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Interface Shear Behavior of Sensitive Marine Clays –Leda Clay

by

Ahmed M. Taha

Thesis submitted to the
Faculty of Graduate and Postdoctoral Studies
In partial fulfillment of the Requirements
For the M.A.Sc degree in Civil Engineering

Civil Engineering Department
Faculty of Engineering
University Of Ottawa

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Interface Shear Behavior of Sensitive Marine Clays – Leda Clay

Submitted by
Ahmed M. Taha

In partial fulfillment of
the requirements for the degree of
Master of Science
in
Civil Engineering

Prof. M. Fall, Ph.D.
(Thesis supervisor)

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Abstract

Leda clay, which is a type of sensitive marine clay in Canada, is a hazardous soil that could undergo sudden collapse and flow upon wetting and remolding. This type of soil causes many landslide disasters and foundation damage. The existence of Leda clay at or near the proximities of developed cities makes it challenging for infrastructure expansion, and therefore, challenging for geotechnical engineers. At the location where this sensitive marine clay exists, many foundation designers have adopted the use of deep foundations, such as pile foundations to support heavy structures. The shear behavior and strength parameters at the interface between the (friction) pile and soil are key design parameters. A sufficient knowledge of these interface shear behaviors and strength parameters is also essential for the safe and cost-effective design of several other geotechnical structures (e.g., retaining walls, reinforced soils, and buried structures). However, no studies have yet been implemented on the interface shear behavior between Leda clay and structural material. There is therefore, a need to generate more knowledge about the interface shear behavior of Leda clay.

This thesis deals with an experimental study of the shear behavior at the interface between Leda clay and structural material, such as steel and concrete. The effects of several factors, such as surface roughness of the construction material, Leda clay's overconsolidation ratio (OCR), saturation degree, density, and salt content on interface shear behavior are also investigated. Laboratory tests have been carried out by using an automatic direct shear machine connected to a linear variable differential transformer (LVDT), loading cell and a data logging system. The results of the interface shear tests show that under consolidated drained (CD) and saturated conditions, the interface friction angle increases with an increase in the clay's OCR. The results also indicate that increasing the salinity of Leda clay's pore water enhances its frictional resistance at the interface. Furthermore, the results reveal that Leda clay with a higher dry density shows

higher interface shear resistance. On the other hand, the results also show that the interface shear resistance decreases as the degree of saturation of the Leda clay increases.

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I would like to thank my Lord, the almighty Allah, for his endless bounties. I would like to thank my supervisor, Dr. Mamadou Fall for all of his support and the motivation that he gave to me during the progression of my graduate work. His experience has immensely helped me and also refined my understanding of the problems. Special thanks to my colleagues, Otham Nasir and Jean Claude Celestin, for their exceptional help in the geotechnical lab. I would like to thank my family; my father Muahmmed, my mother Khadiga, my wife Hana, my brother Abdullad and my sisters Inas, Halah and Iman, for their support during my study.

Chapter 1

Introduction

1.1. Problem Statement

Soft sensitive marine clays are well known in Canada and some regions in the world (e. g., Russia, Scandinavia). In Canada, thick deposits of sensitive clays cover large areas of the provinces of Quebec and Ontario (commonly called Leda clay in Ontario and Champlain Sea clay in Quebec). The sensitive marine clays of Canada are young glacial deposits, less than 12,000 years old [1]. The pore water salt of these marine clays has been leaching out since the last glaciations and left a brittle mineral structure. The most striking feature of this sensitive marine clay is its tendency to convert from a relatively brittle material to a liquid mass when it is disturbed, i.e. it can undergo sudden collapse, and flow upon remolding. This kind of soil has caused many geotechnical problems, such as foundation damage and landslides. The existence of this clay in Ontario and Quebec makes it challenging for infrastructure expansion, and therefore challenging for geotechnical engineers. At the locations where this sensitive marine clay exists, foundation designers can adopt the use of deep foundations, such as pile foundations, especially for heavy superstructures.

The contact zone between the soil and pile structure is commonly referred to as the “interface”. The loading response of a soil-pile system is significantly influenced by the mechanical behavior and the shear strength parameters at the interface between the soil and the pile structural material. Therefore, in terms of theoretical analyses and practical engineering, studying the mechanical characteristics of a soil-structure interface is important to arrive at a safe and cost-effective design for various foundation structures, such as piles as well as retaining walls and cutoff walls [2].

Several researchers have investigated the shear behavior at the interface between sand and steel or concrete [e.g., 3,4], whereas few research have dealt with the study of cohesive soil (e.g., clay) interface shear behavior [e.g., 5,6]. Previous research have investigated the influence of several factors, such as structural material, soil properties, and surface roughness on the interface shear behavior and characteristics [3-7]. Despite the tremendous contributions of the aforementioned studies to allow a better understanding of the interface shear behavior between various construction materials and soil, the obtained results cannot be directly applied to the interface shear behavior between Leda clay and construction material. This is mainly because Leda clay is not a “conventional” cohesive soil; it has a special behavior since it is sensitive marine clay. To date, no studies have addressed the interface shear behavior between Leda clay and structure. Hence, there is the need to better understand the interface shear behavior of Leda clay. The shear behavior and shear strength parameters at the interface between piles and Leda clay are key parameters for a cost-effective and safe design of pile foundations. Moreover, this understanding is not only important for pile design, but also crucial for a cost-effective design and accurate performance predictions for other relevant civil engineering structures (e.g., retaining walls, reinforced earth, shallow foundations, and buried structures (e.g. lined tunnels).

