors between penetrometers and engineering characteristics are verified for Ottawa sensitive marine clays.

Keywords

Ball penetrometers, CPT, Temperature, Gloucester, Ottawa, Leda Clay, Sensitive Marine Clay

4.1 Introduction

Various types of penetrometers have been increasingly used to estimate the engineering design parameters of soils. The different possibilities of measuring profiles have given penetrometers advantages over other conventional means of field testing, such as standard penetration and vane shear testing. Different penetrometer sizes and tip shapes have been used for the purpose of improving the estimations of engineering soil characteristics. Cone, ball, and T-bar penetrometers are the most commonly used. Cone tips are considered the conventional penetrometer tip shape. The literature is rich with work carried out on the correlations between cone penetration tests (CPTs) and soil properties in geotechnical engineering, such as shear strength, sensitivity, grain size, and consolidation history (Battaglio et al. 1981; Lunne and Kleven 1981; Rad and Lunne 1986 and 1988; La Rochelle et al. 1988; Marsland and Powell 1988; Powell and Quarterman 1988; Robertson and Campanella 1988; Suzuki et al. 1995; Karlsrud et al. 1996; Lunne et al. 1997; Low et al. 2010). However, although these correlations have been verified with a variety of soils, not all types of soils have been investigated. These correlations have not been specifically verified with sensitive marine clays in Ottawa. The verification and modification of these correlations with local experience (Lunne et al. 1997) are recommended. Ball and T-bar penetrometers (full-flow penetrometers) were initially used for the centrifugation of samples (Stewart and Randolph 2001). Later, the use of full-flow penetrometers was extended to estimating the undrained and remolded shear strength and sensitivity of clays due to the remolding effects from the plastic behaviour of soil around the tip and probe. Furthermore, as full-flow penetrometers have larger projected areas of the tip in comparison to cone penetrometers, they have been used in extremely soft soils. Full-flow penetrometers are less influenced by overburden stress which makes them more accurate in the estimation of undrained shear strength than cone penetrometers (DeJong et al. 2010). However, our understanding of the use of ball penetrometers for the purpose of characterizing the geotechnical properties of sensitive marine clays, particularly those in Ottawa, is still limited. Research is needed to address this issue.

Temperature effects are expected to be the cause of the erroneous estimations of engineering characteristics from penetrometer tests. Negative tip resistance and sleeve friction have been found by Boylan and Long (2007). Radical temperature changes may be experienced by penetrometer equipment. Temperature records from The Weather Network have shown that the average high and low temperatures in Ottawa are 26.4°C and -14.8°C in the summer and winter respectively while the ground temperature is constant between 8°C and 9°C at a depth of around 6 m throughout the year. Therefore, there is a temperature change of up to 18°C from the surface to a depth of about 6 m that may cause a shift in penetrometer readings. Thus, there is the need to better understand the effect of temperature changes on penetrometer testing for Ottawa soils. Moreover, research is needed to verify existing correlations and empirical factors between CPTs and the engineering characteristics of marine clays in Ottawa. Therefore, this research aims to address the aforementioned needs.

The main objectives of this research are: (i) to characterize the engineering properties of sensitive marine clays in Ottawa (located at the Canadian Geotechnical Research Site No. 1) by using penetrometers; (ii) to understand the effect of temperature changes on penetrometer testing in soils in Ottawa; (iii) to assess existing correlations and empirical factors between CPTs and engineering properties of marine clays in Ottawa, with the view to developing guidelines for practicing engineers who work with these sensitive marine soils and other similar soils worldwide.

For the purpose of this research, field and laboratory tests are performed at the Canadian Geotechnical Research Site No. 1. The field tests performed include two CPTs with a cone tip that is 36 mm in diameter (CPT-1 & CPT-2), two ball penetrometer tests (BPTs) with a tip that has a 40 mm ball (BPT-1 & BPT-2), and one set of Nilcon field vane tests. Undisturbed samples are also collected and water table monitoring wells are installed. The variables in the laboratory tests that are investigated include unit weight, moisture content, plasticity limits, salinity and specific gravities and consolidation parameters. Furthermore, the grain size distribution of the samples is determined in the laboratory. Laboratory testing on temperature change is also carried out on the penetrometer to verify the effects of temperature changes on the penetrometer readings.

4.2 Site description

The Canadian Geotechnical Research Site No. 1 is a representative site for sensitive marine clay deposits in Ottawa (Eastern Canada). Previous researchers have performed field testing on the site, such as cone penetration, full-flow penetrometer, vane, and laboratory tests, to examine the plasticity properties by using oedometers and through triaxial testing (Lo et al. 1976; Yafrate and Delong 2006; Yafrate et al. 2009; DeJong et al. 2011). Therefore, there is an abundance of reference information available in the literature about the site.

4.2.1 Geographical location

The Canadian Geotechnical Research Site No. 1 (Figure 4-1) is located in south-east Ottawa on an average geodetic elevation of 80.9 m, northing of 5016868.8, and easting of 459267.3 according to the Transverse Mercator projection system (NAD_1983_MTM_8 projected coordinate system).

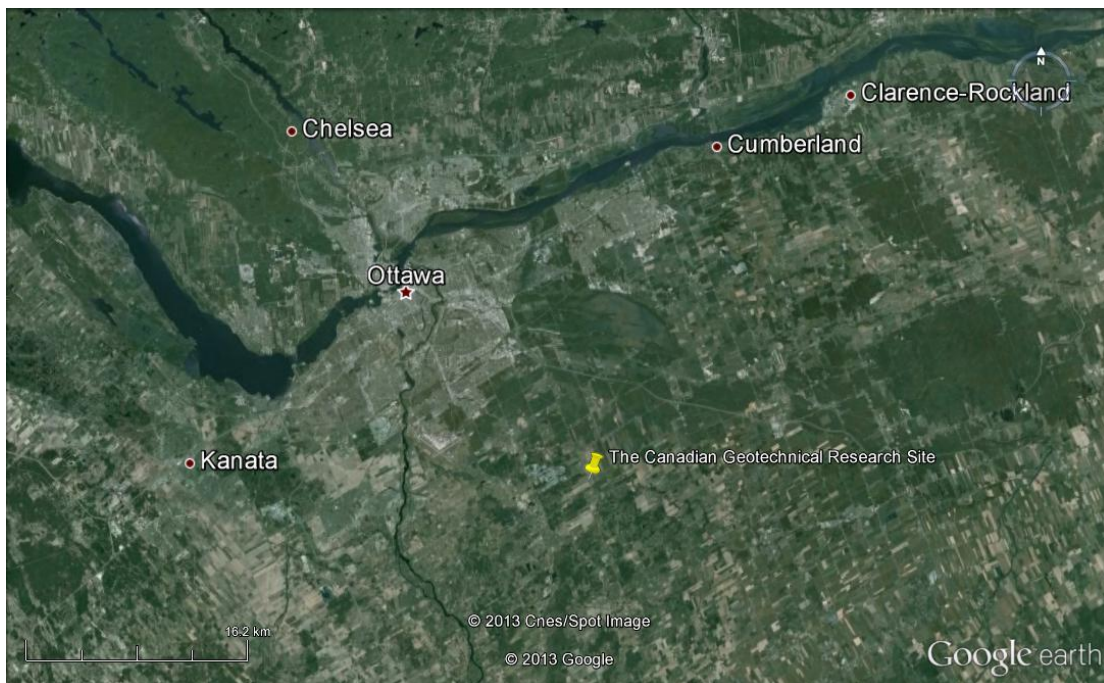


Figure 4-1 Geographical location of the Canadian Geotechnical Research Site No. 1

4.2.2 Geological characteristics

At the end of the glacial period, areas of Canada were depressed for hundreds of meters. After the glacial retreat, marine waters flooded the St. Lawrence River valley, thus forming the Champlain Sea (Crawford 1968). The Champlain Sea existed between 12000 and 8000 B.P. (Karrow 1961). During that period of time, there was an isostatic adjustment which resulted in the land rising as much as 230 m (Kenney 1964). The highest sea level was recorded near the Ottawa area at about 226 m above the present sea level (Johnston 1917). Streams that entered the sea from the north side as a result of the ice retreat deposited fine materials and formed deltas. Later, when the sea level rose, these deposits eroded and were redeposited back into the sea (Karrow 1961). Studies have mentioned that marine deposits were deposited into fresh water as floods invaded the sea from the great lakes. This assumption is based on the existence of non-oxidized, non-calcareous, and non-fossiliferous clay layers that lie on unoxidized calcareous fossiliferous clay layers (Gadd 1962). This also explains the drop in salt content in Ottawa clays (Crawford and Eden 1965).

The stratigraphy of the Canadian Geotechnical Research Site No. 1 generally consists of a clay layer that is 20 m in thickness under which lies bedrock or refusal layer that has a depth of about 22 m or a geodetic elevation of 59 m. The clay layer includes a 2 m crust near the ground surface.

Yafrate and DeLong (2006) provided a geological profile of the site. According to their profile, there is a crust layer that is approximately 2 m in depth, a meter of brown silty clay layer, 4.5 m of grey silty clay, 7 m of grey clay, and less than a meter of grey silty sand. The difference in the geotechnical profile description between the literature and this research may be due to the location of the field investigation. The field investigation performed for this study is near the gate and the sign of the site location while the location of field investigation carried out by previous researchers has not been clearly indicated.

4.3 Experimental Programs and Soil Characterization

4.3.1 Field testing programs

4.3.1.1 CPT Tests

Two CPTs were performed in accordance with ASTM D 5778. The tip stress, sleeve friction, porewater pressure, and temperature were recorded in each sounding. An average penetration rate of 20 mm/sec was used. The diameter of the cone tip is 36 mm and the push rods have the same diameter. Porewater pressure was measured at Position 2 which allows for tip stress correction.

4.3.1.2 Ball Penetrometers

Soundings were performed by carrying out two BPTs, one with two penetrometers that have a 40 mm ball, and one with a penetrometer that has a 113 mm ball. A penetration rate of 20 mm/s was used for the latter while a rate of 12 mm/sec (0.2-0.3 times the diameter per second) was used for the former as recommended by DeJong et al. (2010).

4.3.1.3 Field Vane Shear Tests

A standard electric field vane was used (ASTM D2573) with a tapered lower end that has a dimension of 130 x 65 mm. A rate of 0.1 deg./s was used and ten rapid rotations were performed before the remolded shear strength was determined. A series of tests on undrained and remolded shear strength at a 1 m interval were performed.

4.3.1.4 Drilling and sampling

Undisturbed samples were collected for laboratory testing in accordance with ASTM D 1587 for thin-walled tube sampling. Shelby tubes that are 0.0762 m (3 in) in diameter and 0.762 m (2.5 ft) in length were used. The tubes meet the criterion for a 1% inside clearance ratio as specified by ASTM standards. A piston sampler was used to ensure the quality of sampling and avoid the material from falling out of the tube due to suction. The boreholes were pre-augered in advance to a depth of 1.5 m (5 ft) and sampling started at the same depth at intervals of 0.762 m (2.5 ft) and spacing of 0.0015 m (0.5 ft). The piston was kept in a locked position at the bottom of the tube while it was being pushed to the desired depth. Then, the piston was fixed at a specific

depth and only the tube was pushed to a distance of 0.43 m rather than 0.762 m in order to prevent the disturbance of the sample by the piston head. While pulling the tube up to the surface, the piston was locked at the top of the tube to provide pressure from suction which would prevent any material from falling out of the tube. Also, a distance of 0.15 m (0.5 ft) was placed between samples to minimize disturbance.

4.3.1.5 Wells installation

Multi-level monitoring wells were installed in accordance with ASTM D 5092 to monitor the water level or porewater pressure at two different depths. Temporary auger casing was provided to a geodetic elevation of 59 m. Screens (1.5 m in length) were installed at elevations of 59.3 and 74.5 m, and surrounded with clean sand up to 0.3 m above the screen level. Above the sand, a 1 m seal of bentonite was situated to prevent water from penetrating into the screen from the higher layers. Grout was used around the riser up to 1 m below the surface where 1 m of bentonite was used to seal the well from the surface. A 0.5 m stick up with protective steel casing was used. Readings of the water level in the two wells started the next day from the installation time on an average of once a day for ten days and once a month afterwards, as shown in figure 4-2. A general increase in the water level in the two wells can be noted towards the winter months. This is probably a result of the increased overburden stress due to snow accumulation. Porewater pressure readings at Position 2 (u_2) were measured by the cone penetrometer, and penetrometers with 40 and 113 mm balls. The readings are shown in figure 4-3 with hydrostatic pore water pressure. As expected, there is a reduction in the excess pore water pressure as the tip projected area increases. Furthermore, u_2 sensitivity to stratification and identification of the layer boundaries is reduced as the tip projected area increases.

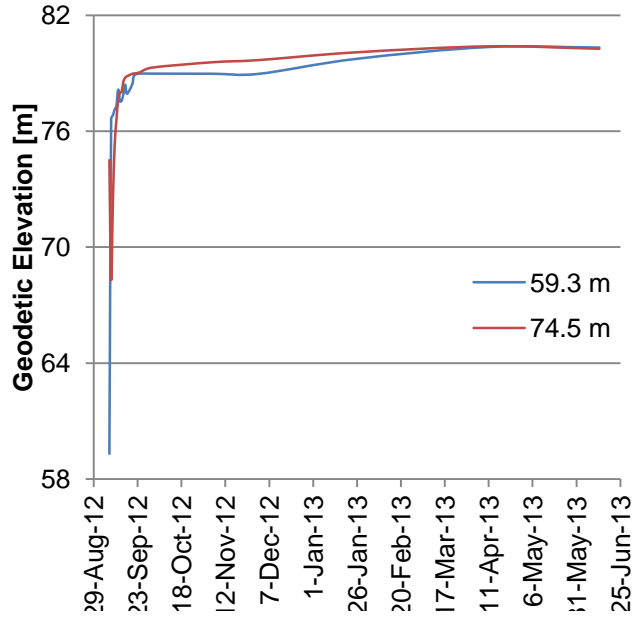


Figure 4-2 Water table geodetic elevations with dates from the multilevel monitoring wells

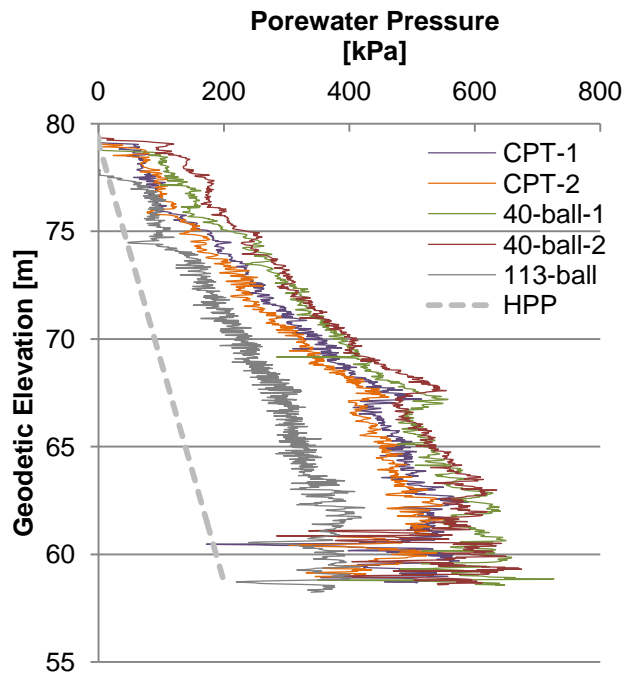


Figure 4-3 Porewater pressure readings from cone, 40 mm ball, and 113 mm ball tips and hydrostatic porewater pressure with geodetic elevations, CPT-1 and CPT-2 are porewater pressure readings from cone tip tests 1 and 2, 40-ball-1 and 40-ball-2 are porewater pressure readings from 40 mm ball tip tests 1 and 2, 113-ball is porewater pressure readings from 113 mm ball tip test, HPP is hydrostatic porewater pressure readings from monitoring wells readings

4.3.1.6 Geodetic Elevations and coordinates

Table 4-1 is a summary of the geodetic elevations, easting, and northing of all field tests.

Table 4-1 Geodetic surface elevations, easting, and northing of field testing points

Label	Geodetic Surface Elevation [m]	Northing	Easting	Description
BPT-1	80.801	5016876.239	459278.466	40 mm ball
BPT-2	80.655	5016874.562	459286.533	40 mm ball
CPT-1	80.449	5016861.039	459294.614	36.6 mm cone
CPT-2	80.899	5016869.712	459275.890	36.6 mm cone
Bore Hole	80.670	5016865.117	459281.522	Undisturbed Samples and multi-level Monitoring Wells
Field Vane	80.735	5016870.089	459292.803	Nilcon Field Vane
BPT-3	80.907	5016869.305	459279.829	113 mm ball

4.3.2 Laboratory testing programs and soil characterization

4.3.2.1 Unit weight, moisture content, and specific gravity

Unit weight, moisture content, and specific gravity were determined for 11 samples collected from the field in accordance with ASTM D 7263, ASTM D 2216, and ASTM D 854 respectively (Figure 4-4). The displacement method was used to determine the unit weight, while specific gravity was determined by using oven-dried specimens. The site is characterized by three layers: a top layer of 0.75 to 1.2 m, an intermediate layer of 19 m, and a bottom layer of 1 m down to a geodetic elevation of 59 m (21 m in depth). The top, intermediate and bottom layers have a unit weight of 22.8 kN/m³, 16 kN/m³ and 19.1 kN/m³ respectively. The moisture content follows a pattern that is similar to the unit weight, in which the top, intermediate and bottom layers have an average moisture content of 11.4%, 68.3% (range from 46% to 89.2%), and 37%, respectively. The specific gravity has a narrow range from 2.74 to 2.79 with the lowest value recorded at the top layer.

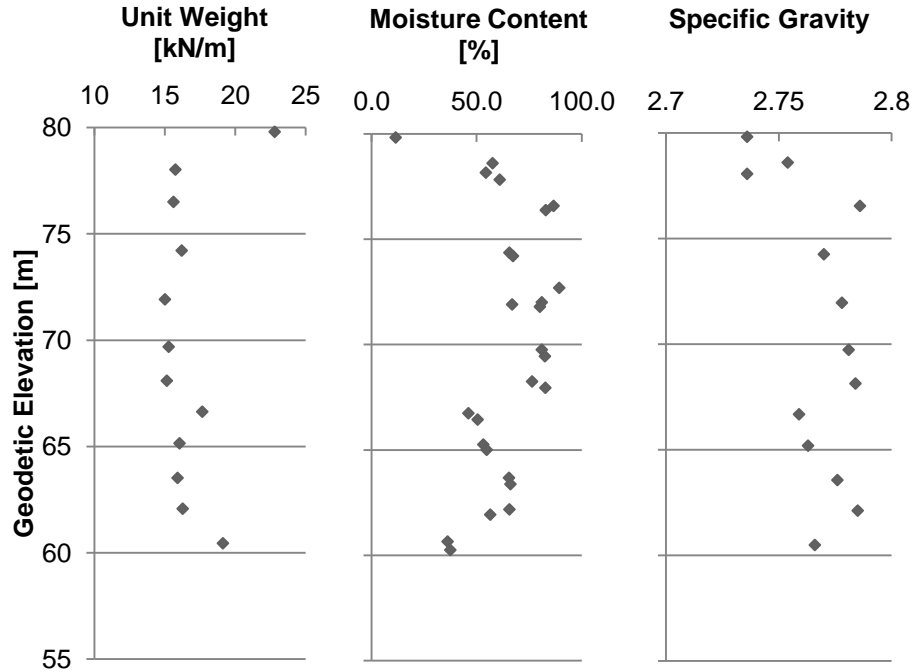


Figure 4-4 (a) unit weight with geodetic elevations (b) moisture content with geodetic elevations (c) specific gravity with geodetic elevations

4.3.2.2 Grain size distribution and soil index properties

Grain size distribution and plasticity limits were determined in accordance with ASTM D 422 and ASTM D 4318 respectively. The top layer is a sandy silt layer with 16.9% clay and 2.6% gravel. Under the top layer is a deep silty clay layer up to 20 m thick. The fines content ranges from more than 80% to less than 80% clay particles. This type of deposition has been reported by Leroueil (1999) as a result of summer silty and winter clayey depositions, but in terms of smaller couplets that are a few centimeters in thickness. With respect to geodetic elevation, the silty deposition layer ranges from 76 to 78 m, 63.5 to 67.5 m, and 59 to 61.5 m where CPT refusal takes place at 59 m, with transition layers of 1.5 to 3 m, respectively. Clayey deposition layers vary in thickness of 2 to 8.5 m. The moisture content parallels with the layering in that there is a decrease in the former when there is an increase in the silt content as in geodetic elevations of 60.5 and 66.5 m. There is also an increase in the unit weight at the same elevations to 17.7 kN/m^3 and 19.1 kN/m^3 respectively, while specific gravity shows a decrease at these depths (Figure 4-4 and 4-5). The plastic limits, liquid limits, and plasticity index range between 26.3% to 59.9%, 14.8% to 37%, and 11.5% to 28.1% respectively (Figure 4-6). DeJong et al. (2011)

reported higher values at two depths. He reported the plastic limit, liquid limit, and plasticity index to be 27, 71, and 44 for a depth of 4.4 m, and 28, 62, and 34 for a depth of 8.3 m respectively.

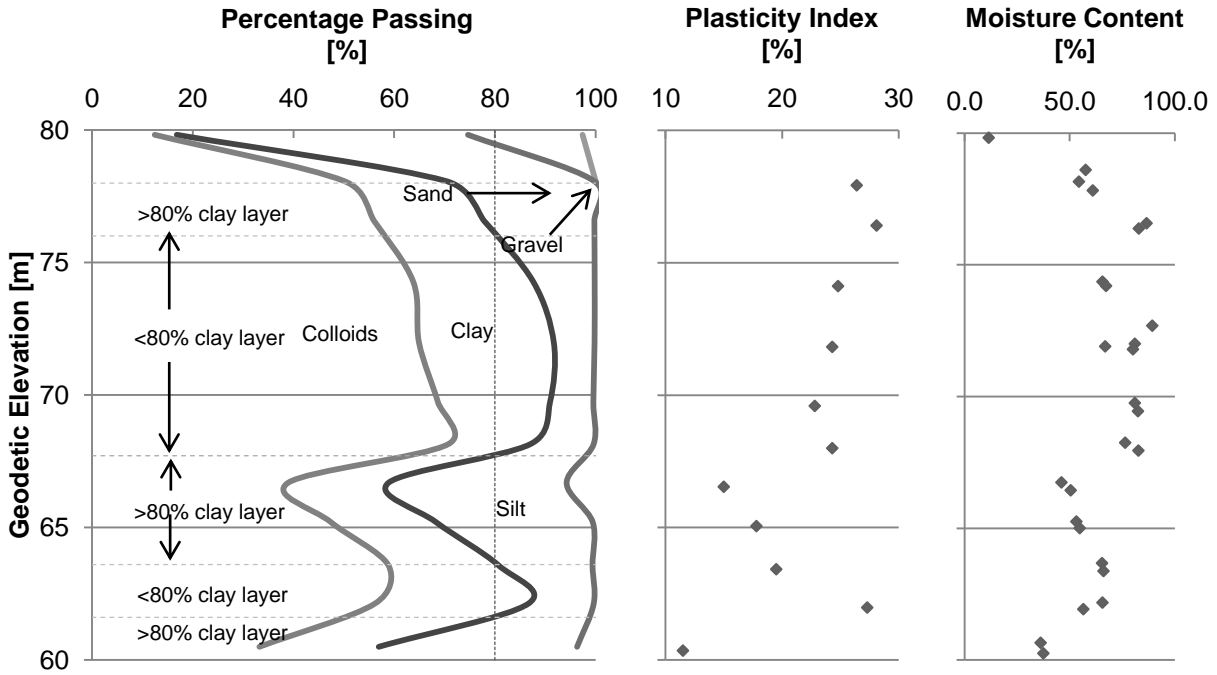


Figure 4-5 (a) percentage particles size with geodetic elevations (colloids <0.001 mm, clay 0.001-0.005 mm, silt 0.005-0.075, sand 0.075-0.04.75 mm, gravel 4.75-75 mm) (b) plasticity index with geodetic elevations (c) moisture content with geodetic elevations

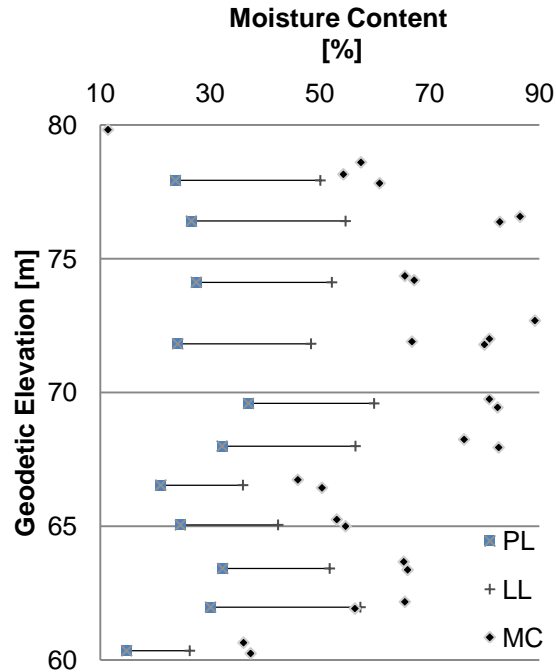


Figure 4-6 Moisture contents, plastic limits, and liquid limits with geodetic elevations, PL is the plastic limits, LL is the liquid limit, MC is the moisture content

4.3.2.3 Preconsolidation Pressure

One-dimensional consolidation tests were performed in accordance with ASTM D 2435. Preconsolidation pressure and OCR values are shown with geodetic elevation in Figure 4-7. It is evident that there is a general increase in the preconsolidation pressure and decrease in the OCR with depth. The soil is lightly overconsolidated at an elevation of 75 to 80 m, and normally consolidated at an elevation lower than 75 m. DeJong et al. (2011) reported that the soil in their investigation is overconsolidated to 1.8 at a depth of 4.4 m and 1.5 at a depth of 8.3 m.

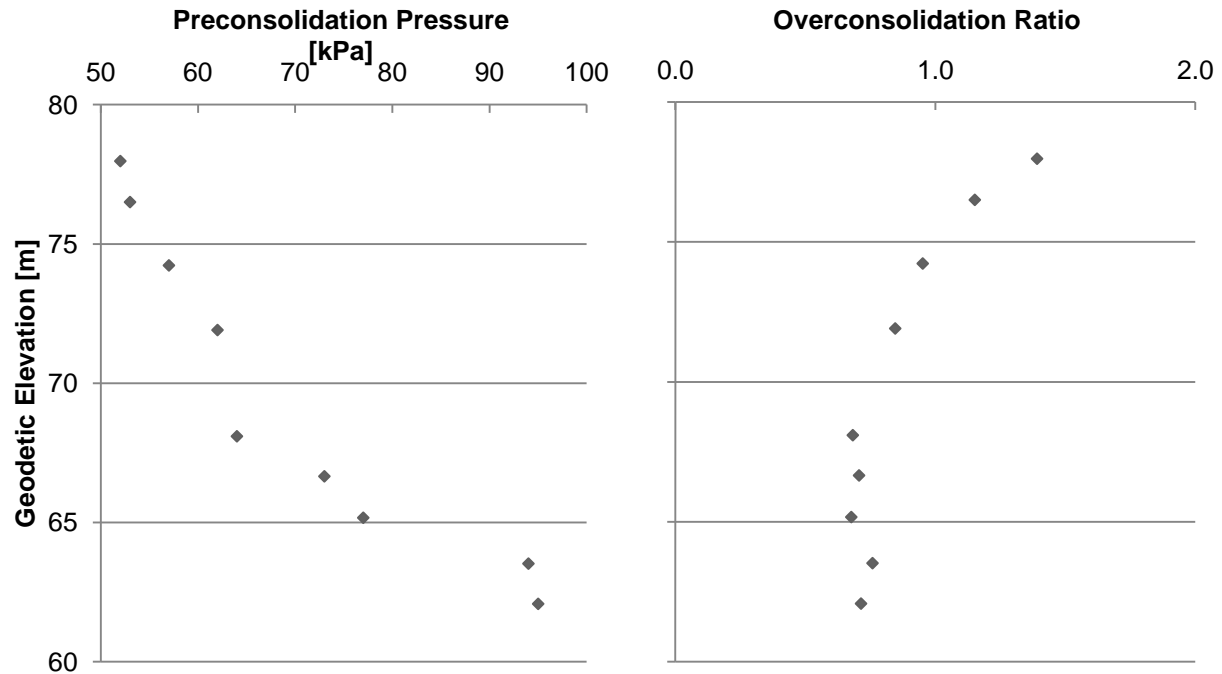


Figure 4-7 (a) preconsolidation pressure with geodetic elevations (b) overconsolidation ratio with geodetic elevations

4.3.2.4 Porewater Chemistry

The pore water chemistry was analyzed for sodium, potassium, magnesium, and calcium cations and chloride anions (Na^+ , K^+ , Mg^{+2} , Ca^{+2} , and Cl^-). Chloride anions seem to parallel the sensitivity and plasticity index. There are noticeable increases in the chloride concentrations at elevations of 63.5, 70 and 76.5 m, and similar increases in the sensitivity (figure 4-14) and plasticity limits (figure 4-18) at similar depths. There is also similar behavior in the sodium, magnesium, and potassium profiles, but with less clarity than in the chloride profile. Therefore, the relation between chloride, sodium, magnesium, and potassium on the one hand and sensitivity and plasticity on the other hand is positive. In other words, there seems to be an increase in the sensitivity and plasticity as chloride, sodium, magnesium, and potassium concentrations increase. Calcium does not seem to be related to the sensitivity or plasticity (figure 4-8).

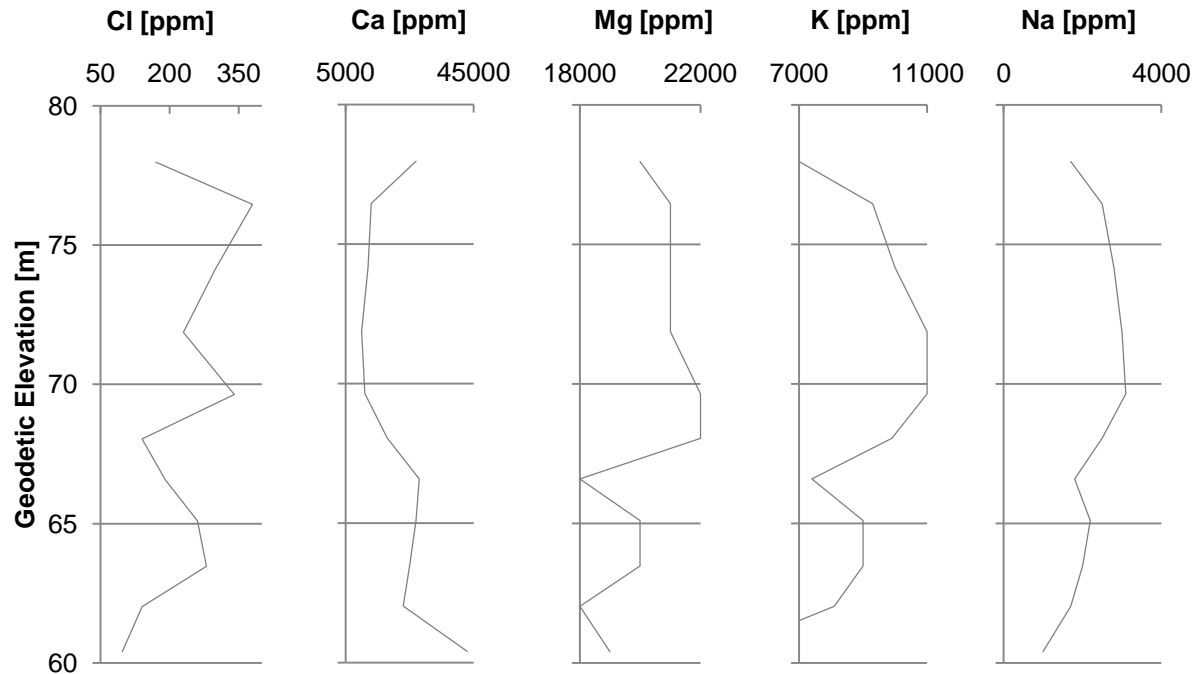


Figure 4-8 (a) chloride Cl anions (b) calcium Ca cations (c) magnesium Mg cations (d) potassium K cations (e) sodium Na cations with geodetic elevations

4.3.2.5 Cone Equipment Calibration and Temperature Effects

The results gathered by CPTs on sensitive marine clays have been problematic. Temperature effects may induce negative readings and/or increases or decreases in tip resistance, sleeve friction, and porewater pressure readings. Temperature effects may also cause a shift in the baseline reading of the soundings. According to ASTM D 5778, baseline readings must take place before and after each penetrometer sounding where the equipment is vertically and freely hung with zero loads. The initial baseline reading is used as a reference or zero transducer reading while the final baseline reading with reference to the initial baseline reading is used to verify the reliability of the data or determine the baseline drift due to temperature or other effects. According to The Weather Network statistics for Ottawa, an average high temperature of 26.4°C is recorded for July, and an average low temperature of -14.8°C is recorded for January. According to Williams and Gold (1976), at a depth of 5 to 6 m, the ground temperature is constant at 8°C to 9°C, and changes about 1°C for every 50 m in depth due to the geothermal heat from the centre of the earth. Therefore, penetrometer equipment may experience a

temperature change of up to 18°C, assuming that it is not practical to perform penetration testing in temperatures below -10°C.

In order to verify the temperature effects on penetrometer readings, temperature experiments were performed in the laboratory. To address the cooling effects that result from performing penetrometer sounding in hot weather, the penetrometer was connected and left to stabilize in room temperature for 30 minutes. Then, the penetrometer was transferred to a refrigerator with controlled temperatures. Thereafter, regular readings were taken for tip stress, sleeve friction, and porewater pressure. To address the heating effects that result from performing penetrometer sounding in cold weather, the penetrometer was left to stabilize in a refrigerator. Then, it was transferred from the refrigerator to the laboratory to stabilize at room temperature. Figures 4-9, 4-10, and 4-11 show the effects of temperature changes on penetrometer readings.