1.2. Research Objectives

The main objective of this thesis is to understand the shear behavior at the interface region between Leda clay and structural material, such as steel and concrete. Furthermore, this thesis aims to experimentally investigate and understand the impacts of the following factors on the interface shear behavior of Leda clay:

- the interface roughness,
- overconsolidation ratio (OCR),
- initial saturation degree,
- dry density, and

- pore water salinity.

1.3. Thesis organization

This thesis has five chapters.

- Chapter 2 provides background knowledge on the geological, geochemical and geotechnical properties and behavior of sensitive marine clays and the interface problem. These technical aspects are essential to better understand the results presented in Chapters 3 and 4.
- Chapter 3 presents technical paper I which deals with the study of the interface shear behavior between Leda clay and steel. The details of the experimental program conducted and the results obtained are presented and discussed.
- Chapter 4 includes technical paper II which presents and discusses an experimental study of the interface shear behavior between Leda clay and concrete.
- Finally, Chapter 5 presents the main conclusions and recommendations.

It should be mentioned that because the main results of this thesis are presented in the form of technical papers (paper based thesis manuscript), some of the information are repeated. This is because each paper is independently written (i.e. without taking into account the content of the other papers or the rest of the document) for journal submission in accordance to the preparation instructions. Furthermore, each chapter has its own list of references, which is common practice in paper based thesis manuscripts.

1.4. References

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Chapter 2

Background on Geological and Geotechnical Characteristics of Sensitive Marine Clays and the Interface Problem

An understanding of the geological and geotechnical features of the sensitive clays is essential for a good investigation and analysis of the interface shear behaviour of Leda clay. Also, the understanding of the interface characteristics and parameters is essential for the correct analysis of the interface testing results. Therefore, the objective of this chapter is to provide technical background on sensitive marine clay from two main perspectives: (i) the geological, and (ii) geotechnical perspectives. In addition, a brief background on the interface problem will be given.

The geological study is accomplished by: (i) explaining the sedimentology of sensitive marine clay in three countries around the world, Canada, Norway and Sweden; (ii) giving an overview of the geochemical and mineralogical properties of sensitive marine; (iii) describing the pore water chemistry of sensitive clays and its correlation with sensitivity; (iv) finally by explaining the structure of Leda clay.

In a review of the geotechnical characteristics of sensitive clays, the main mechanical properties which include consolidation, compression and compressibility predictions, and stress-strain behavior are presented and discussed. Also, this section presents and discusses the in-situ undrained shear strength (C_U) of sensitive clays.

The interface background include information on: (i) interface definition; (ii) interface thickness; (iii) surface roughness measurement methodologies; (iv) and finally, the interface shear strength parameters.

2.1. The geological characteristics of sensitive clays

2.1.1. Sedimentology of sensitive clays

This section describes how sensitive clays originated, their common locations, and description of their main types existing in Canada, Sweden and Norway.

2.1.1.1. Sedimentology of sensitive clays in Canada

Canadian sensitive clays are the products of sedimentation in proglacial lake basins which existed between 18,000 and 6000 years before present (BP). These soft clays result from sediments of varved clays that originated from heavy and low density turbidity currents in fresh water lakes. Marine clay deposits also originate from low density freshwater currents. The main types of soft clay deposits that a geotechnical engineer could find are listed in Table 1[1].

Table 2-1: Types of soft clayey soils [1].

Types of deposit	Origin
Waterlaid tills	Unsorted lacustrine sedimentation below floating ice
Lacustrotills	Sauaqueous, proximal, flow deposits in proglacial lakes
Mudflow deposit	Subaerial and submarine flows
Turbidity current deposits	Heavy-density current deposits generated by mudflow dilution, floods, ice calving, slumping, etc
Varved clays	Turbidity current summer deposits and winter clay deposition by settling
Marine clays	Salt-water flocculation and sedimentation

Waterlaid tills

Based on the research carried out by May [2] on Alberta clay, the waterlaid till is actually lacustrine clay sedimentation at or below a floating ice sheet. These deposits overlay hard basal tills and have upward stratification of varved lacustrine sediments.

Lacustrotills and mudflow deposit

The lacustrotills originated from submarine mud flows in proglacial lakes. The mud flows were deposited into the lacustrine environment in thin layers and interbedded with varved clays. Although waterlaid tills are generally deposited in thin layers, very thick deposits were discovered in larger lakes, such as the Sarnia stony clay deposits.

Turbidity current deposits and varved clays

Lake turbidity currents play a major role in the process of varved clay sedimentation. The sedimentology process is categorized into three major types of lake currents, specifically heavy density turbidity currents, interflows, and low density turbidity currents. During the summer, varved clay deposits are mainly silts and sand deposited from overflow currents. In the winter, varved clay deposits become finer as cold currents overflow the heavier (4°C) fresh water in the lake.

The sedimentology processes have been translated into the present varved clay structures (Figure 2-1).

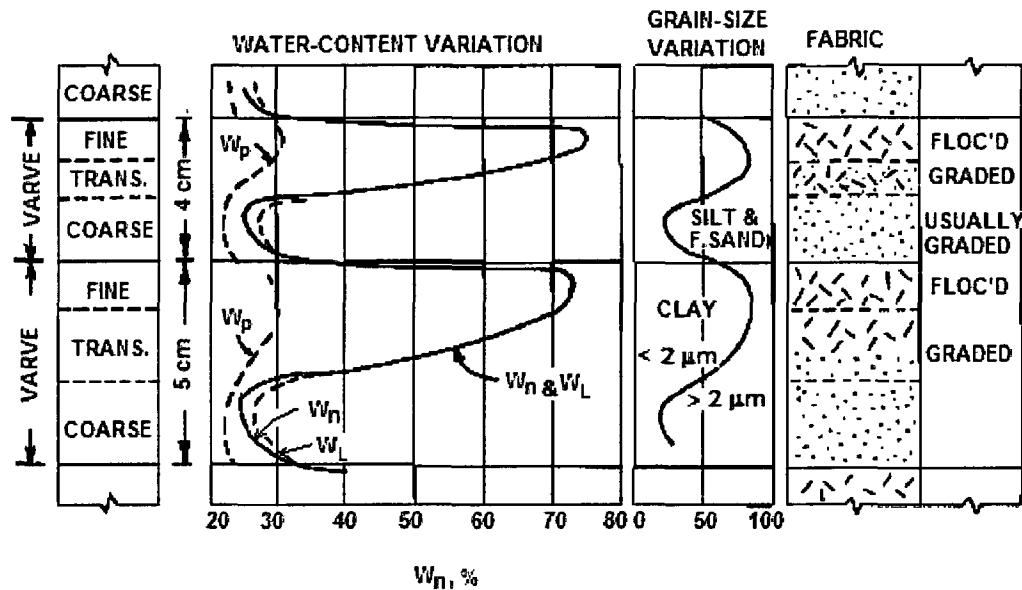


Figure 2-1: Typical thick-layer varves from New Liskeard, Ontario (primarily from Chan and Kenney [3]) [1].

Figure 2-1 shows a repeated fabric of varved clay which features a summer layer that is rich in silt, and overlaid by a finer transition layer. The figure also shows a clay-rich layer on top of a transition layer with gradual variation in particle flocculation. The high water content in the clay layer signifies an open flocculated structure.

Clay deposition in Inland Marine sea

Meltwater stream which carries suspended sediments enters the sea as overflow. This fresh water overflow has been depositing clays by flocculation and settling, causing the Champlain Sea to rebound (Figures 2-2 and 2-3).

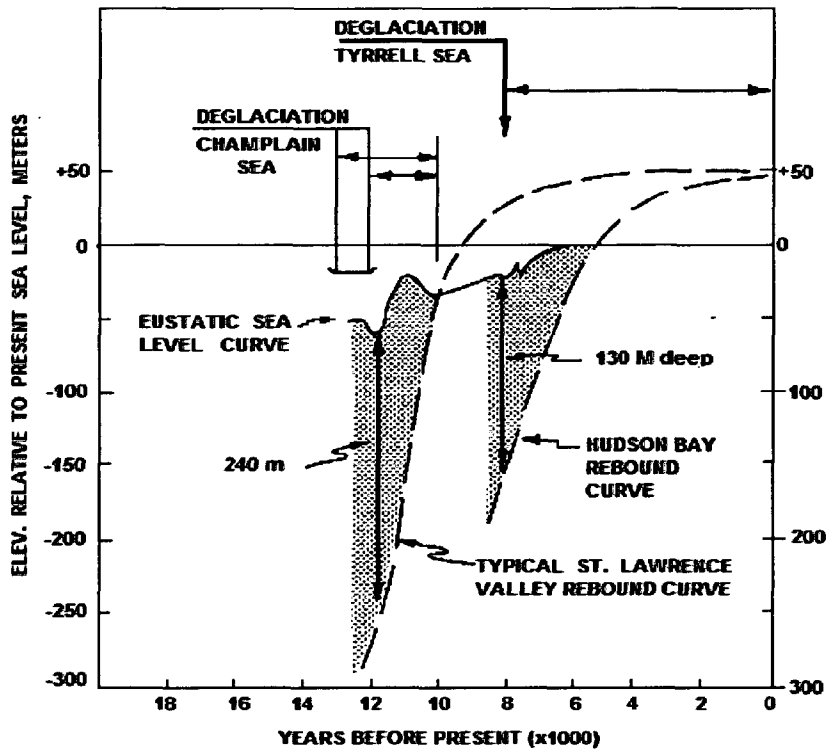


Figure 2-2: Marine transgression and expulsion by crustal rebound. Champlain and Tyrrel Seas (adapted from Kenney [4]) [1].

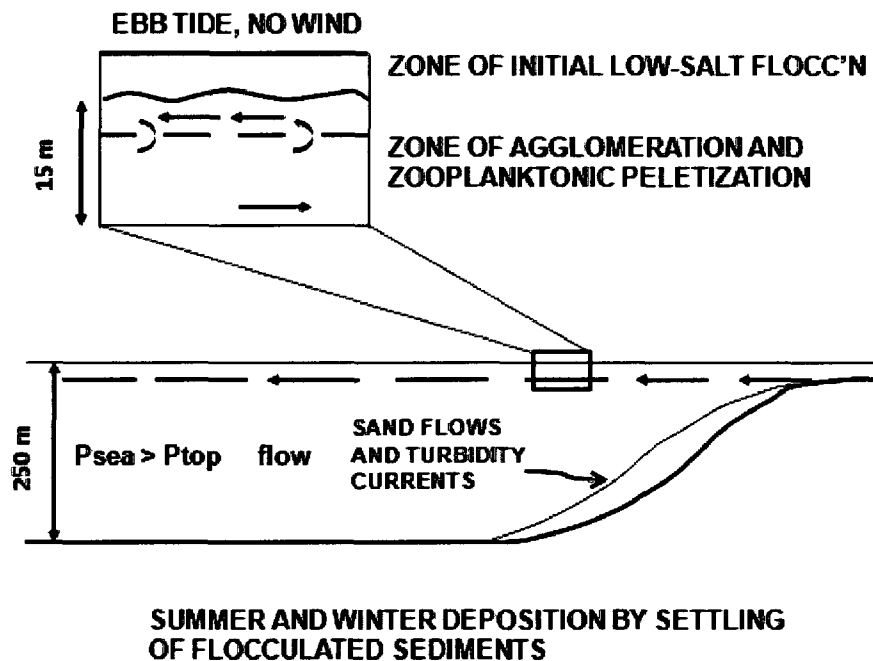


Figure 2-3: Deposition of marine clay by crustal rebound. Flocculation and settling from low-density overflows in Champlain and Tyrrel Seas (adapted from Syvitski [5]) [1].