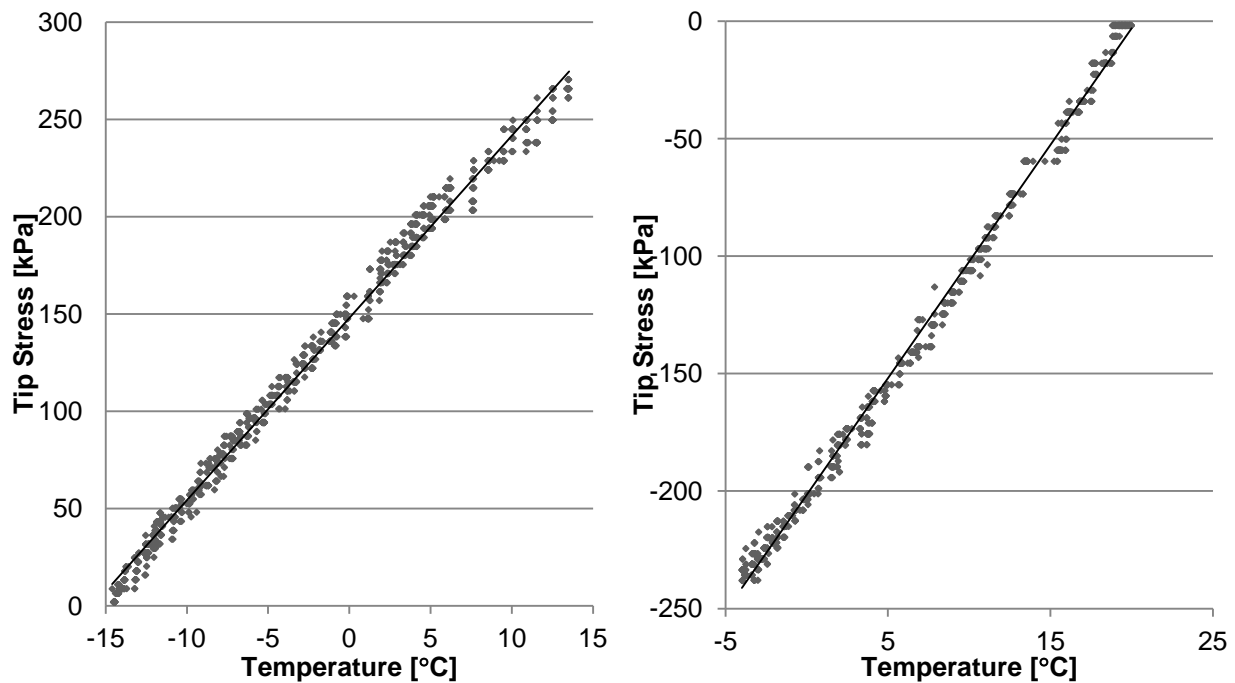


Figure 4-9 (a) tip stress gain with temperature change due to heating (b) tip stress loss with temperature change due to cooling

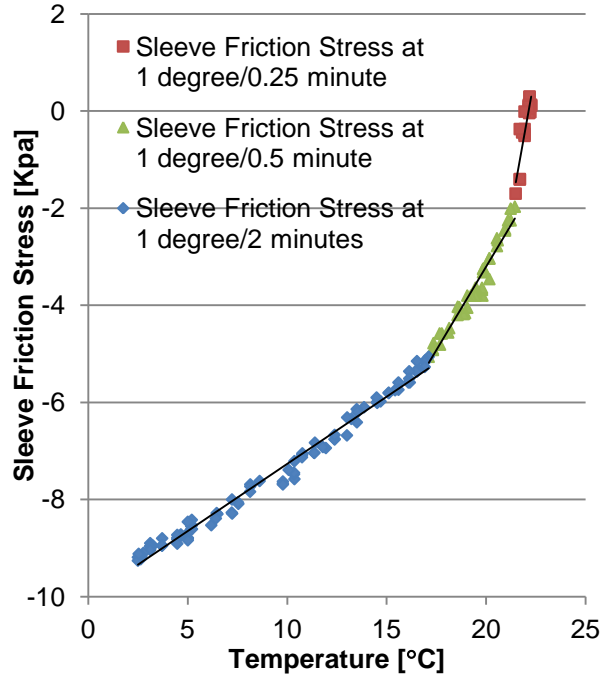
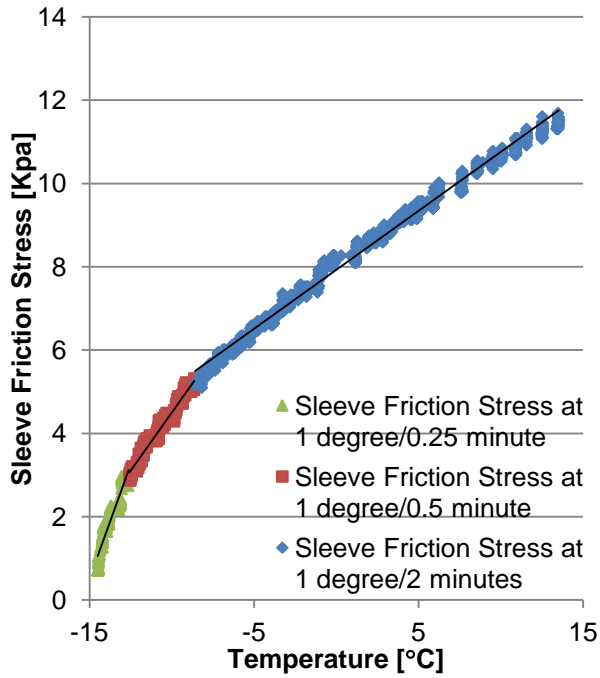


Figure 4-10 (a) sleeve friction stress gain with temperature change due to heating (b) sleeve friction stress loss with temperature change due to cooling

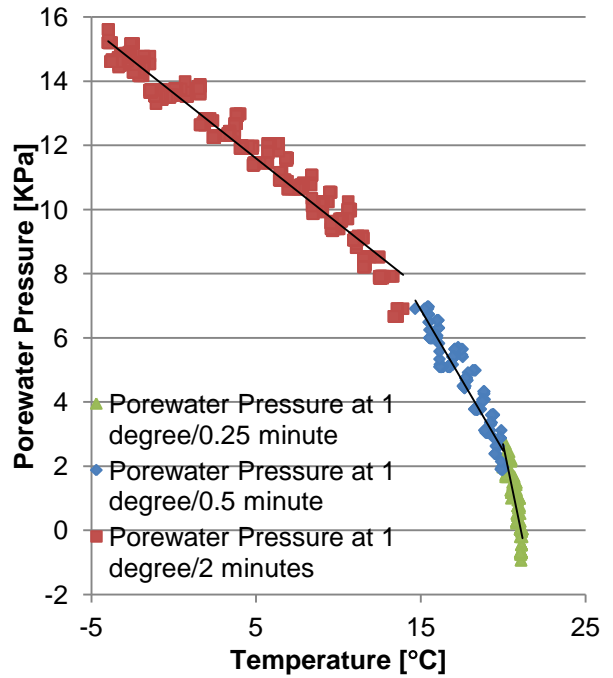
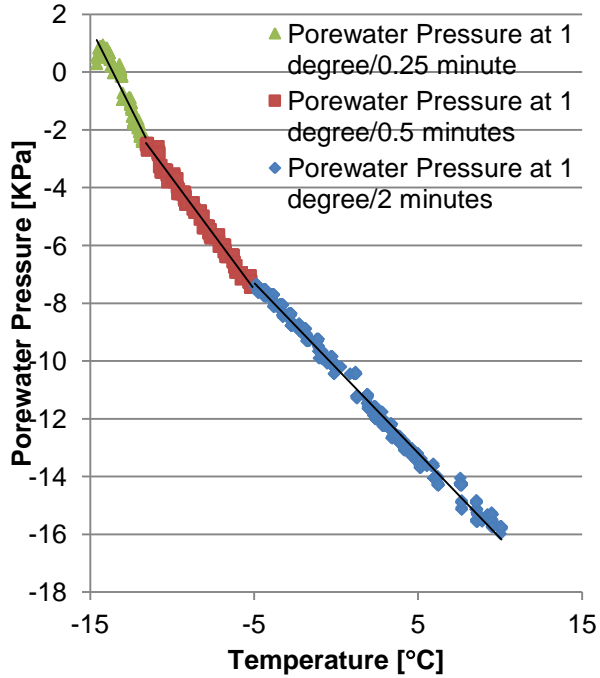


Figure 4-11 (a) porewater pressure loss with temperature change due to heating (b) porewater pressure gain with temperature change due to cooling

As a result of heating, there is an increase in the tip stress and sleeve friction readings, but a loss in pore water pressure readings. On the other hand, as a result of cooling, there is a loss in the tip stress and sleeve friction readings, but a gain in pore water pressure reading. Tip stress changes that result from temperature changes are independent of the rate of temperature change. However, sleeve friction stress and porewater pressure readings are affected by the rate of the temperature change. Temperature change curves could be modelled by using a best fit linear equation. Also, it was found that the rate of the temperature change in the field is close to 1°C/0.5 minute. Therefore, the stress change rates with respect to temperature in Table 4-2 are adopted. There is no difference in the stress change rates by using the cone, 40 mm ball, or 113 mm ball to measure sleeve friction and porewater pressure. However, the changes in tip stress readings in the three penetrometers are different. The cone tip reading is greatly affected by temperature changes, the 113 mm ball reading is least affected by temperature changes, and the 40 mm ball reading is somewhat affected. Therefore, the stress reading changes due to temperature effects are dependent on the tip projected area. When the tip projected area is larger, stress changes due to temperature changes are lower (Table 4-2).

Table 4-2 tip stress, sleeve friction, and porewater pressure changes due to temperature effect according to author's laboratory testing

	Tip Stress change [kPa/°C]			Sleeve Friction change [kPa/°C]	Porewater Pressure change [kPa/°C]
	Cone Tip [36 mm]	Ball Tip [40 mm]	Ball Tip [113 mm]		
Heating	8.3	6.75	0.85	1.18	-0.77
Cooling	-9.27	-7.54	-0.95	-1.24	0.88

According to the information that the authors were able to obtain from the penetrometer manufacturers, the manufacturers test the load sensors used on the penetrometer probes rather than the probes themselves. According to a document obtained by the authors, load sensors are heated from 75°F to 150°F, which is approximately equivalent to 23.9°C to 65.6°C. The sensors exhibited reading changes that were equivalent to approximately 3.2 kPa in sleeve friction, 221.6

kPa in cone tip resistance, and 4.6 kPa in pore water pressure. These changes meet the ASTM standards. Table 4-3 shows the tip resistance, sleeve friction, and porewater pressure changes due to the temperature effect according to the manufacturer's testing.

Table 4-3 Tip stress, sleeve friction, and porewater pressure changes due to temperature effect according to manufacturer's testing

Tip Stress Change [kPa/°C]			Sleeve Friction Change [kPa/°C]	Porewater Pressure Change [kPa/°C]
Cone Tip [36 mm]	Ball Tip [40 mm]	Ball Tip [113 mm]		
5.33	4.32	0.54	0.08	0.11

It is mentioned in ASTM D 5778 that the tip and sleeve friction cells can take a 2% shift of the baseline of a full-scale output or the maximum load for cone penetration testing. For example, if the full-scale output for the tip stress and sleeve friction is 34181.5 kPa and 689.5 kPa respectively, the allowed baseline shift would be 696.4 kPa and 13.8 kPa respectively. A penetrometer sounding for a typical summer in Canada may experience a temperature change of up to -18°C. Therefore, the cone penetrometer used in this experiment may experience a loss of tip resistance up to 166 kPa and a sleeve friction up to 22 kPa. This magnitude of stress loss may induce negative and erroneous values for soundings made in sensitive marine clays where the tip resistance and sleeve friction could be as low as 100 kPa and 1 kPa respectively. Three

temperature profiles with depth are shown in Figure 4-12 as an example.

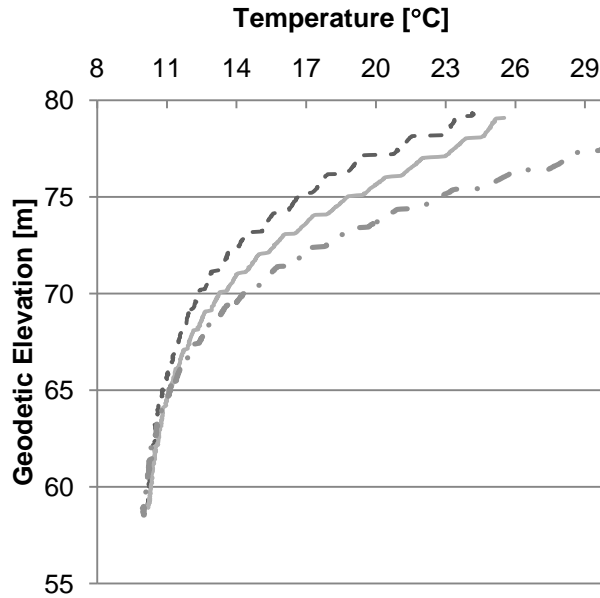


Figure 4-12 temperature profile from three penetrometers tests

4.4 Results of the penetrometer tests and discussion

4.4.1 Soil Specific Cone Factors

4.4.1.1 Cone Penetrometer

Two CPTs were performed and the results are similar, see Figure 4-13. The tip resistance ranges from approximately 100 to 800 kPa for a geodetic elevation of 61 to 78 m. Similarly, the sleeve friction ranges from 1 to 19 kPa and porewater pressure from 100 to 650 kPa.

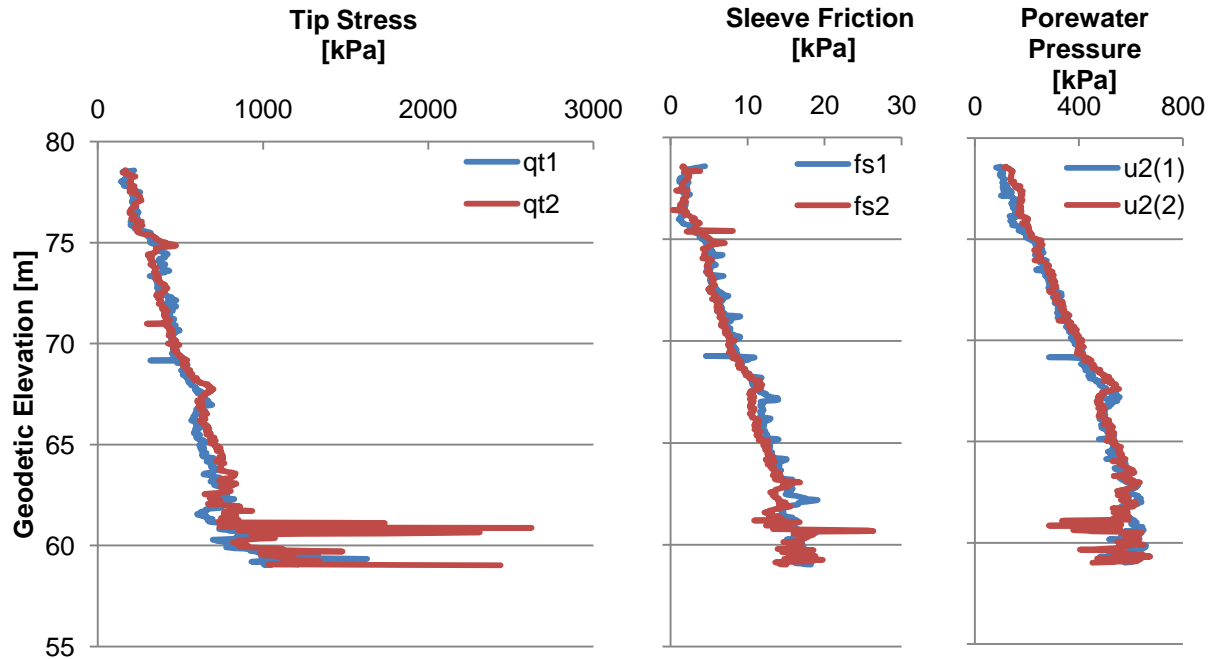


Figure 4-13 cone penetration tests results with geodetic elevations for (a) tip resistance (b) sleeve friction (c) porewater pressure, qt1 and qt2 are tip resistances for tests 1 and 2, fs1 and fs2 are sleeve frictions for tests 1 and 2, u2(1) and u2(2) are porewater pressures for tests 1 and 2

Previous studies have shown that N_{kt} ranges between 8 and 16 for clays relative to triaxial and direct shear tests (Aas et al. 1986; Lunne et al. 1997). The studies mentioned that this range is for plasticity indexes between 3% and 50%. An increase in N_{kt} is also associated with a decreasing plasticity index. Figure 4-14 shows that for this site, a decrease in the plasticity index is associated with a general increase in N_{kt} at elevations of 66 and 75 m. This response is also observed in the N_k and $N_{\Delta u}$ values at the same depths. It is also shown that N_{kt} can be between 11 and 18 (La Rochelle et al. 1988). Another study also mentioned that N_{kt} may vary between 8 and 29 relative to triaxial tests and also to the OCR (Rad and Lunne 1988). It was mentioned that N_{kt} values may vary between 10 and 20 depending on the plasticity index, but this range may increase to 10 and 30 in fissured and overconsolidated clays (Powell and Quarterman 1988). A study was carried out on Danish till which provided N_{kt} values that ranged between 8.5 and 12 with an average of 10 (Luke 1995). At this site, N_{kt} values between 8.1 and 11.8 for only two depth intervals were reported (DeJong et al. 2011). Similar values are obtained in this study. However, temperature corrections are used. Temperature corrections have increased the N_{kt} values from 2.4 to 8.1 to a similar range of that in previous studies which is 8.0 to 11.8 with an

average of 9.4. Temperature corrections have also decreased the standard deviation of the N_{kt} values from 1.5 to 1.1. Temperature corrections have also improved the values and decreased the range and standard deviation for N_k and $N_{\Delta u}$ values for this site as shown in Table 4-4. For normally consolidated clays, N_k values with reference to field vane tests vary between 11 and 19 with an average of 15 (Lunne and Kleven 1981). However, from this study, it was found that N_k values, after temperature corrections, range between 6.0 and 11.3 with an average of 7.6 and standard deviation of 1.2. It was found that $N_{\Delta u}$ values relative to field vane tests on three Canadian clays range between 7 and 9 for an OCR that ranges from 1.2 to 50 (La Rochelle et al. 1988; Lunne et al. 1997). Another study used triaxial tests as reference and found that $N_{\Delta u}$ varies between 6 and 8 (Karlsrud et al. 1996). From this study, after temperature corrections, the range for $N_{\Delta u}$ values was found to be 5.5 to 10.4 with an average of 8.7 and standard deviation of 1.2. The N_{kt} , N_k , and $N_{\Delta u}$ values are presented in Table 4-4.