Figure 2-2 shows that the Champlain Sea was extremely deep and then, its depth was reduced by crustal rebound about 10,000 years BP. Figure 2-3 shows that the upper 5 m of salt sea water was mixed with freshwater overflow. Within this level, low-salt flocculation of clay particles was initiated. Below this level, a process known as pelletization progressed with extensive biologic activity which modified the clay floccs by ingestion [1].

2.1.1.2. Sedimentology of sensitive clay in Norway

In northwest Norway, the landslides at the Målselv valley were caused by heterogeneous sediments of sensitive clay. The landslides exposed the quick clay stratigraphies, which were glaciomarine and marine sediments of interbedded clays with silt and sand [6].

The sediments were deposited at variable depths of the valley during the relative sea-level drop and the final retreat of the Fennoscandian ice sheet. Furthermore, clay-rich glaciomarine and marine sediments were deposited down to a depth of 125 meters. Examination of glaciomarine sediments that were deposited close to the ice-margin of the valley exposed a complex stratigraphy. Fine-grained glaciomarine mud commonly interbedded with coarse-grained sand beds. For example, fine-grained mud occurs above and below the thick units of sandy ripples (Figures 2-4 and 2-5) [6].

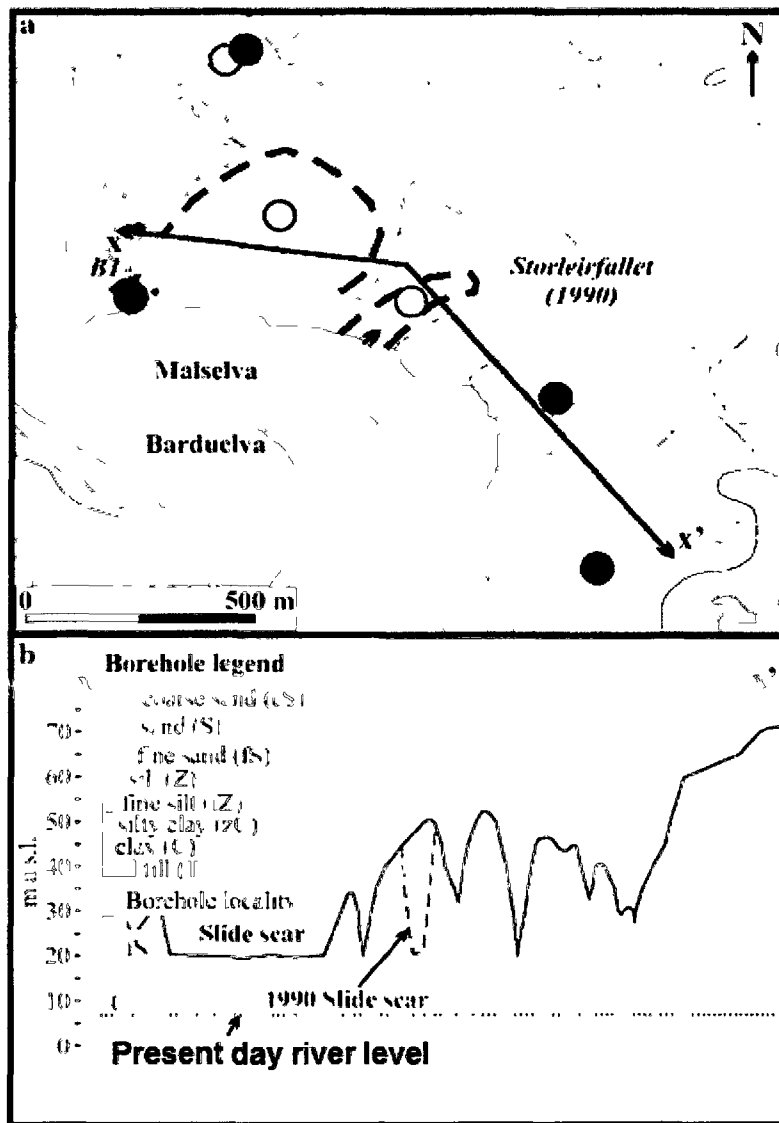


Figure 2-4:a) Details of the Brentmoen / Fosshaug area which show landslide locations. (b) Cross-profile with borehole sediment data and inferred stratigraphy[6].