Table 4-3 soil specific values for the cone tip

	Without Temperature Correction			With Temperature Correction		
	N_{kt}	N_k	$N_{\Delta u}$	N_{kt}	N_k	$N_{\Delta u}$
Average	5.8	4.0	9.1	9.4	7.6	8.7
Range	2.4-8.1	0.5-7.8	5.8-10.7	8.0-11.8	6.0-11.3	5.5-10.4
STDev	1.5	1.6	1.3	1.1	1.2	1.2

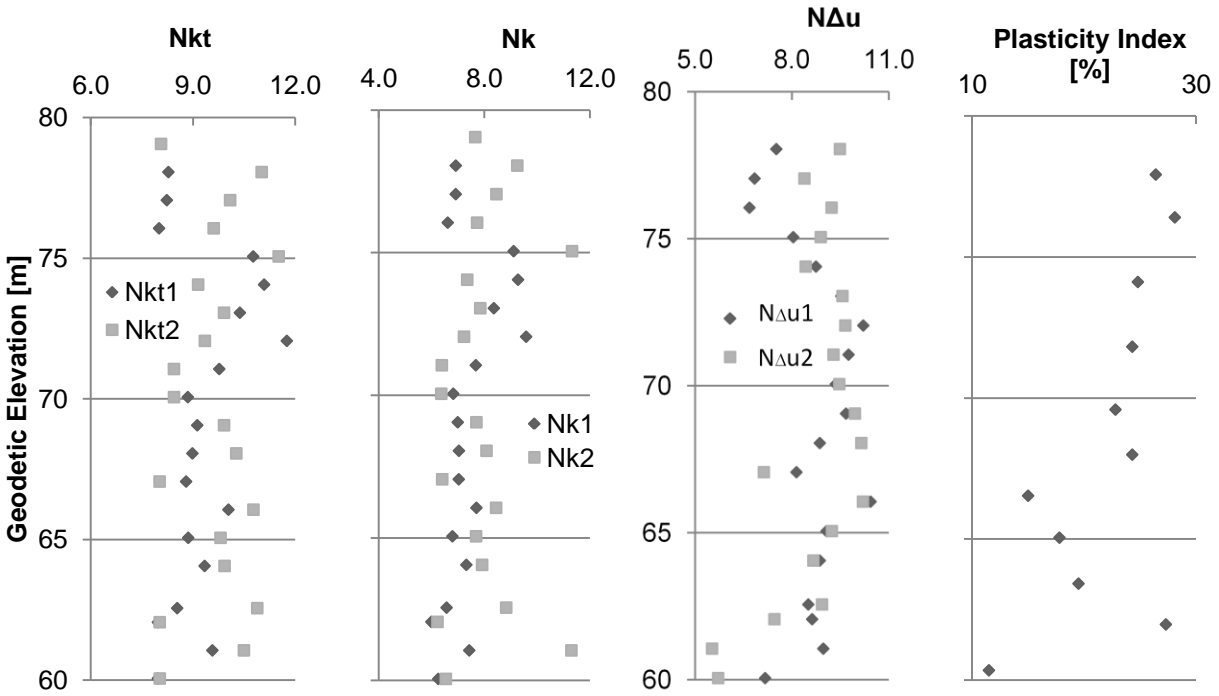


Figure 4-14 cone soil specific factors and plasticity index with geodetic elevations, Nkt1 and Nkt2 are cone soil specific factors from corrected tip resistance for cone tests 1 and 2, Nk1 and Nk2 are cone soil specific factors from uncorrected tip resistance for cone tests 1 and 2, $N_{\Delta u1}$ and $N_{\Delta u2}$ are cone soil specific factors from excess porewater pressure for cone tests 1 and 2

Lunne et al. (1997) suggested that there is a correlation between N_{kt} and the pore pressure ratio (B_q), which is the ratio of the excess porewater pressure to the difference between the corrected tip resistance and the total overburden pressure. Their study suggested a range of 6 to 15 for N_{kt} , which increases with a decreasing OCR. For this site, N_{kt} also seems to decrease when the pore pressure ratio is increased and OCR is decreased, but with a range of 8.0 to 11.8 as shown in Figure 4-15(a). Marsland and Powell (1988) and Powell and Quarterman (1988) proposed that there is a correlation between the N_k value and the index of plasticity. Their study shows an increase in N_k with an increase in the plasticity index. For this study, there is also a general increase in N_k within a range of 6.0 to 11.3 with an increase in the plasticity index as shown in Figure 4-15(b). Furthermore, the N_k values have a slightly higher scatter level with a high plasticity index. Lunne and Lacasse (1985) found a correlation between $N_{\Delta u}$ and B_q for North Sea clays that ranges from 4 to 10 with reference to triaxial tests. This study has also found that

there is an increase in $N_{\Delta u}$ with increasing B_q , but for a range of 5.5 to 10.4 and reduced scatter towards higher values of B_q as shown in Figure 4-15(c).

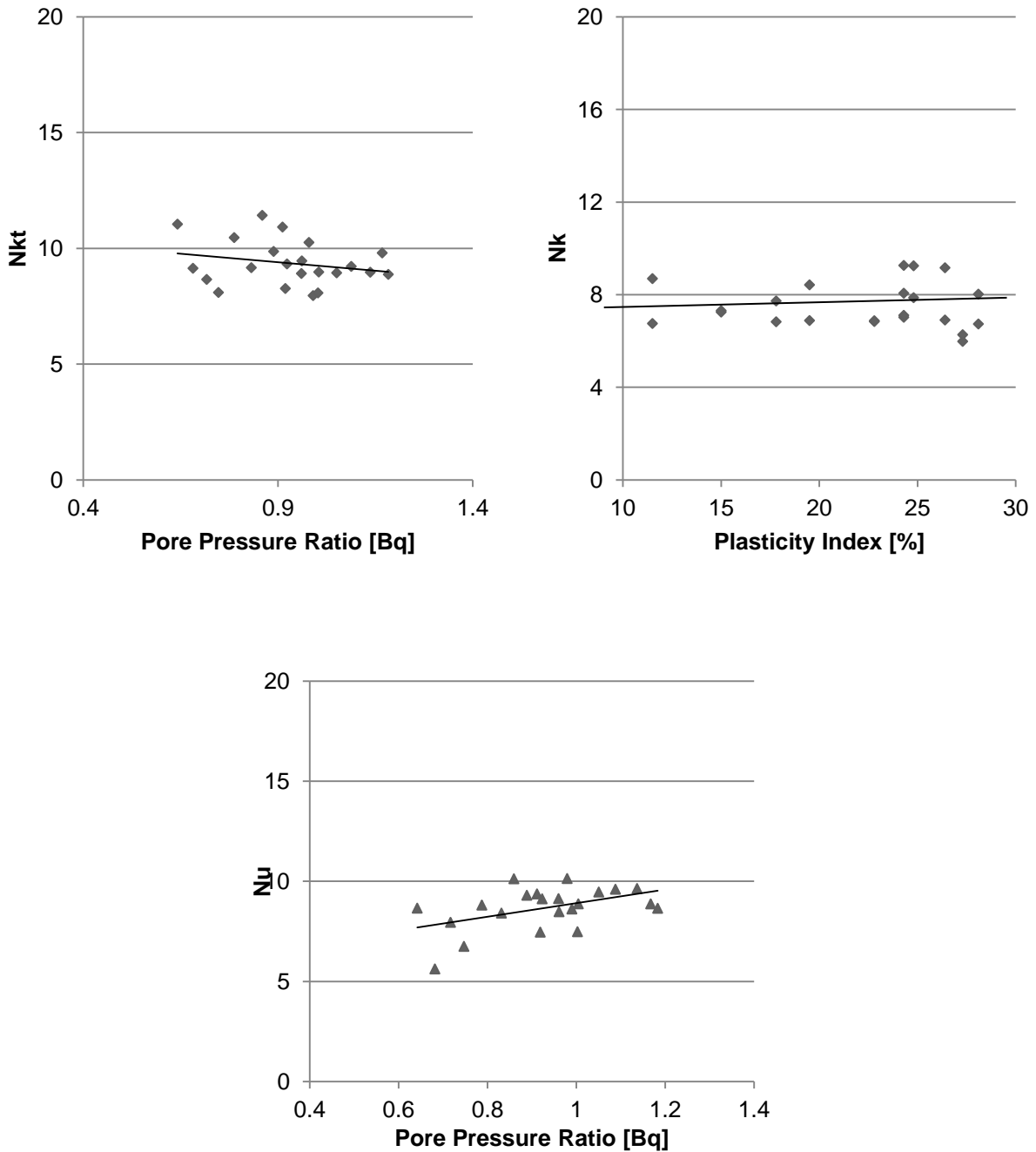


Figure 4-15 (a) cone soil specific factor N_{kt} from corrected tip resistance with porewater pressure ratio (b) cone soil specific factor N_k from uncorrected tip resistance with plasticity index (c) cone soil specific factor N_u from excess porewater pressure with porewater pressure ratio

4.4.1.2 Ball Penetrometers

Two tests were carried out by using a ball penetrometer with a 40 mm ball and one was carried out with a 113 mm ball (Figure 4-16). The tip resistance from the two 40 mm balls was approximately the same. The tip resistance when the 40 mm ball was used ranged between 100 and 600 kPa from 61 to 88 m in terms of the geodetic elevation. The tip resistance when the 113 mm ball was used ranged between 0 and 450 kPa from 61 to 99 m in terms of the geodetic elevation.

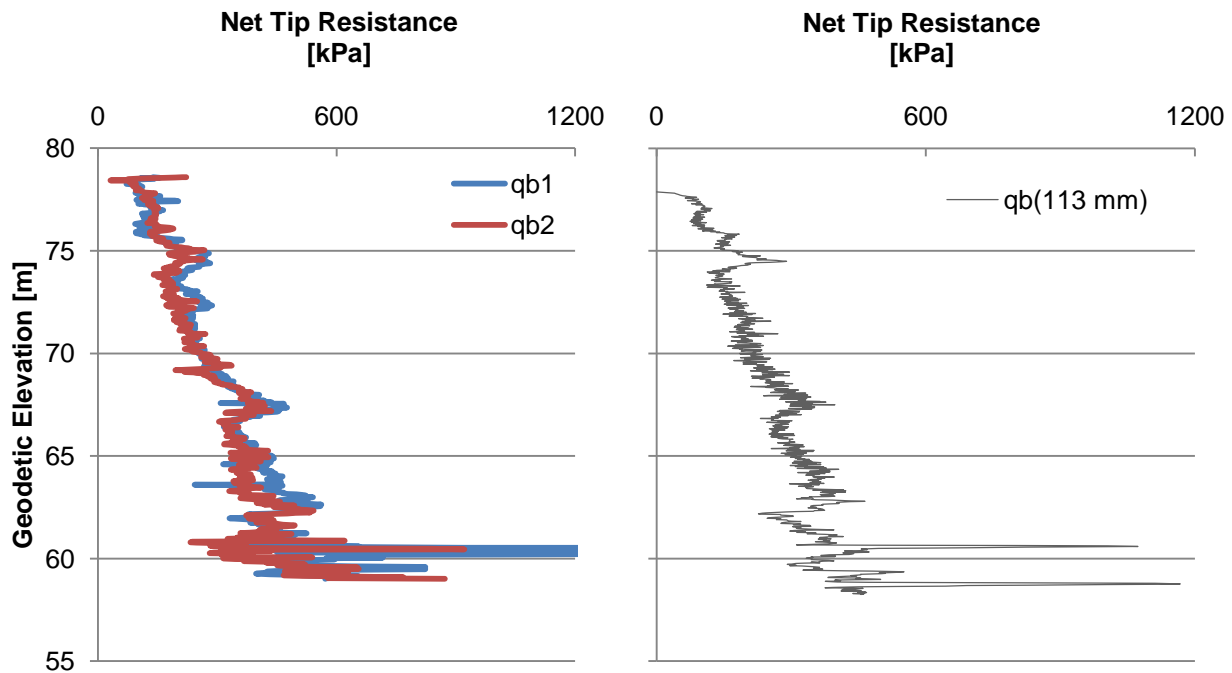


Figure 4-16 Net tip resistance with geodetic elevations for (a) 40 mm ball tip (b) 113 mm ball tip, qb1 and qb2 are net tip resistances for 40 mm tip tests 1 and 2, qb(113mm) is net tip resistance for 113mm ball tip test

Previous studies have proposed N_{su} values for the penetrometer with a 113 mm ball from five different sites (Yafrate and DeLong 2006; Yafrate et al. 2007; DeJong et al. 2009 and 2011). For varved, medium stiff and lightly overconsolidated clays, a range from 9.6 to 11.0 was proposed by DeGroot and Lutenegro (2003). For high plasticity, low overconsolidated, and soft to medium stiff clayey silts, a range from 13.6 to 15.9 was proposed by Chung (2005). For highly plastic and uniform marine clays with low OCR, N_{su} values in the range of 11.4 to 12.5 were obtained by Lunne et al. (2003). For Canadian sensitive marine clays, Louiseville in Quebec and

Gloucester in Ontario were considered. Louiseville has medium stiff homogeneous clay with high plasticity and sensitivity. N_{su} values proposed for Louiseville were 8.3 to 8.7. Gloucester, on the other hand, has soft to medium stiff sensitive clay with three different layers. It was also mentioned that the undrained shear strength ranges between 10 and 57 kPa and the sensitivity is between 20 and 100. The proposed N_{su} values were in the range of 5.8 to 6.0 for two depths. However, from this study, it was found that the undrained shear strength ranges between 13 and 63 kPa and the sensitivity ranges between 0.5 and 26. The N_{su} values from this study range between 5.4 and 6.9 with an average of 6.2 and a standard deviation of 0.5; see Table 4-6 and Figure 4-17(a). It is worth mentioning that temperature corrections do not seem to affect the penetrometer with a 113 mm ball as much as that with a conventional cone due to the increased projected area. The difference in N_{su} before and after temperature correction is 0.3 and there is no effect on the standard deviation (Table 4-6).

For the Canadian Geotechnical Research Site No. 1, an average value for N_{su-40} for the penetrometer with a 40 mm ball is proposed. It was found that N_{su-40} ranges between 7.5 and 11.4 with an average of 8.5 and a standard deviation of 1.1 as indicated in Table 4-5. It can be noticed that the penetrometer with a 40 mm ball is less affected by temperature correction than the conventional cone penetrometer as a result of the increase in the tip projected area. The difference between the corrected and uncorrected average values of N_{su-40} is 2.2 and there is no difference in standard deviation. On the other hand, the difference between the average corrected and uncorrected values of N_{kt} is 3.6 and the difference in the standard deviation is 0.4.

In order to meet the full-flow conditions, the tip projected area has to be ten times the projected area of the push rods (Randolph 2004; Randolph and Anderson 2006; Boylan and Long 2007; Yafrate et al. 2009; DeJong et al. 2010). Therefore, when reducing the tip projected area, the push rods should also be reduced in size (Kelleher and Randolph 2005; Peuchen et al. 2005; DeJong et al. 2010). Therefore, a penetrometer tip that is a 40 mm ball does not constitute a full-flow penetrometer. A penetrometer tip that is a 40 mm ball was adopted for this study for ease of field implementation. As part of the field practice, the top few feet from the surface were augered in order to ensure that the penetrometer was pushed in a vertical direction as much as possible into the ground and also to avoid possible damages to the penetrometer due to the presence of surface rocks or boulders. Therefore, it is easier to use the penetrometer with a 40 mm ball and

augers than that with a 113 mm ball or T-bar. As the ratio of the tip projected area to push rods decreases, the tip resistance increases up to a ratio of 1:1 where there is full displacement as in the case of a cone tip. However, it has been mentioned that there is no significant increase or less than 10% increase between a ratio of 10:1 and 5:1 (Chung and Randolph 2004; Weemees et al. 2006; Yafrate et al. 2007; DeJong et al. 2010). From this study, the tip resistance of the penetrometer with a 40 mm and a 113 mm ball is approximately 44% and 54% respectively less than that recorded with the cone tip. Therefore, the difference between the 10:1 ratio for the penetrometer with the 113 mm ball and 1.26:1 for the penetrometer with the 40 mm ball is approximately 10%. A point to emphasize here is that by using a penetrometer that has a ball for the tip with a slightly larger diameter instead of that with a cone tip (40 mm as opposed to 36 mm), there is approximately a 44% decrease in the tip resistance.

Table 4-4 soil specific values for the 40 mm ball tip

	Without Temperature Correction	With Temperature Correction
	N_{su-40}	N_{su-40}
Average	6.3	8.5
Range	4.6-9.1	7.5-11.4
STDev	1.1	1.1

Table 4-5 soil specific values for the 113 mm ball tip

	Without Temperature Correction	With Temperature Correction
	N_{su}	N_{su}
Average	5.9	6.2
Range	5.1-6.5	5.4-6.9
STDev	0.5	0.5

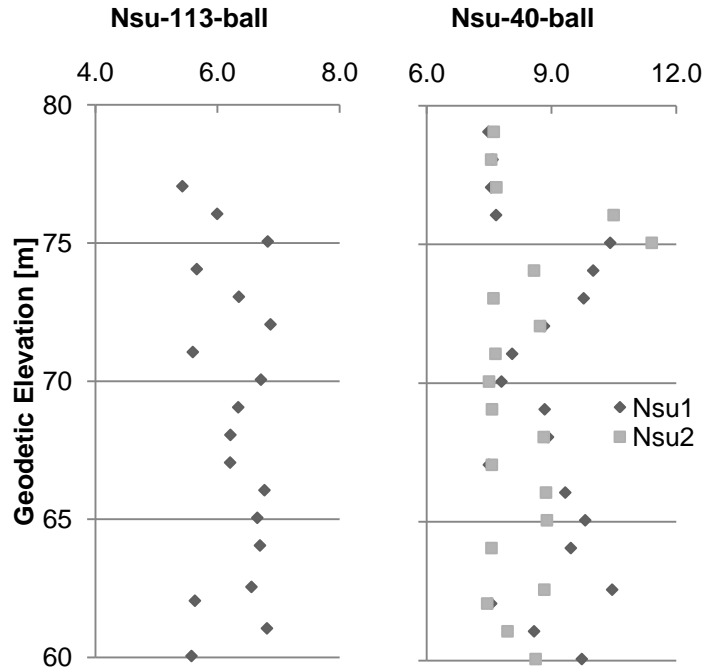


Figure 4-17 (a) ball soil specific factor Nsu-113-ball for 113 mm ball tip with geodetic elevation (b) ball soil specific factor for 40 mm ball tip with geodetic elevation, Nsu1 and Nsu2 are ball soil specific factors for 40 mm ball tests 1 and 2

4.4.2 Sensitivity Estimation

An undrained shear strength profile was obtained from the Nilcon vanes (Figure 4-18). The undrained shear strength starts under a stress of 50 kPa at a geodetic elevation of 79 m, and then drops to 13.4 kPa at a geodetic elevation of 78 m. Then, the undrained shear strength generally increases up to 63 kPa at a geodetic elevation of 60 m. The remolded shear strength generally follows the same pattern. The range of the sensitivity has a constant pattern of increase with depth, which is between 1.9 and 8.0 with an outlier of 23.5 at a geodetic elevation of 78 m. It is worth mentioning that the undrained shear strength ranges between 13 and 63 kPa which is similar to the values mentioned by Bozozuk and Law (1976).