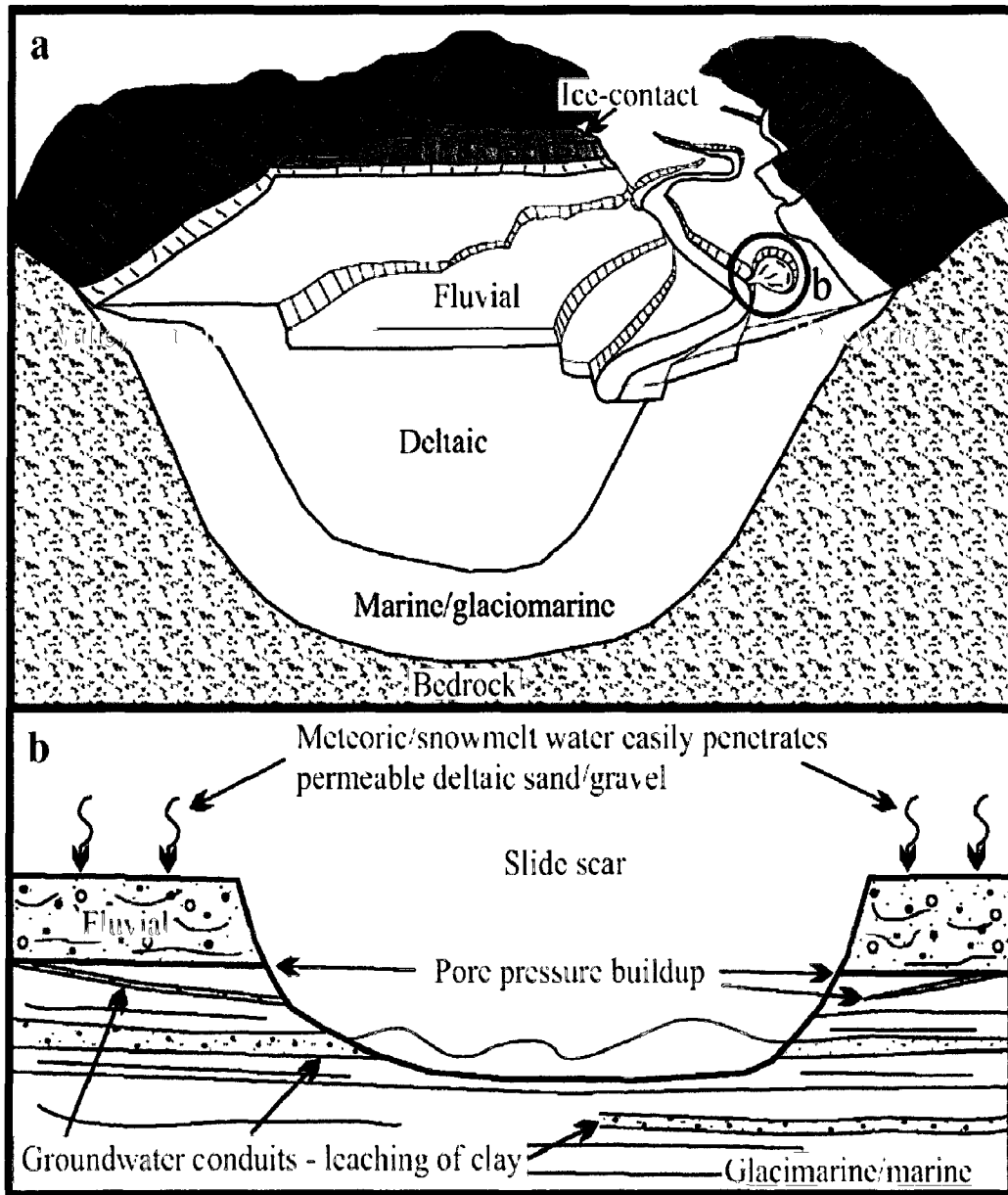


Figure 2-5: Schematic presentation of valley-fill stratigraphy. (a) Whole valley. (b) Clay slide [6].

Figure 2-4 shows the complex stratigraphy of the Målselv valley in a plane view and a vertical profile. Figure 2-5 shows a schematic presentation of the valley's fill stratigraphy.

2.1.1.3. Sedimentology of sensitive clay in Sweden

The in situ properties of sensitive clay have been investigated at three locations near the Swedish west coast in a region which features a rugged, glacially eroded landscape (Figure 2-6)



Figure 2-6: Map of the south of Sweden which shows the locations of sensitive clays: Surte, Hogstorp and Grästorp [7]

At Surte (borehole-1), the sampling site is located 100 m from the Gota Alv River. The upper 50 to 70 meters of thick sediments consist of early Holocene clay underlain by post-glacial late Weichselian.

At Hogstorp (borehole-2), which is located about 100 km north of Göteborg, post-glacial sediments consist of 30 m of sensitive clay bed. The clay shows an uninterrupted depositional sequence during the late Weichselian and early Holocene.

At Grästorp (borehole-3) the sampling site is located near the Nossa Stream. The layers sequence consists of 0.5 to 1 m of Weichselian sand underlain by 20 m of sensitive clay sediment. The clay sediment overlies thin layers of glacial till and coarse sand over bedrock [7].

Most of the serious landslides in Canada, Sweden and Norway occurred in sensitive clay deposits. Shock disturbance or pore water pressure build up initiate liquefaction in high sensitive clays. There are many factors that lead to an increase in the sensitivity of clay. Several geochemical and mineralogical studies have been carried out to investigate this phenomenon.

2.1.2. The geochemical and mineralogical properties of sensitive clay

The geochemical and mineralogical properties of sensitive clay control its sensitivity levels. Sensitivity (S_t) is commonly defined as the ratio (τ_u/τ_r) between the undisturbed undrained shear strength (τ_u) and the remolded shear strength (τ_r). Quigley [1] summarised the main geochemical and mineralogical characteristics that can lead to high sensitivity in sensitive clays (Tables 2-2 and 2-3).

Table 2-2: Geochemical and mineralogical controls on sensitivity of clayey soils. Factors that produce high undisturbed strength and high sensitivity [1] are shown.

Factors Producing High Undisturbed Strength and High Sensitivity
1 Depositional Flocculation <ul style="list-style-type: none">• Saline (Low Zeta Potential)• High Sediment Concentration• Divalent Cation Adsorption
2 Slow Increase In Sediment Load
3 Cementation Bonds
Presentation of Carbonates & Sesquioxides (amorphous matter)

Table 2-3: Geochemical and mineralogical influential factors of sensitivity of clayey soils. Factors that produce low undisturbed strength and high sensitivity [1] are shown.

Factors Producing Low Remolded Strength and High Sensitivity
1 High in-situ water content ($W_n \geq W_{LL}$)
2 Low specific surface of soil grains <ul style="list-style-type: none"> • High silt content or high rock flour in $< 2 \mu\text{m}$ fraction • High primary mineral = low clay mineral content
3 High Zeta Potential <ul style="list-style-type: none"> • Expanded double layers = high inter-particle repulsion • High Inter-particle repulsion = dispersed or peptized state • Low salinity by leaching ($< 2 \text{ g/L}$) • Organic and inorganic dispersants (anion adsorption) • High monovalent cation adsorption relative to divalent cations
4 Low amorphous content
5 Low smectite content

Quigley's research indicated the presence of amorphous materials as cementing agents in sensitive clays. The main amorphous constituents were complex compounds of alumina, silica and iron. In another piece of research on dominant clay minerals that exist in Leda clay, Hanyes [8] performed an analysis by using X-ray diffraction which indicated that the dominant clay minerals of Leda clay are illite-mica, chlorite and amphibole with small, but considerable amounts of smectite [1].

At Hawkesbury in Ontario, Hayens and Quigley [9] carried out testing on four boreholes in Leda clay to study its geochemical and mineralogical profiles. By assessing the minerals that exist in Leda clay, they concluded that sensitivity is directly correlated with Leda clay carbonate content and inversely with its salinity.

Also, they found that there is no correlation between sensitivity and amorphous matter as determined by HCl/ NaOH Segalen extraction [1] (Figure 2-7).

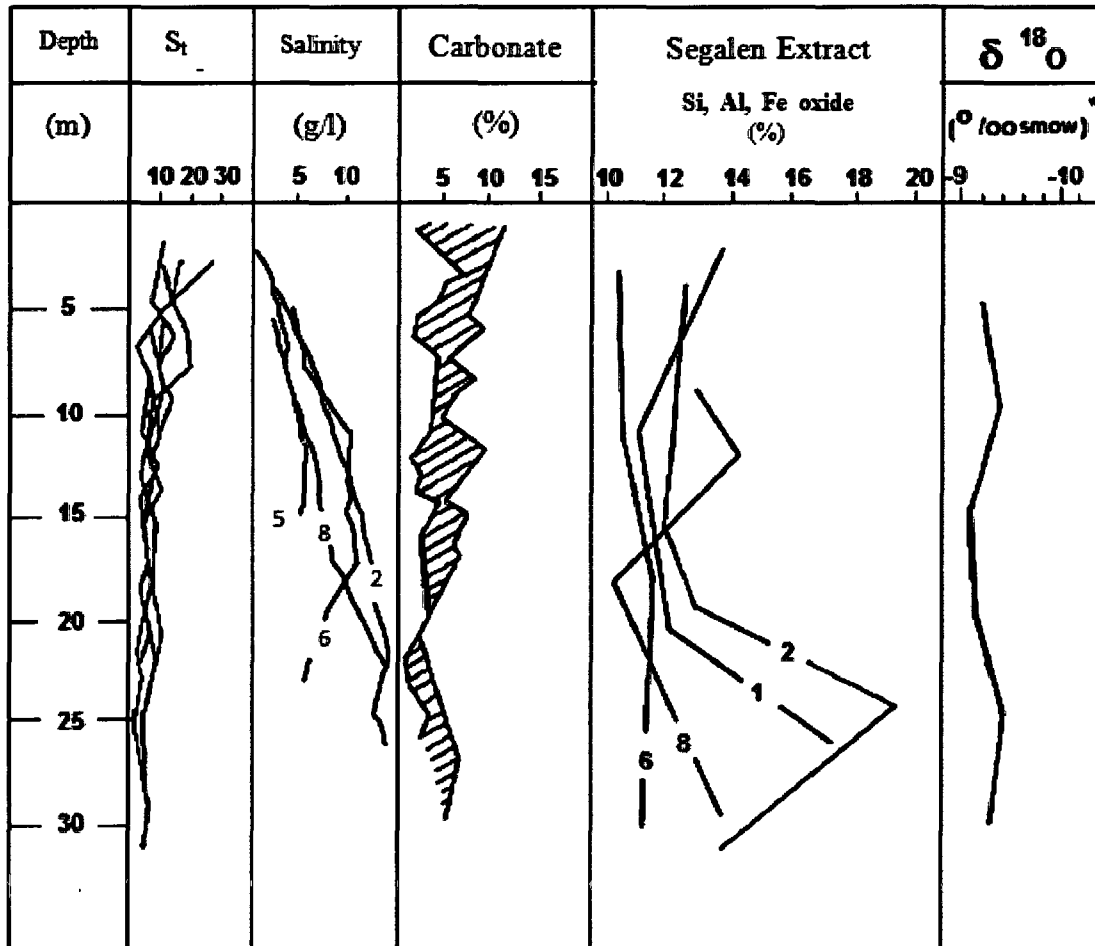


Figure 2-7: Sensitivity, salinity, carbonate, “amorphous” matter, and ^{18}O vs. depth relationships, Hawkesbury Leda clay. $^{\circ}\text{Smow}$ = standard mean ocean water [1].

Figure 2-7 shows the composite profile of four boreholes near Hawkesbury, Ontario. The contour lines show the effect of salinity, carbonate content and amorphous matter on sensitivity.