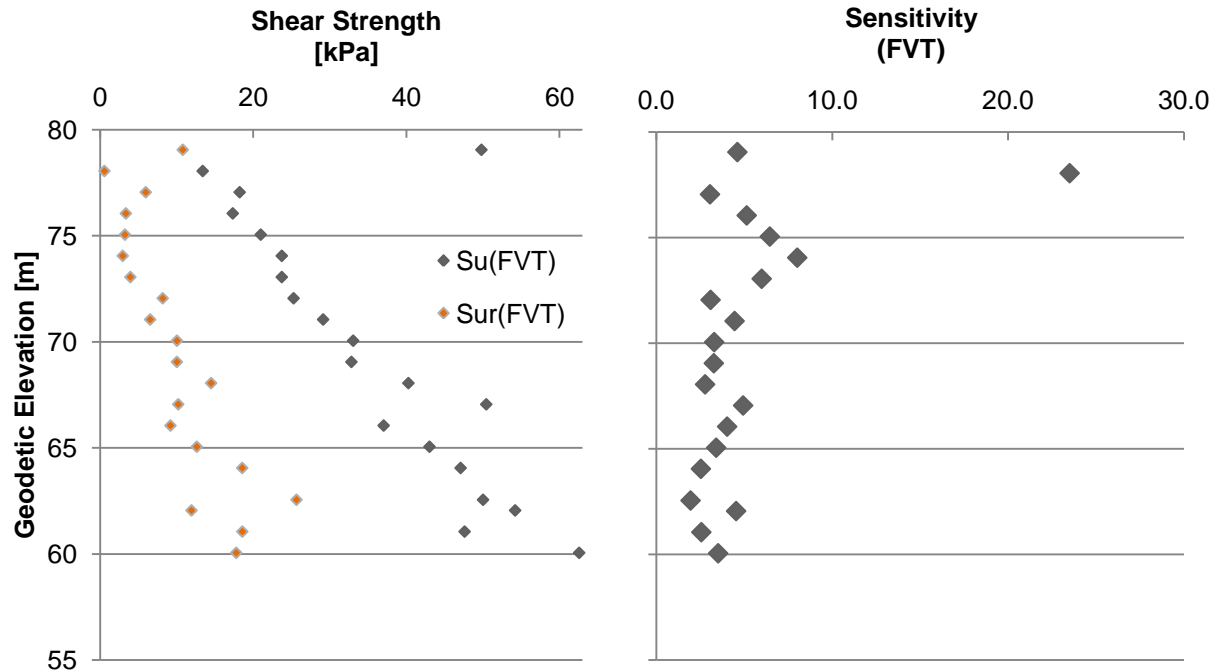


Figure 4-18 (a) undrained shear strength with geodetic elevations from field vane tests (b) sensitivity with geodetic elevations from field vane tests, S_u is the initial undrained shear strength, S_{ur} is the remolded undrained shear strength

As mentioned earlier, remolded shear strength is recognized to be approximately equal to the sleeve friction (Lunne et al. 1997). Therefore, sensitivity can be estimated by using the ratio of the undrained shear strength that was estimated by using the tip resistance (q_c) and the sleeve friction. Sensitivity measured by field vane tests and sensitivity parameters from CPT with respect to geodetic elevation is shown in Figure 4-19(a). There is a relatively good agreement between the values obtained by the field vane tests and the estimated values especially in layers with low sensitivity. The measurements of sensitivity from the field vane tests are plotted against the estimations of sensitivity in Figure 4-19(b). There is good agreement between the values from the field vane tests and the estimated values up to a sensitivity of around 8. Following this value, there is considerably more scatter most likely due to that the method used for the estimation of sensitivity which is not ideal for sensitive marine clays in Ottawa. To verify the scattering observed in the estimation of sensitivity, the tip resistance is plotted against sleeve friction in figure 4-19(c). There is good agreement, as indicated by a straight line, between the tip resistance and sleeve friction up to a value of about 650 kPa for the former and 13 kPa for the latter. Beyond 650 kPa and 13 kPa, the level of scattering considerably increases.

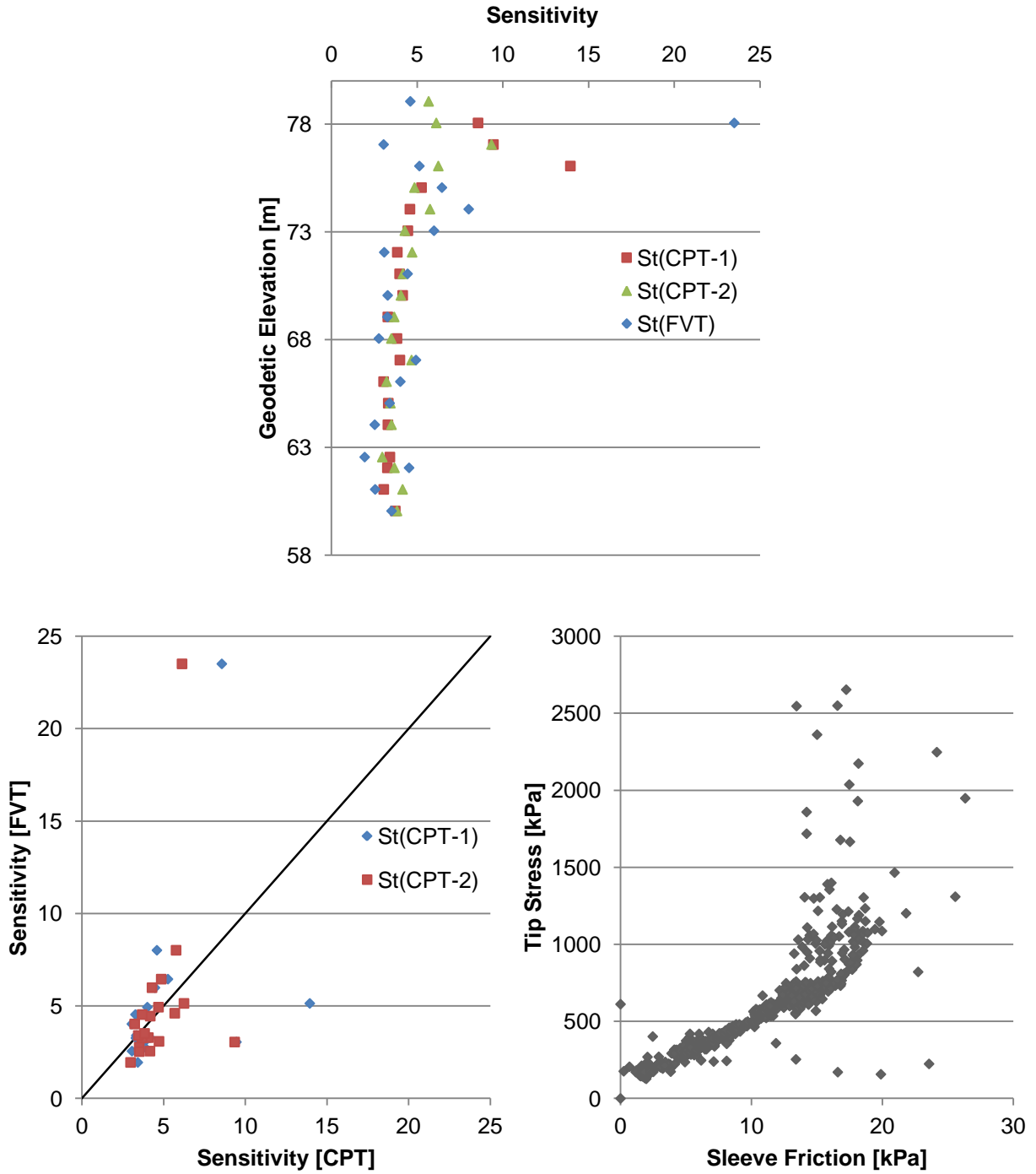


Figure 4-19 (a) sensitivity from cone penetration tests 1 and 2 St(CPT-1) and St(CPT-2) and from field vane tests St(FVT) with geodetic elevations (b) sensitivity from cone penetration test CPT with sensitivity from field vane tests FVT (c) sleeve friction with tip resistances from cone penetration tests

According to Schmertmann (1978), sensitivity can be estimated by using the ratio of N_s and the friction ratio (R_f) in a percentage (Lunne et al. 1997). He suggested an N_s value of 15. Other studies have suggested an N_s value of 6 with reference to vane tests (Robertson and Campanella 1988). Rad and Lunne (1986) found that N_s range between 5 and 10 with an average of 7.5. Lunne et al. (1997) suggested an average value of 7.5 or a range of 6 to 9. However, they claimed that there is no unique value for N_s as it depends on the mineralogy and OCR. They also suggested the calibrating of N_s with local experience, especially with sensitive clays when the sleeve friction readings are low. From this research, the average value of N_s was found to be 7.0, which is similar to the recommended 7.5. However, a relatively large range was encountered, from 2.3 to 15.9, with a standard deviation of 3.0. figure 4-20(a) shows the variation of N_s with geodetic depth for two CPT soundings. It can be noticed that there is a reverse relationship between the plasticity and the sensitivity represented by N_s as shown in a comparison, see Figure 4-20(a) and Figure 4-20(b).

It is worth mentioning that the sensitivity of this site ranges between 2 and 23, unlike the values reported by Yafrate and DeLong (2006) of up to a hundred.

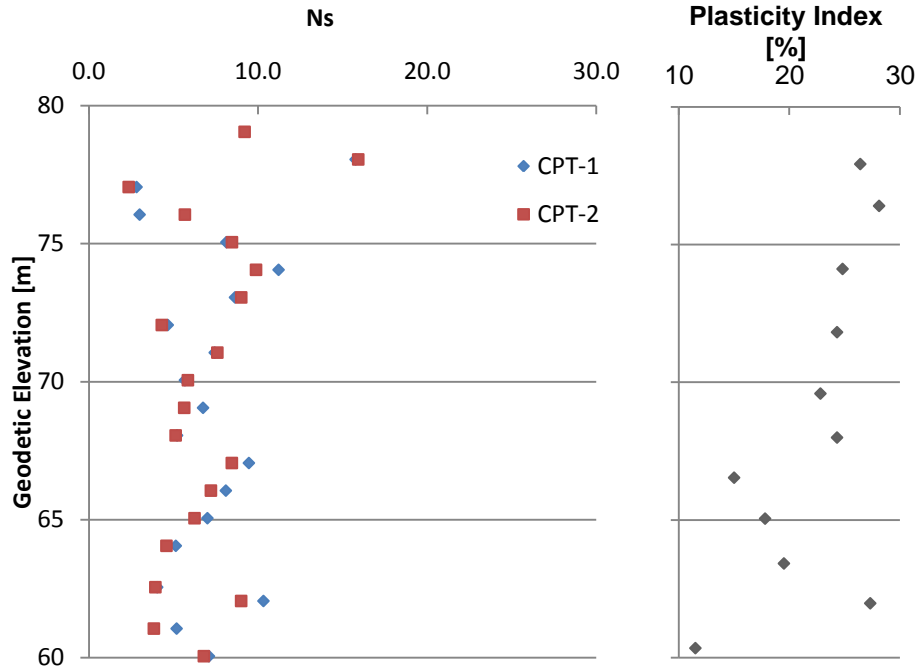


Figure 4-20 (a) sensitivity factor N_s from cone penetration tests 1 and 2 CPT-1 and CPT-2 (b) plasticity index with geodetic elevations for the purpose of comparison

4.4.3 Soil Geotechnical profile

Figure 4-21 shows the soil type by using CPT sounding per Jefferies and Davies (1991) and soil classification by using the Cassagrande plasticity chart (CPC). It can be seen that soil that comprises high plasticity inorganic clay per the CPC is recognized as clay by the CPT. Low to medium plasticity inorganic clay or silty clay is recognized as organic soil. There is a difference in the geodetic elevation of the layers probably due to the interlayers and stratification. The CPT does not appear to recognize the medium to high plasticity organic clay to silty clay.

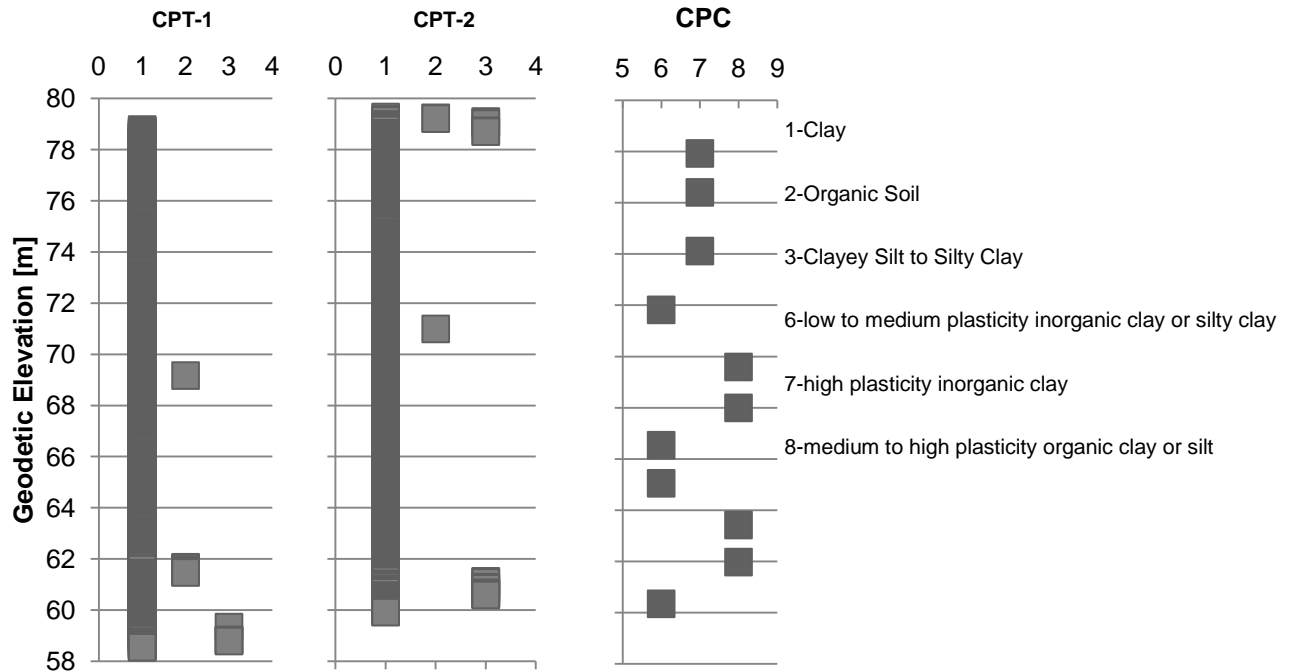


Figure 4-21 (a) soil type classification according to Jefferies and Davies (1991) for cone penetration test 1 with geodetic elevations (b) soil type classification according to Jefferies and Davies (1991) for cone penetration test 2 with geodetic elevations (c) soil type according to Cassagrande's Plasticity Chart CPC from plasticity results with geodetic elevations, horizontal axis is the soil class number or number representing the soil name

According to the fines content (silt and clay portion) suggested by Lunne et al. (1997), Suzuki et al. (1995), and Robertson (1990), fines content from hydrometer testing in the laboratory and cone penetration testing are plotted with geodetic elevation as in Figure 4-22. There is an underestimation in the fines content. The actual fines content is close to 100% while the CPT provides an average of about 60%.

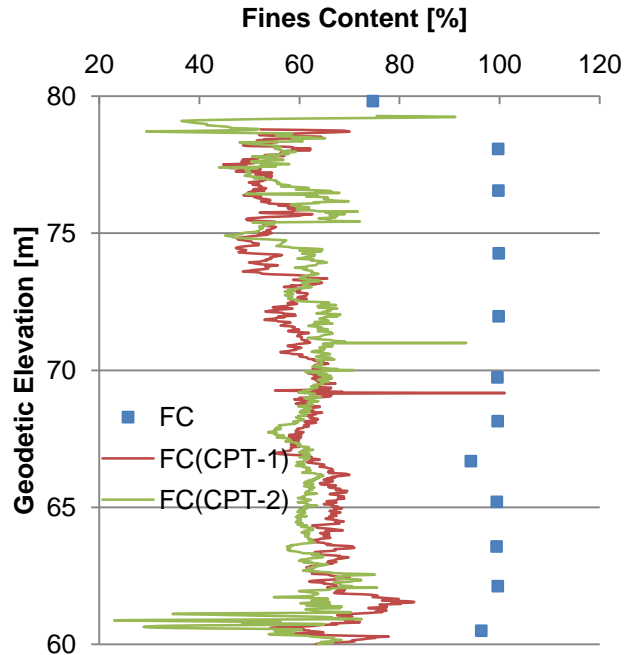


Figure 4-22 fines content according to the method suggested by Lunne et. al. (1997) from cone penetration tests 1 and 2 FC (CPT-1) and FC (CPT-2) and from laboratory testing FC with geodetic elevations

According to the plasticity limits and moisture contents recorded with geodetic elevation, all of the soil profiles have exceeded the liquid limits except at an elevation of 62 m where they are close to the liquid limit. This explains the small tip resistance and sleeve friction recorded in the penetrometer pushes. The plastic limits, liquid limits, and plasticity index are all affected by the varved deposition of sensitive marine clays. As the silt content increases, the plastic limits, liquid limits, and plasticity index decrease, such as at elevations of 60.5 and 66.5 m, see Figure 4-6.

4.4.4 Consolidation History

Lunne et al. (1997) mentioned that if Q_t is within 2.5 to 5, then the soil is normally consolidated. However, if Q_t is greater than this range, then the soil is overconsolidated. From Figure 4-23(a), it can be seen that the soil is lightly overconsolidated at about an elevation of 76 m and above, while Q_t exceeds 5 only when the elevation is greater than 78 m, as shown in Figure 4-23(b). This is probably a result of the interlayers and stratification. An average k value of 0.3 was proposed by Robertson (2000) with a range of 0.2 to 0.5, and higher values are recommended for aged and heavily overconsolidated soils. From this research, an average value of 0.44 was

obtained with a range of 0.21 to 0.56 with outliers of 1.14 and 1.4 for CPT 2 at geodetic elevations of 74.3 and 76.6 m respectively, and a standard deviation of 0.34, see Figure 4-23(c).