Similarly, in southwestern Sweden, research carried out by Sköld et al. [7] indicated that the major clay mineral is illite. Other clay minerals include chlorite, quartz, feldspar and calcite. The remaining minerals are micas, kaolinite, chlorite,

heavy minerals and carbonates with small amounts of vermiculite and montmorillonite [7]

Likewise, researchers in Norway collected thirteen sediment samples from four different landslide sites and analysed them by laser diffraction to document their mineral types and grain sizes Eilertsen et al [6] reported that the clays in the landslide deposits predominantly consist of quartz, plagioclase, mica (mainly illite), chlorite and a few percent of swelling minerals of the smectite type

The above discussion leads to the conclusion that the main clay mineral that exists in sensitive clays is illite Other minerals of sensitive clays include quartz, feldspar and chlorite

2.1.3. Pore water chemistry of sensitive clays

Haynes and Quigley [9] investigated pore water chemistry throughout a depth of 30 meters in Hawkesbury Leda clay They reported that the dominant cation present was sodium ion (Na^+) with minor concentrations of magnesium ion (Mg^{2+}), calcium ion (Ca^{2+}), and potassium ion (K^+) This mix of cation yields salinities that vary from about 2 to 4 g/L at surface to around 15 g/L at depth At these high salinities, the effects of factors that lead to high sensitivity, such as peptizing constituents, are neutralized (see Figure 2-8) [1]

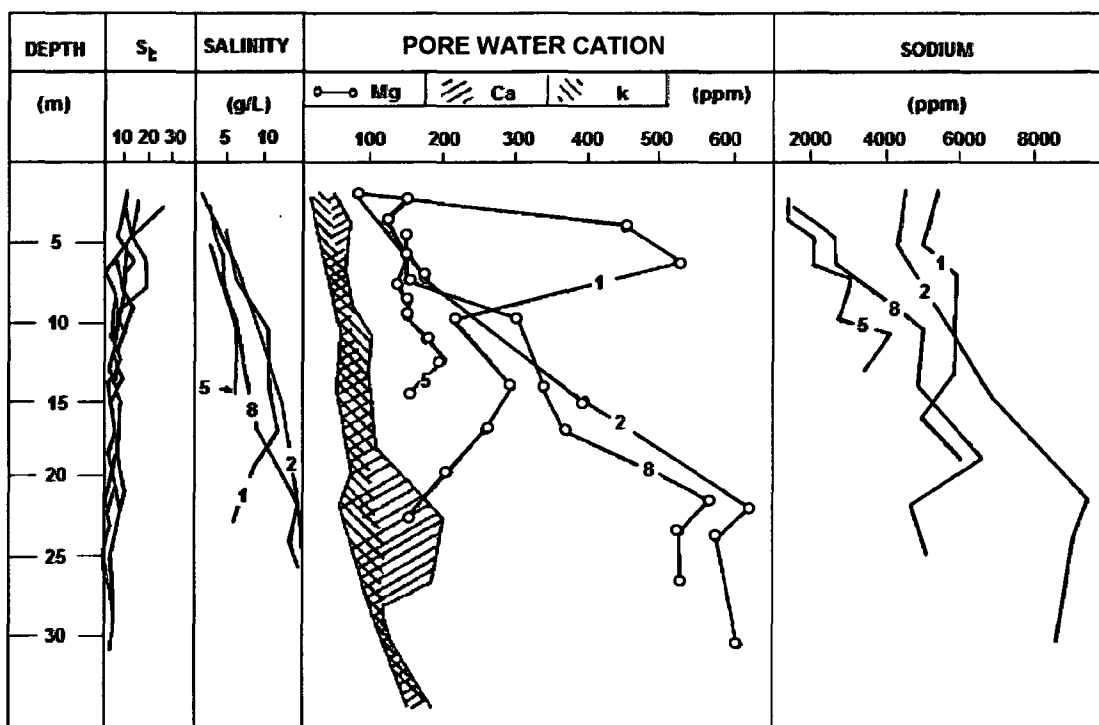


Figure 2-8: Sensitivity, salinity, and pore water cation chemistry vs. depth, for Hawkesbury Leda clay [1].

In another attempt to explore the pore water chemistry of sensitive clays, Sköld et al. [7] analysed pore water concentrations in sensitive clay around the Göta River in southwest Sweden. Similar to Canadian Leda clay, the dominant cation present was Na^+ at all depths. Other existing cations were K^+ , Mg^{2+} and Ca^{2+} and all follow a pattern of increasing concentration from 4 to 6 meters in depth. Interestingly, down to a depth of 15 m, there is positive correlation between sensitivity and the ratio of Na^+ to K^+ , Mg^{2+} and Ca^{2+} . On the other hand, below a depth of 15 meters, the correlation is negative with increasing sensitivity and reduction in Na^+ (see Figure 2-9)[7].