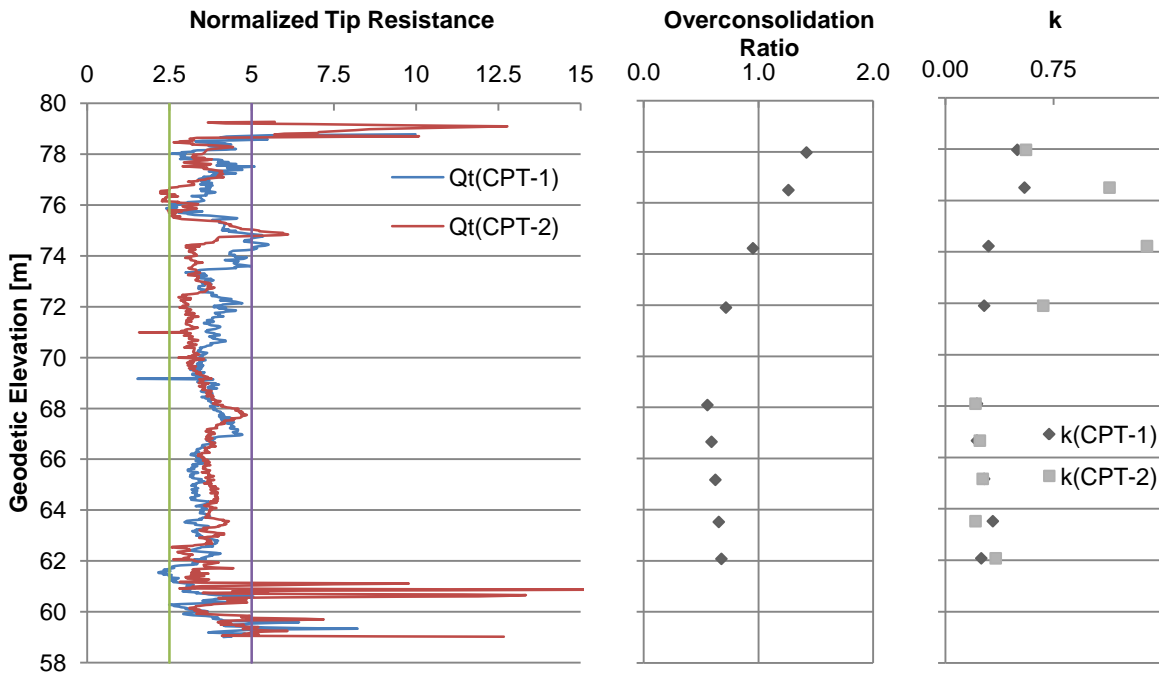


Figure 4-23 (a) normalized tip resistance from cone penetration tests 1 and 2 Q_t (CPT-1) and Q_t (CPT-2) with geodetic elevations (b) overconsolidation ratio from laboratory testing with geodetic elevation for the purpose of comparison (c) k factor for consolidation history from cone penetration tests 1 and 2 k (CPT-1) and k (CPT-2) with geodetic elevations

For sites with available data, the correlation of OCR with other parameters is suggested, such as Q_t and B_q , in order to better estimate the OCR and preconsolidation ratio. However, from this research, the correlation of OCR with Q_t and B_q is scattered and does not follow a clear pattern as shown in Figure 4-24. The reason is that these correlations do not apply well to sensitive marine clays in Ottawa.

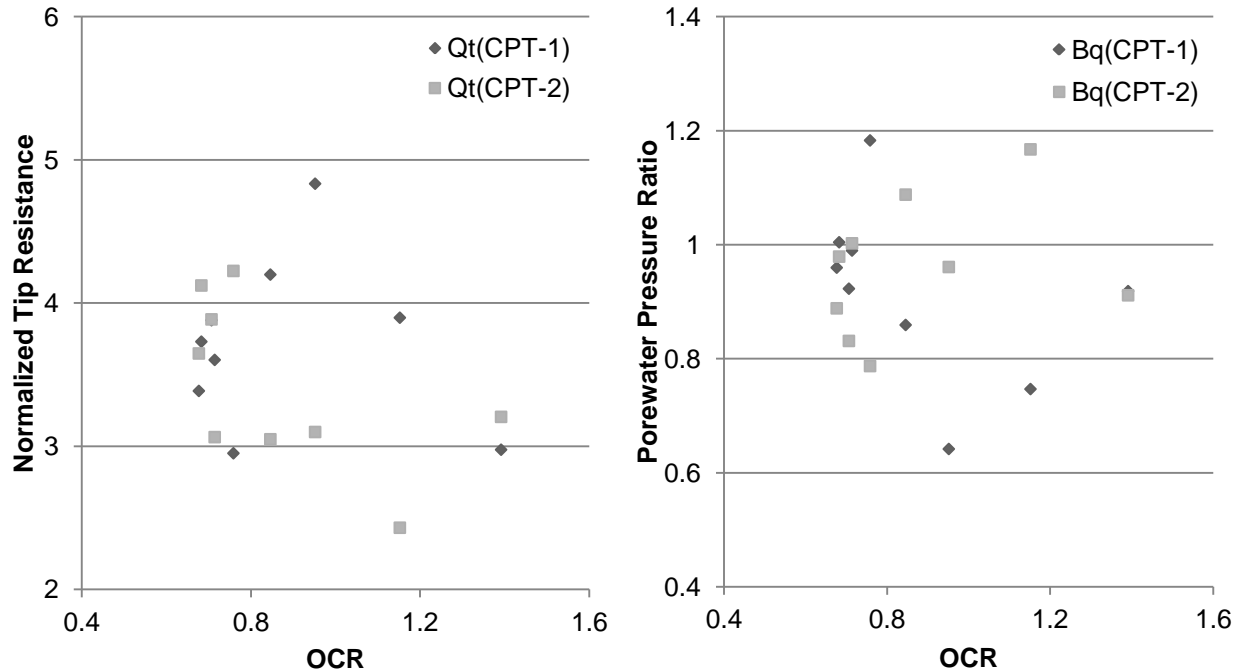


Figure 4-24 overconsolidation ratio from laboratory testing with (a) normalized tip resistance from cone penetration tests 1 and 2 $Q_t(\text{CPT-1})$ and $Q_t(\text{CPT-2})$ (b) porewater pressure ratio from cone penetration tests 1 and 2 $B_q(\text{CPT-1})$ and $B_q(\text{CPT-2})$

Chen and Mayne (1996) studied the relationships between CPTs and the stress history of different types of soils from around the world including Canadian soil. They indicated that correlations of preconsolidation pressure and stress difference such as $q_t - \sigma_{ov}$, Δu_m , and $q_t - u_m$ render better estimations than those that incorporated B_q . They also indicated that the incorporating of I_p may increase accuracy in estimation.

4.5 Summary and conclusions

Sensitive marine clay deposits in Ottawa have been characterized by using penetrometer tests. The Canadian Geotechnical Research Site No. 1 has been the target of researchers as a representative site of sensitive marine clays in Ottawa. Correlations between penetrometer tests and soil engineering characteristics have been developed in the literature. However, it is recommended that these correlations be verified and modified in accordance with local and site specific experiences. Temperature effects on penetrometer readings have also been verified. Samples from cone penetration, ball penetrometer and vane tests, as well as undisturbed samples,

have been collected from the site. A penetrometer with a tip that is a 40 mm ball is pushed into sensitive marine clays for the first time to verify the suitability of this type of tip for the soil. It is used for the ease of implementation in the field. It is found that there is a decrease in the tip stress of approximately 44% when the tip is a ball that is slightly larger than a cone tip.

Typically, the tip stress drops as the tip projected area increases. Laboratory tests are performed on the undisturbed samples as reference for the field data.

In terms of the cone equipment used in this study, it is found that a temperature change of 18°C may cause a gain or loss of 166 kPa and 22 kPa in tip resistance and sleeve friction readings respectively. This effect may result in negative and erroneous readings from penetrometers. A temperature correction factor is established for the specific penetrometers used in this experiment and applied to each tip, sleeve, and pore pressure reading.

Empirical correlations between CPTs and soil engineering properties have been verified for sensitive clays in Ottawa. The correlation of the undrained shear strength with the CPT parameters, N_{kt} , N_k , $N_{\Delta u}$, have average values of 9.4, 7.6, and 8.7 respectively. The correlation factor, N_{Su-40} , of the undrained shear strength based on tests carried out by a penetrometer with a 40 mm ball is 8.5. The correlation factor, N_{Su} , of the undrained shear strength based on tests carried out by a penetrometer with a 113 mm ball is 6.2. It is found that N_{Su} has a smaller range and standard deviation than those of the CPTs and tests carried out by the penetrometer with a 40 mm ball. It is also found that the penetrometer with a 113 mm ball is the least affected by temperature change. The sensitivity has a reduced value of scattering up to 8 where then, the scattering considerably increases. The sensitivity correlation factor, N_s , has an average value that is equal to 7.0. The consolidation history factor, k , has an average value of 0.44. It is recommended that the correlation of the consolidation history be modified with normalized tip resistance and porewater pressure ratio. However, no clear relation is found with the OCR. Grain size profile and soil types from the CPC are compared with the method used in Jefferies and Davies (1991) to determine soil behaviour. It is found that CPC can correctly detect layers with coarser grain sizes, but not organic layers. More specific zones need to be determined for sensitive marine clays in Ottawa. The fines content is also underestimated.

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Chapter 5 Conclusions and Recommendations

5.1 Summary

In this thesis, the engineering characteristics of sensitive marine clays in Ottawa have been assessed. There has been an attempt to provide background and reference information for geotechnical engineers who are working with these soils. The thesis has also provided background about the origin, geological settings, sedimentation, and structure of sensitive marine deposits. Two correlations in the Canadian Foundation Engineering Manual have been examined. The first correlation is between SPTs and undrained shear strength. The second correlation is between shear wave velocity and undrained shear strength, and site class. Values have been compared in terms of the ratio of the secondary compression index to the compression index proposed by previous researchers and this research. In general, this thesis has proposed ranges of values for engineering characteristics based on fifteen sites and those by previous researchers.

This thesis has studied the effect of temperature on cone penetration testing in sensitive marine clays in Ottawa. Correlations between CPTs and the engineering characteristics of these clays are examined. Furthermore, the values of the coefficient of consolidation from oedometer and pore pressure dissipation testing have been examined by using CPTs. The work here has also attempted to increase local experience on the use of ball penetrometers in these clays.

5.2 Conclusions and Future Research Recommendations

The following conclusions and recommendations are made based on this study.

(i) Undrained Shear Strength and SPT Correlation: The Canadian Foundation Engineering Manual presented a correlation between the N_{60} value from SPTs and the undrained shear strength in Table 3.3 in the manual. According to the research in this thesis, the undrained shear strength estimated from SPTs in fourteen sites that are well spread over the capital region is underestimated compared to the undrained shear strength resultant of vane tests.

(ii) Site Class, Shear Wave Velocity, and Undrained Shear Strength Correlation: The Canadian Foundation Engineering Manual also presented a correlation between the shear wave

velocity and site class, and the undrained shear strength and site class. Although the recommended depth for site classification has not been reached in the available data sets, the correlation is examined for specific depths. It is found that the site class determined from the shear wave velocity is approximately one class lower than that determined from the undrained shear strength which again indicates an underestimation in the undrained shear strength of sensitive marine clays in Ottawa. Data sets used to examine this correlation are only from three sites. Therefore, the conclusion is not confirmative. Future research may be initiated for a larger number of sites to confirm the correlation or propose new boundaries for site classes estimated from the undrained shear strength.

(iii) Remolded Shear Strength from Nilcon Vanes: From this research, difficulties in determining the remolded shear strength from FVTs are encountered. To address this issue, electronic or Nilcon vanes were used. It has been observed that the soil may continuously gain back partial strength during testing. In other words, the graph for the undrained shear strength versus the degree of vane rotation may continue to show increases even after exceeding 180 degrees of rotation. Furthermore, at deep testing, the issue may be present in rod friction. So, soil around the push rods may gain back some strength which is the dominant factor in the misestimation of the remolded shear strength. ASTM D2573 has indicated that the rate of rotation for soft soils may be increased to the maximum allowed. Therefore, future research should consider the evaluation of the applicability of the rotation rate of the vanes or propose a higher rate or rotation.

(iv) Laboratory and Field Vane Results: Laboratory vanes are compared to field vanes in this research. For Site 6 or Canadian Geotechnical Research Site No. 1, miniature laboratory vane results are corrected for the rotation rate in accordance with Landon-Maynard et al. (2011) and for plasticity in accordance with Ladd (1975) and Larsson (1980). The laboratory vane results show mostly an underestimation of the undrained shear strength compared to the field vane results except for a few of the depths where there are overestimations. With reference to Sites 3 and 5 where laboratory and field vanes were available, the former also underestimates the undrained shear strength of sensitive marine clays in Ottawa. Due to the limited number of sites with available data on this issue, the data are not confirmative. Future research should consider

better correction factors for these clays or the use of the same rate of rotation for both lab and field vanes to obtain a closer estimation of the undrained shear strength.

(v) Oedometer Testing Loading Scheme: In terms of the oedometer testing in this research, double and random loading schemes are used in different occasions. It is found that the double loading scheme renders curves closer to the theoretical curves from the available methods for the determination of the coefficient of consolidation. Therefore, this may increase the accuracy and ease the process in the determination of this coefficient. However, due to the sudden and sharp break of sensitive marine clays in Ottawa near the preconsolidation pressure in consolidation testing, a double loading scheme may reduce the accuracy of the determination of the preconsolidation pressure thus creating difficulties. Therefore, it is recommended that a random loading scheme be used if determining the preconsolidation pressure is more critical than determining the coefficient of consolidation. However, it is recommended that a double loading scheme be used if determining the preconsolidation pressure is less critical than determining the coefficient of consolidation.

(vi) Secondary Compression Index to Compression Index Ratio: A ratio of the secondary compression index to the compression index has been determined in this research. The range of values suggested is higher than the values suggested by Holtz and Kovacs (1981) in general for marine clays in Canada. A ratio of the compression index to the recompression index range has also been suggested.

(vii) Penetrometer Temperature Corrections: It is found that temperature changes may cause a shift in the readings of the penetrometer equipment. This shift may be significant given the radical weather changes in Canada, including Ottawa, and the stability of ground temperatures below a depth of 6 m according to Williams and Gold (1976). Shifts in penetrometer readings could result in negative readings because sensitive marine clays in Ottawa may exhibit low tip resistance and sleeve friction. Erroneous penetrometer readings may still be acceptable due to the high tolerance of errors in the ASTM D5778. It is suggested that corrections for penetrometer readings (tip resistance, sleeve friction, porewater pressure) be implemented depending on temperature changes. The temperature compensations are found to merit the results. It is also found that temperature effects on the tip resistance decrease as the tip projected area increases. Therefore, the cone tip is the most affected by temperature changes, the 40 mm ball tip is less

affected than the cone tip, and the 113 mm ball tip is the least affected by temperature changes. Temperature corrections are obtained by using laboratory calibration. Due to the limitations of the cooling and heating equipment, there was no control over the rate of temperature change. Future research should use a controlled heating and cooling environment in order to apply temperature corrections that are closer to the field condition in the rate of temperature change during penetration. The rate of temperature change may be different depending on the time of the year or ambient temperature and soil type; whether it has high or low thermal conductivity. Future research may also consider a variety of soil types and penetrometer equipment.

(viii) Penetrometers Tip Type: Tip resistance shape and the projected area may significantly affect the tip resistance measured. The 40 mm ball tip has 44% less resistance than the 36 mm cone tip even though the former is only approximately 4 mm greater in diameter. Furthermore, the 113 mm ball tip is only 10% lower in tip resistance than the 40 mm ball tip even though there is a 73 mm difference in diameter.

(ix) Penetrometer Empirical Correlations: Sensitivity has been compared from field vanes and a correlation suggested by Lunne et al. (1997), who suggested that the sensitivity can be estimated by using the ratio of the undrained shear strength estimated through the use of the tip resistance (q_c) and the sleeve friction. It is found from the testing carried out at Site 6 that this correlation gives a good approximation up to sensitivity of 8.

Another correlation has been examined which estimated the soil type or soil class. The correlation was suggested by Jefferies and Davies (1991) and compared to a soil classification by using Cassagrande's plasticity chart. It is found from the testing carried out at Site 6 that this correlation is not accurate in detecting fine material layers.

Another correlation has been examined which is to estimate the fines content in the soil mixture. The correlation was suggested by Lunne et al. (1997), Suzuki et al. (1995), and Robertson (1990). The correlation has been compared by hydrometer testing. It is found from testing carried out at Site 6 that this correlation gives an underestimation of the fines content.

Due to the limitation of available data sets or sites, none of the aforementioned is affirmative. Future research is encouraged to take on a number of sites and soil types.

(x) **Implementation Practice Guide for CPTs for Stantec Consulting Limited:** An engineering practice guide for CPTs has been developed for Stantec Consulting Limited (see Appendix #1).

5.3 References

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6 Appendixes

6.1 Appendix #1: Implementation Practice Guide for CPTs for Stantec Consulting Limited

6.1.1 Cone Penetration Test Equipment

The CPT equipment at Stantec Consulting Limited is the portable type that can be used with hydraulic, mechanical, or any other means of push methodology. However, the push machine should be equipped with a push rate control. Stantec have been using the same drill rigs that are used for SPTs to push the cones. The equipment parts include:

1. The probe: a shaft that is approximately 1 m with a cone tip diameter of 36 mm at the front end and 40 mm diameter at the back end,
2. Push rods: 1 m in length and 36 mm in diameter,
3. Computer: runs the test software at the time of the testing. The computer is also used as a display screen at the time of the testing and storage for the data when the testing is completed,
4. Seismic trigger: fixed onto a hammer or a wood log,
5. Depth marker or transducer: 10 cm x 10 cm box fixed onto the drill rig at the time of the testing to measure the depth of the push,
6. Data acquisition system (DAS) box: connects the CPT equipment to the lap top computer,
7. Connection wires: 3 wires that connect the probe, depth marker, and seismic trigger to the DAS, 2 wires that connect the DAS to the computer, and 2 power wires for the DAS and the computer, and
8. Two seismic wood logs that are 1 m in length and have cross sections of 20 cm x 20 cm.

6.1.2 Field Implementation

All of the equipment should be loaded onto a truck or a cargo van prior to the test date. The following steps should be followed.

1. The probe, depth marker, seismic trigger, and lap top computer are connected to the DAS. The DAS and computer are run, and the DAS is run on the computer and left running for a minimum of half an hour. It is recommended by ASTM D 5778 to warm up the equipment circuitry for 15 to 30 minutes to avoid electronic or electrical shifts in the readings. However, a new recommended practice has indicated 30 minutes to 1 hour of circuitry warm up (DeJong et al. 2010).
2. The technician is to hold the probe and point the probe downwards as vertical as possible off the ground without touching anything around the probe and take the baseline reading. If the probe tip or sleeve is loaded in any way, erroneous readings will be obtained.
3. The depth marker box is fixed onto a safe place on the drill rig. The drill operator brings the rig up and down to a full 1 meter in distance to make sure that the rig does not crush the box or go beyond the maximum wire length of the marker. The push limits are marked on the drill rig. Often, the drill operator crushes the depth marker box or snaps its wires, thus causing testing delays. The operators should be advised that the connection wires are delicate and treated with care and minimum bending.
4. The cone tip should be tightened with a pipe wrench, a 2 m push is to be performed with no connections, and the cone tip retightened or ensured that it is tight and in place. It is also recommended that the 2 m push is performed twice to ensure that the cone tip is actually tight. It has been found that the cone tip may come loose during the push thus resulting in negative or erroneous readings in the tip resistance and sleeve friction. This 2 m push will also reduce the temperature effects due to temperature differences between the ground surface and approximately 6 m in the ground depth.
5. During the first two “test pushes”, an understanding should be developed on the contents of the top layer and its depth by examining the auger sides during drilling. If the top soil is hard (such as fill or aggregate), then the top layer is augured through to a softer layer at its bottom. The top hard layer may not necessarily break the cone, but may cause excessive inclination in the push that would carry on for the rest of the push. Then it should be determined if the push is performed at the surface or a specific depth and the depth is recorded. A sample of the top layer may be obtained for lab testing if necessary.
6. As the rods are 1 m in length, the operator will be waiting for a signal prior to each rod push. The “Run” button is then pressed and operator should be signaled to start the push.

The rate of the push on the screen is monitored and the operator is to be instructed to adjust the push rate as deemed necessary. The “Pause” button is pushed before the operator lifts the drill rig to avoid inconsistency in the test readings due to unloading effects and upward depth readings.

7. If refusal is encountered, there is the possibility of extensive inclination. Extensive loading in bedrock for example will result in extensive inclination that may cause extensive bending in the push rods especially at great depths, thus causing damage to the rods and breaking off of one of the rods, which will leave the probe underground. Depending on the soils encountered during the push, refusal can be determined. Refusal is high tip resistance or extensive inclination, whichever comes first. The maximum tip resistance should be obtained for the cone from the user manual.
8. The machine is paused while pulling the push rods up. It is recommended that the depth marker box be removed from the drill rig at this point. After the probe is pulled out, it is cleaned, and any material that may be on the probe is also cleaned off, which may cause loading on the tip or the sleeve. The technician is to hold the probe and point the probe downwards as vertical as possible off the ground without touching anything around the probe and take the final baseline reading.
9. If seismic testing is required, the operator should install the seismic logging equipment beneath the legs of the drill rig so that it is well positioned into the ground due to the weight of the drill rig. The logging equipment is to be aligned parallel to the cone geophones. For Stantec CPTs, the ends of the seismic logs need to point towards the hole. The cone has two geophones aligned 90 degrees to each other. So, by having two seismic logs that point to the hole at approximately 90 degrees to each other, at least one log axis will be directly pointed to a geophone axis. It is recommended that the pusher of the drill be removed from the push rods to avoid noise due to transfer of energy from the drill rig to the push rods and then to the geophones. It is also recommended that the drill rig is turned off during seismic testing to reduce signal noise. To determine the arrival of the shear wave, the seismic logs should be hit from the sides with a medium power hit. To determine the arrival of the compression wave, hit the top middle side logs with medium intensity. High intensity hits may result in noisy signals and soft hits may result in weak signals. Therefore, it is important to find the balance of the hit strength. Multiple hitting

is recommended at the same depth to obtain the best signal possible. The gain is started at 10 near the surface and increased as necessary with depth. Measurements are taken on the orientation of the seismic logs relative to the hole. The x and y distances are measured from the centres of the seismic logs to the centre of the hole.

10. If pore pressure dissipation testing is required, as the required depth is reached, the dissipation button on the screen is pressed as soon as the operator ends the push interval. The drill rig pusher should not be lifted from the push rods as this will result in unloading the cone and reducing the excess porewater pressure. Error is minimal if the time between the stop of the push and the start of the test is minimal. The maximum porewater pressure is recorded at the start of the testing and the testing is stopped when the pressure is lower than 50% than that at the beginning of the testing.

6.1.3 Data Reduction and Interpretation

Three types of information are obtained from CPT equipment, which are the CPT down hole push (tip resistance, sleeve friction, porewater pressure, and temperature), seismic profile (shear and compression waves), and pore pressure dissipation.

6.1.3.1 CPT

A DAT file is obtained from the CPT software and stored in the computer. The file can be opened with the notepad program. Software called “ProDAT” is available to process the data. After processing the file by using this software, an Excel file with the following information is obtained: depth, sleeve friction, tip resistance, corrected tip resistance, over consolidation ratio, reconsolidation pressure, density, undrained shear strength, overburden pressure, soil class, standardized blow count for SPT, friction angle, and inclination. However, only the actual push information is used, which are the depth, tip resistance, sleeve friction, and porewater pressure. The reason that the other estimated soil properties are not used is that the software assumes theoretical correlation factors which cannot be amended. It is found that these theoretical correlation factors vary from one soil type to another and exist in ranges that are not constant in value. Therefore, a tailor made Excel sheet with correlations for the engineering soil properties needed is adopted. According to the ASTM, the difference between the initial and final baselines should not exceed 2% of the maximum load that the cell can measure. In other words, the

difference between the initial and the final tip resistances, sleeve frictions, and porewater pressure readings should not exceed 2% of the maximum load that the tip, sleeve, and porewater load cells can measure. This information is found in the user manual for the cone. An easy way to check for the reliability of the data is to process the data once with the initial baseline and again with the final baseline, then obtain the difference between the processed data. As shown in Figure 6-1, the initial baseline is marked with a red rectangle. There is also a red rectangle on a random reading line in the same sequence as that of the baseline reading. The final baseline is the last line in the file.

Channel	0	1	2	3	4	5	6	No
Sensor	Not Connected	Not Connected	Not Connected	P Wave	S Wave 1	S Wave 2	Not Connected	
Range	10.000000	1.000000	1.000000	10.000000	10.000000	10.000000	1.000000	
Gain	1	1	1	40	40	40	1	
Filter	1	1	1	1	1	1	1	
Excitation	1	1	1	1	1	1	1	
Baseline	7.244984E-002	5.384909E-002	4.248112E-002	-2.914473E-003	-2.548257E-003	-2.731365E-003	-4.449531E-002	3.
Cal Factor	0.000000E+000	0.000000E+000	0.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	0.000000E+000	0.
Cal Factor	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.
Cal Factor	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.
Cal Factor	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.
Cal Factor	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000	0.
Unit Factors	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.
Unit Factors	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.
Unit Factors	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.
Unit Factors	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.
Unit Factors	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000	1.
Dig. Sensor	Rod Depth	Digital NC	Digital NC	Digital NC				
+7.6295109483E-002	+5.8442053864E-002	+4.9744411383E-002	-2.5940337224E-003	-1.3733119707E-003	+7.6295109483E-004	-4.2420080873E-000		
+7.5837338827E-002	+5.7984283207E-002	+4.7150377661E-002	-2.8992141604E-003	-3.3569848173E-003	+4.5777065690E-004	-4.1657129778E-000		
+6.9428549630E-002	+5.1270313573E-002	+3.9978637369E-002	-2.5940337224E-003	-2.2888532845E-003	-5.6458381018E-003	-4.5777065690E-000		
+7.2022583352E-002	+5.4169527733E-002	+4.3640802625E-002	-3.0518043793E-003	-2.1362630655E-003	-3.3569848173E-003	-4.5319295033E-000		
+7.1259632258E-002	+5.3406576638E-002	+4.3793392844E-002	-1.9836728466E-003	-9.1554131380E-004	-3.5095750362E-003	-4.5624475471E-000		
+7.2480354009E-002	+5.4169527733E-002	+4.4098573281E-002	-2.7466239414E-003	-1.3733119707E-003	-2.8992141604E-003	-4.3945983062E-000		
+7.1564812696E-002	+5.3406576638E-002	+4.2725261311E-002	-2.4414435035E-003	-3.0518043793E-003	-3.3569848173E-003	-4.6845197223E-000		
+7.2175173571E-002	+5.4016937514E-002	+4.4403753719E-002	-4.1199359121E-003	-2.1362630655E-003	-2.4414435035E-003	-4.5929655909E-000		
+7.3701075761E-002	+5.5695429923E-002	+4.5777065690E-002	-2.7466239414E-003	-2.5940337224E-003	-2.5940337224E-003	-4.4098573281E-000		
+7.2938124666E-002	+5.5237659266E-002	+4.5319295033E-002	-4.1199359121E-003	-2.4414435035E-003	-2.8992141604E-003	-4.5014114595E-000		
+7.3090714885E-002	+5.4932478828E-002	+4.3945983062E-002	-1.8310826276E-003	-2.4414435035E-003	-2.1362630655E-003	-4.5014114595E-000		
+7.1717402914E-002	+5.4016937514E-002	+4.3793392844E-002	-2.4414435035E-003	-2.2888532845E-003	-2.2888532845E-003	-4.5929655909E-000		
+7.3243305104E-002	+5.5542839704E-002	+4.5319295033E-002	-3.2043945983E-003	-2.5940337224E-003	-3.0518043793E-003	-4.2420080873E-000		
+7.2022583352E-002	+5.4322117952E-002	+4.4861524376E-002	-2.5940337224E-003	-2.5940337224E-003	-3.0518043793E-003	-4.4708934157E-000		

Figure 6-1 Baseline and test readings in mV in the data file obtained from DAS for Stantec CPT equipment

6.1.3.2 Seismic Profile

A data file that can be opened with the notepad program is obtained from seismic tests. The file is imported into Excel and the spacing is set as the delimitation between the cells of the worksheet. The information required from the data file is the depth and duration of the test, and amplitude as shown in Figure 6-2. The actual depth of the test start is added if the push was started at a lower grade than the ground surface. The depth column is plotted versus the shear or the compression wave. In geotechnical practices, the arrival time of the shear wave for site classification is the most commonly sought aspect, as shown in Figure 6-3. The Canadian Foundation Engineering Manual classifies sites depending on the travel speed of the shear wave

in the soil. The time of the arrival of the shear wave is estimated from the graph. The straight distance from the centre of the log to the geophone (the cone depth) is calculated. The shear wave velocity is the distance divided by the time of arrival. To obtain the wave speed at each layer separate from the other layers, the following equation can be implemented:

$$V_{s2} = (d_2 - d_1) / (t_2 - t_1)$$

where d_1 and d_2 are the distances from the axis of the log to the geophone for depths 1 and 2 respectively

t_1 and t_2 are the times of the wave arrivals to the geophone for depths 1 and 2 respectively

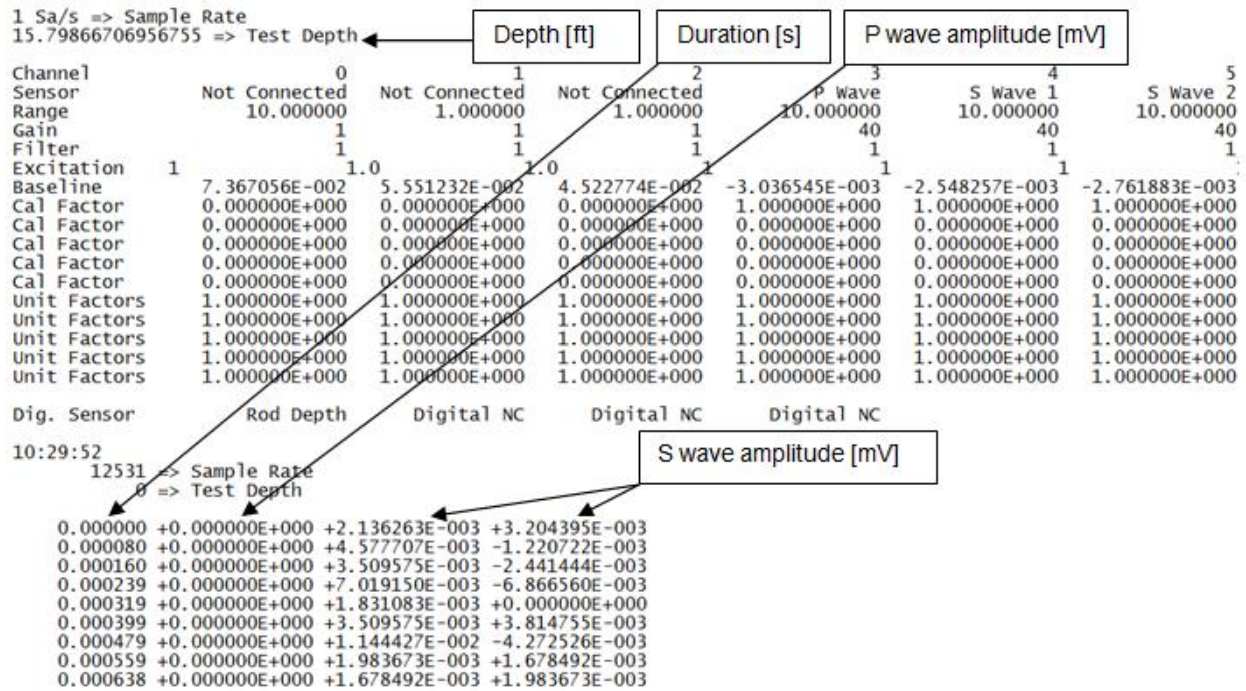


Figure 6-2 Data file for seismic testing obtained from Stantec CPT equipment, and shows where the test depth, duration, and amplitudes can be found

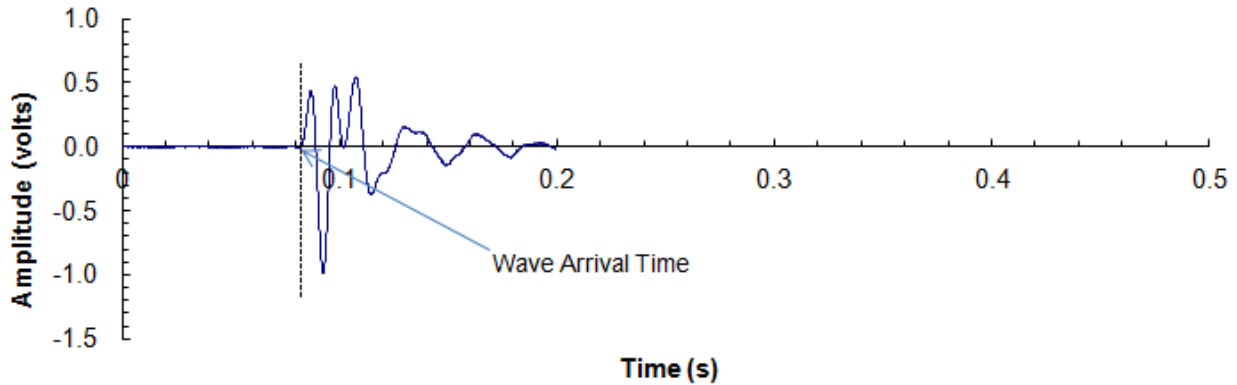


Figure 6-3 Plot of wave amplitude versus time, and shows the estimated arrival time of the shear wave

6.1.3.3 Dissipation Test

A data file that can be opened with the notepad program is obtained from the seismic tests. The file is imported into Excel and the spacing is set as the delimitation between the cells of the worksheet. The information required from the data file are the depth and duration of the test, and porewater pressure, as shown in Figure 6-4. The actual depth of the test start is added if the push was started at a lower grade than that of the ground surface. The porewater pressure column is plotted versus the time column as shown in Figure 6-5. The porewater pressure from the file is in psi and it is not necessary to convert this unit as the time to reach 50% of the porewater pressure is more important and time is in seconds. The time required to reach 50% of the porewater pressure is directly determined from the plot which is the t_{50} . To check the accuracy of the results, t_{50} is approximately the time required to dissipate 50% of the excess porewater pressure. Therefore, if the water table depth is known, t_{50} is the time required to dissipate 50% of the difference in porewater pressure between that at the start of the testing and the hydrostatic porewater pressure. Figure 6-6 shows the approximation of c_h from t_{50} in accordance with Robertson et al. (1990). Therefore, the coefficient of consolidation (c_v) that is used to determine the settlement rate can be determined in accordance with Jamiolkowski et al. (1985). It can be observed that the denominator ranges from 1 to 1.5.

$$c_v = c_h / (1 \text{ to } 1.5)$$

1 Sa/s => Sample Rate
44.16964141821113 => Test Depth

	0	1	2	3
Channel	0	1	2	3
Sensor	Not Connected	Not Connected	Not Connected	Not Connected
Range	1.000000	1.000000	1.000000	1.000000
Gain	1	1	1	1
Filter	1	1	1	1
Excitation	1.0	1.0	1.0	1.0
Baseline	1.046891E+000	1.025147E+000	1.018418E+000	9.796445E-001
Cal Factor	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000
Cal Factor	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000
Cal Factor	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000
Cal Factor	0.000000E+000	0.000000E+000	0.000000E+000	0.000000E+000
Unit Factors	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000
Unit Factors	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000
Unit Factors	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000
Unit Factors	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000
Unit Factors	1.000000E+000	1.000000E+000	1.000000E+000	1.000000E+000
Dig. Sensor		Digital NC	Digital NC	Digital NC
Rod Depth				
	0.377 +8.586818E+001			
	0.703 +8.586818E+001			
	1.110 +8.575709E+001			
	1.699 +8.559047E+001			
	2.289 +8.553493E+001			
	2.609 +8.547939E+001			
	3.000 +8.542385E+001			
	3.406 +8.531250E+001			
	3.796 +8.525696E+001			
	4.109 +8.514588E+001			
	4.500 +8.509034E+001			
	4.906 +8.486791E+001			
	5.297 +8.486791E+001			
	5.703 +8.486791E+001			

Figure 6-4 Data file for pore pressure dissipation testing obtained from Stantec CPT equipment, which shows where the depth and duration of the tests, and porewater pressure can be found

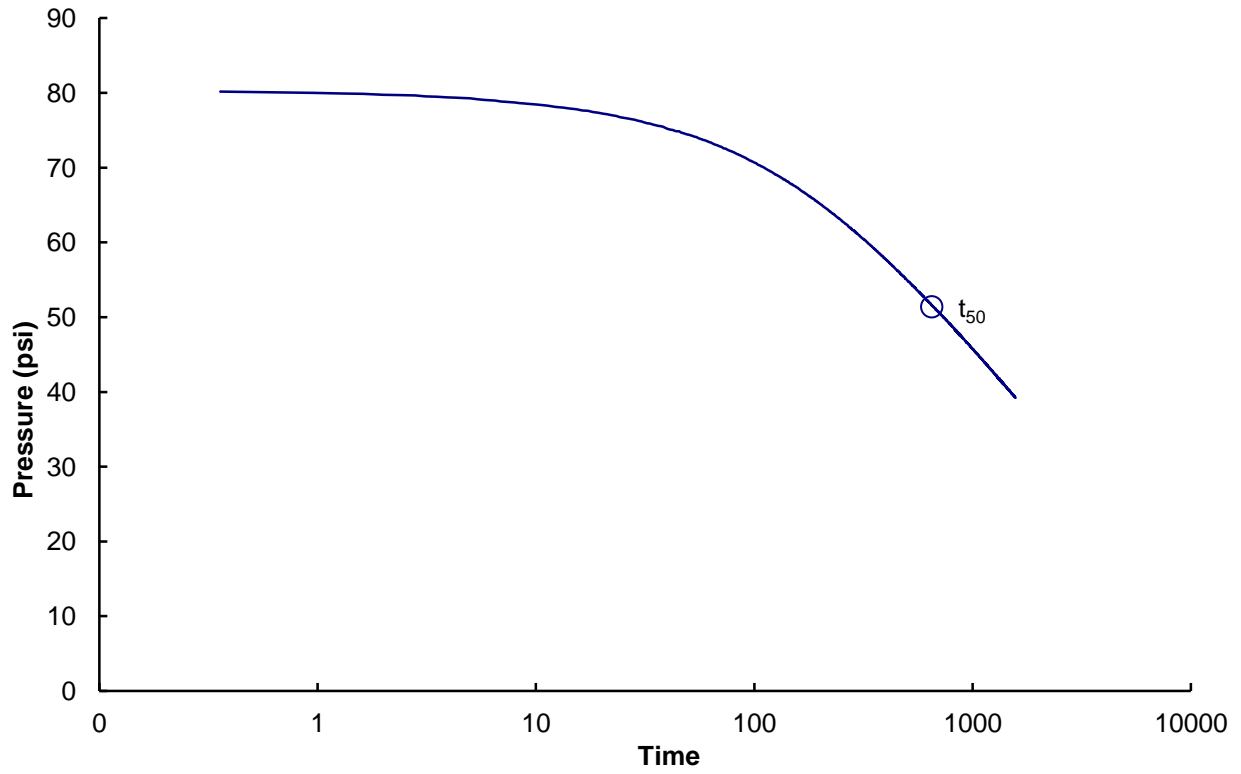


Figure 6-5 Example of t_{50} determination from porewater pressure versus time

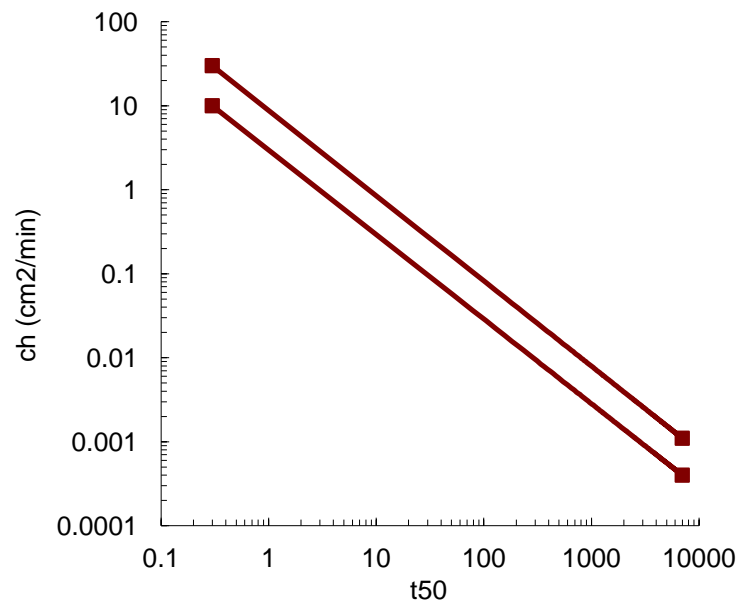


Figure 6-6 Example of determination of c_h from t_{50} by Robertson et al. (1990)

6.1.4 References

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6.2 Appendix #2: Field Photos for Canadian Geotechnical Research Site No. 1



Figure 6-7 (a) The main researcher by site sign with drill rig and CPT van in background (b) Points surveyed by using GPS (c) Site gate and site conditions prior to testing



Figure 6-8 (a) The main researcher inside penetrometer van with penetrometer data acquisition system and push rods (b) Cone tip attached to probe (c) 40 mm ball tip attached to probe



Figure 6-9 (a) 40 mm ball tip attached to penitrometer probe (b) 113 mm ball tip attached to probe (c) Main researcher with 113 mm ball tip attached to penitrometer probe



Figure 6-10 (a) Penetrometer push rods being pulled out of the ground after completion of testing
(b) Penetrometer depth transducer box mounted onto drill rig to measure the push depth (c)
Different sizes of field (Nilcon) vanes



Figure 6-11 (a) Medium size vane attached to the vane push rod (b) Electronic vane motor mounted on drilling augers and clipped to the vane push rods for testing (c) Drilling augers



Figure 6-12 (a) Piston sampler attached drill rig for undisturbed soil sampling by using shelby tubes (b) Shelby tube attached to piston sampler with bags of sand and bentonite for sealing around piezometer (c) Shelby tube with sample after just being pulled out from the ground (d) Main researcher with undisturbed sample in a shelby tube



Figure 6-13 (a) Shelby tube being waxed and capped at the ends to prevent moisture transfer (b) Flowing sensitive marine clay in Ottawa when disturbed (c) Screen attached at the end of tube which is acting as a piezometer to measure the hydrostatic water pressure at different levels (d) Piezometer tube being dropped into the ground to measure the hydrostatic water pressure

6.3 Appendix #3: Lab Graphs for Canadian Geotechnical Research Site No. 1

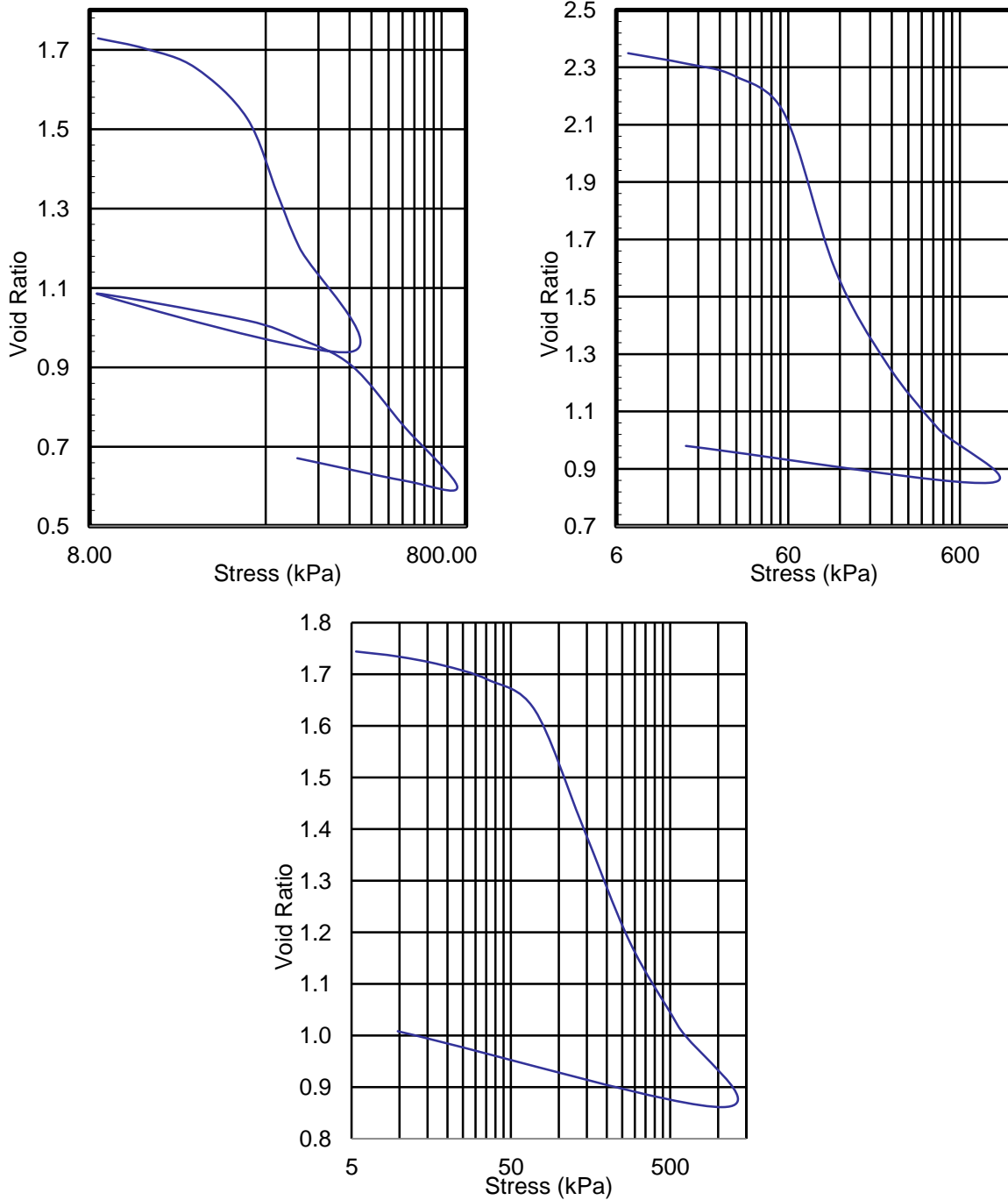


Figure 6-14 (a) Consolidation graph at depth of 2.3 m (b) Consolidation graph at depth of 3.8 m
(c) Consolidation graph at depth of 16.8 m

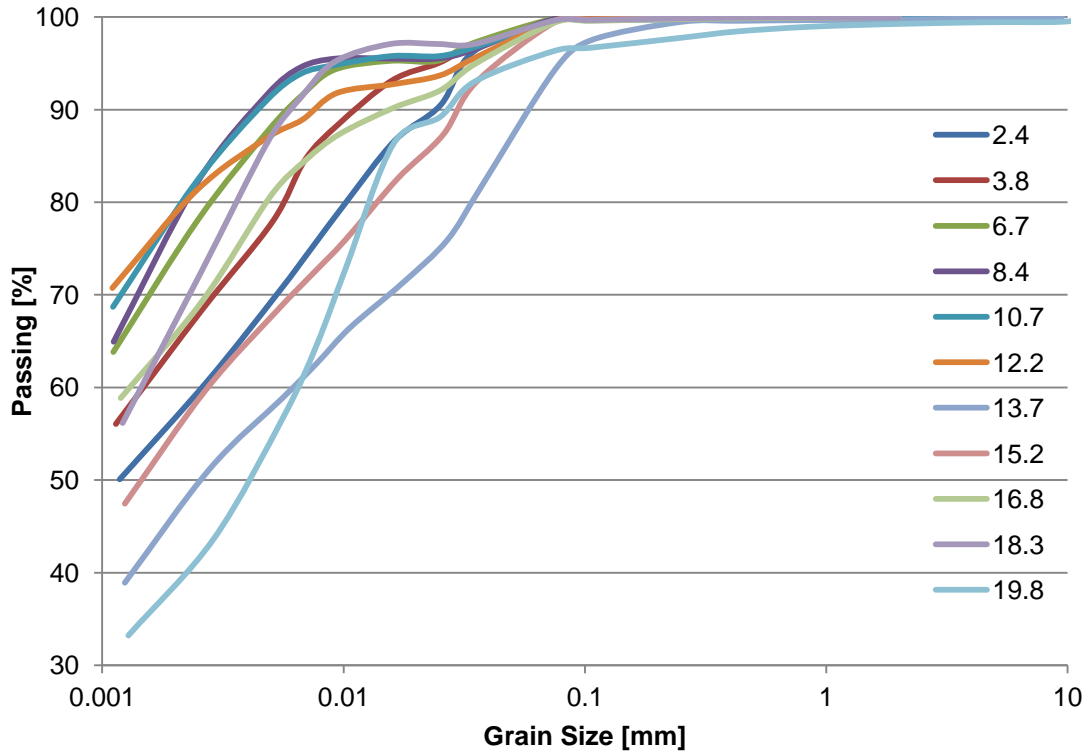


Figure 6-15 Grain size distribution for sample depths of 2.4 to 19.8 m

6.4 Appendix #4: Coefficient of Consolidation from Consolidation and CPT Dissipation Testing

6.4.1 C_v from Taylor's Square Root of Time Fitting Method

According to the Taylor's square root of time fitting method, the dial reading versus the square root of time is assumed to be a straight line for 60% of the consolidation or greater (Holtz and Kovacs 1981). It was observed that for many oedometer test loading intervals in the sites under study, there are curves that do not fit the theoretical curve anticipated by the Taylor's square root of time fitting method. This indicates that there is an unclear borderline between the primary and secondary consolidations. Figure 6-16 shows examples of two curves that do and do not meet the criteria of the Taylor's square root of time fitting method. It was observed that by following a double scheme loading for the oedometer testing, this helps to determine the t_{90} by obtaining a curve that follows the theoretical curve assumed by the Taylor's square root of time fitting method. Therefore, a double loading scheme is recommended when the determination of the coefficient of consolidation is required.

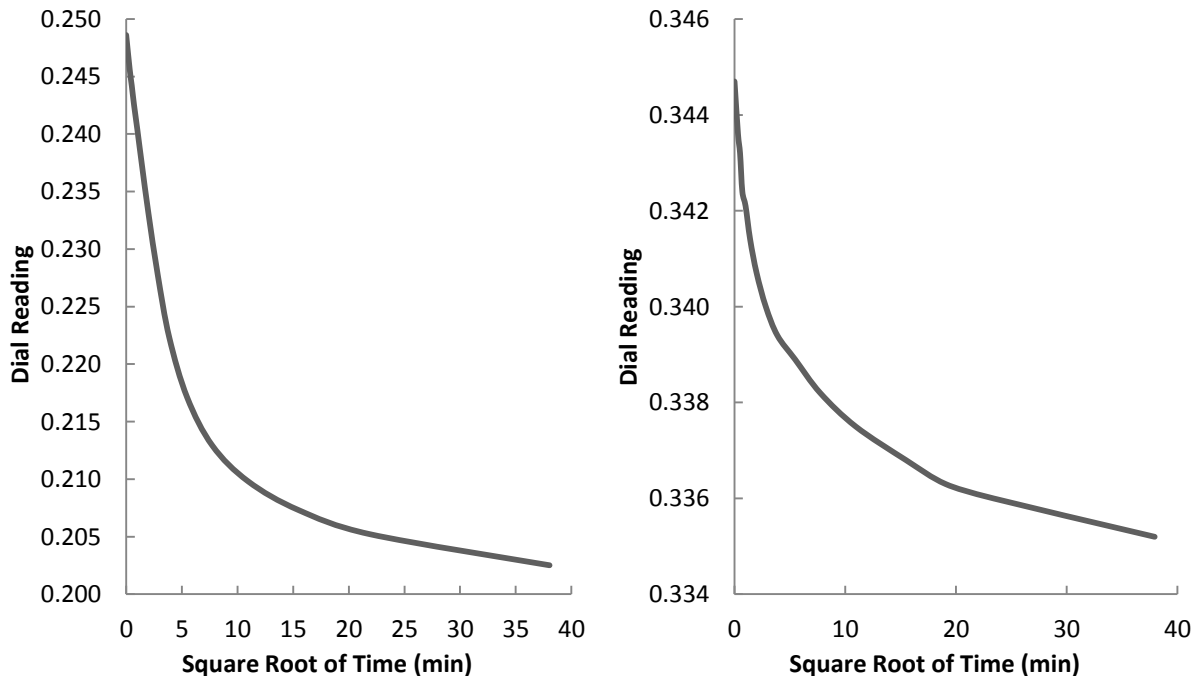


Figure 6-16 Examples of dial readings with square root of time for a loading stage in oedometer test for samples from Ottawa (a) double loading increments (b) random loading increments

6.4.2 Cv from Dissipation Testing

Campanella et al. (1988) performed CPTs with pore pressure dissipation by using different positions of porous stone. Even though they had performed testing on sensitive marine clays in western Canada rather than in eastern Canada, their conclusion may be useful for sensitive marine clays in Ottawa. They observed that for overconsolidated soils, porewater pressure measurements behind the tip and behind the sleeve friction exhibit an increase then decrease in porewater pressure as a result of the gradient. They observed that this is not the case for porewater pressure measurements at the tip. Therefore, the methods of interpretation and estimating of the coefficient of consolidation only apply to the porewater pressure measurements at the tip. However, measurements at the tip may be decreased as a result of stopping or unloading of the tip. On the other hand, for normally consolidated or lightly overconsolidated soils, porewater pressure measurements behind the tip and the sleeve mandrel are desirable because they are not affected by the gradient and the unloading of the tip.

Pore pressure dissipation tests are available for Sites 1 and 7 and no oedometer tests are available for these two sites for direct comparisons with the field pore pressure dissipation tests. However, the coefficient of consolidation determined from these tests for these two sites are between 4.51×10^{-3} and 1.36×10^{-2} cm²/s which is outside and higher than the range determined from the oedometer tests for all of the other sites as shown in Figure 6-17.

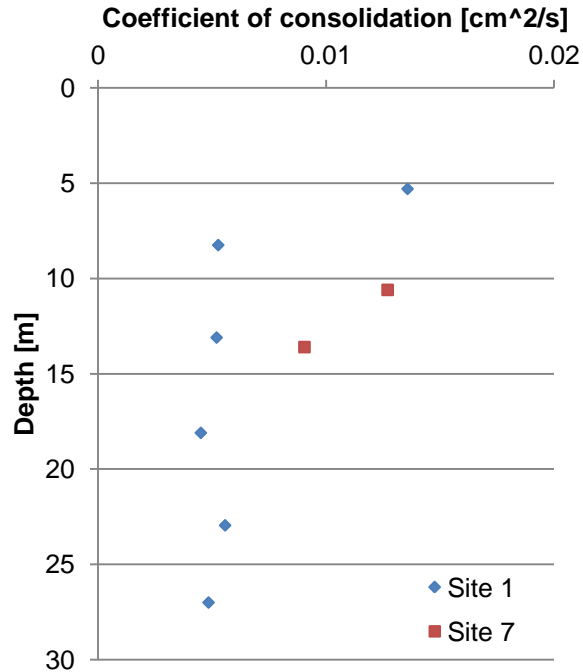


Figure 6-17 Coefficient of consolidation from CPTs by carrying out pore pressure dissipation tests

6.4.3 References

Campanella RG, Sully JP, Robertson P (1988). Interpretation of piezocone sounding in Clay - a case history. Institution of Civil Engineers, Sym. on Penetration Testing in the U.K., Thomas Telford, London.

Holtz, R.D., Kovacs, W. D. (1981). An introduction to geotechnical engineering. New Jersey: Prentice-Hall, Inc.