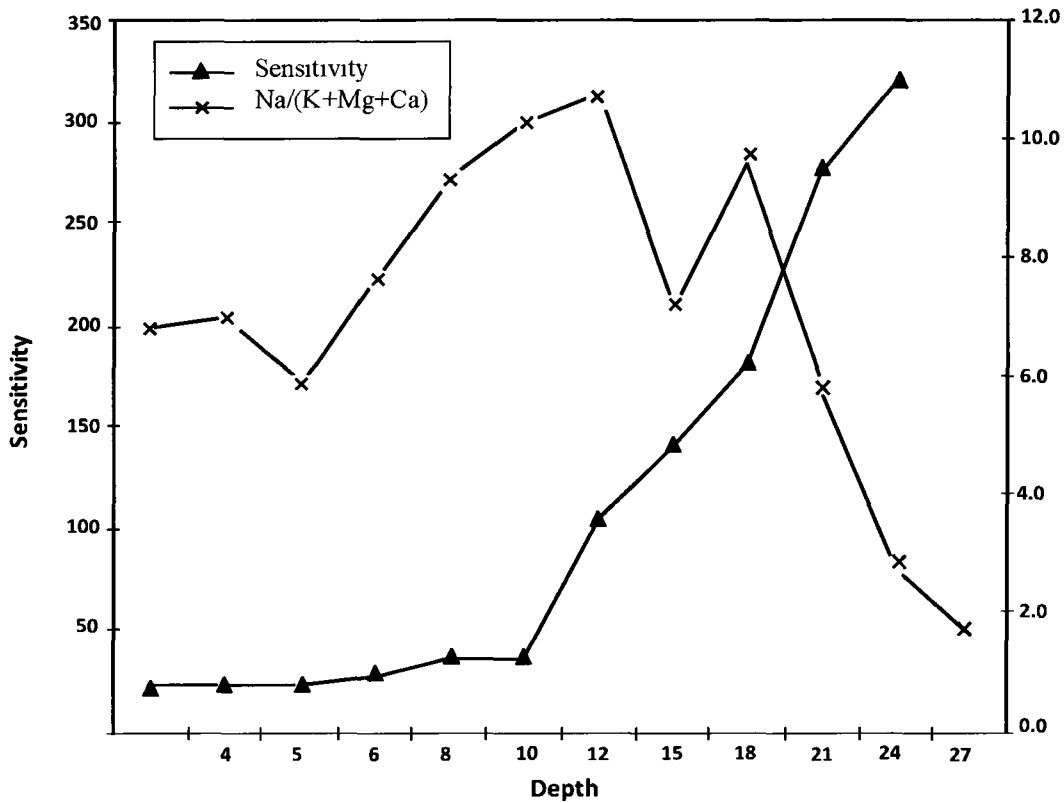


Figure 2-9: The ratio of Na⁺ (eq) versus the sum of K⁺, Mg²⁺ and Ca²⁺ (eq) in the pore water at Surte[7].

2.1.4. Sensitive clays structure

Clay microstructures reflect the entire geologic and stress history of a clay deposit and will also affect the engineering response of that clay. Individual clay particles always flocculate in submicroscopic fabric units known as domains. Domains that aggregate together in a larger fabric, which is visible to a light microscope, are known as clusters. Clusters flocculate to form peds or groups of peds. Peds are visible to the naked eye. Many of the Leda clay domains and peds are fecal pellets (see Figure 2-10)[10]

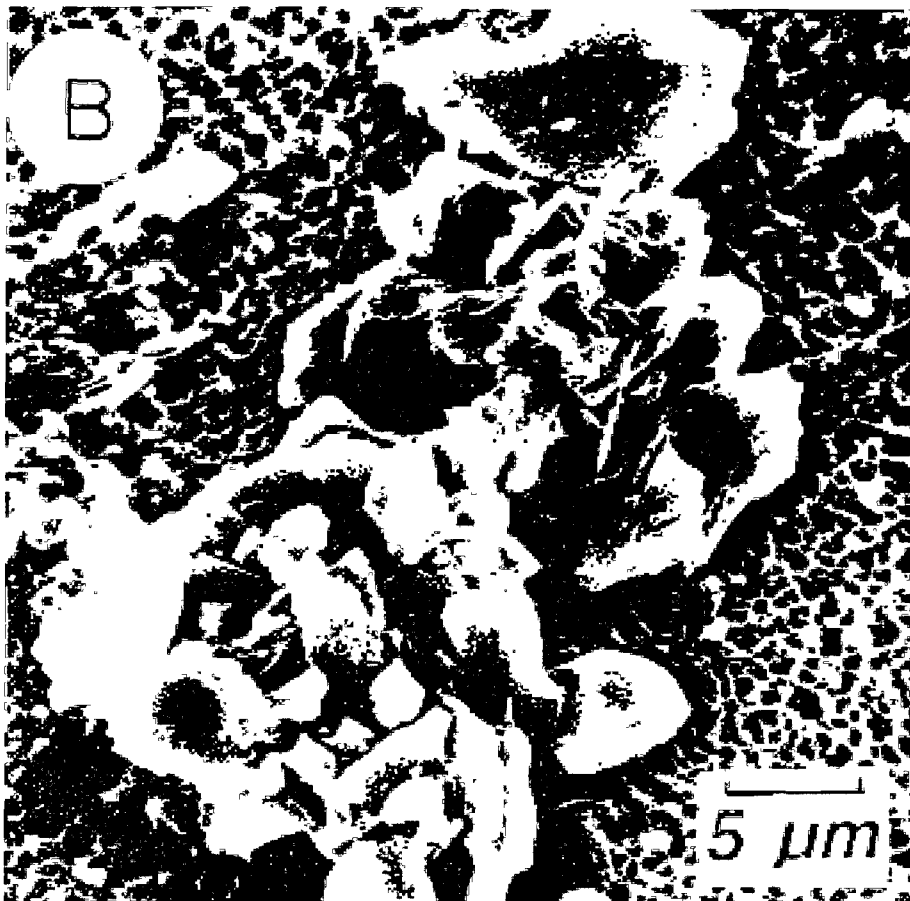


Figure 2-10: Scanning electron photomicrograph of salt-water floccules from water at $S = 20\%$ [1].

Leda clay floccules have large amounts of organic matter with siliceous and calcareous shell fragments. This situation is ideal for in situ bacterial activities, which explains the black mottling in marine Leda clay. The cemented floccules will reserve their structure in situ until ruined by consolidation.

Leda clay structure plays an important role in the mechanical behavior of sensitive clays. Delage and Lefebvre [11] investigated the structure of a medium sensitive Champlain clay. Their investigation applied scanning electron microscopy on natural sensitive clay samples, which was dehydrated by freeze-drying. The natural sensitive clay samples were taken from intact and remolded clays.

Along the vertical plane in the intact samples, the researchers observed an aggregated structure characterized by small platelets and separated by regular porous

networks. These regular porous networks define a large interaggregate porous medium. Some silt particles exist, which are scattered and covered by clay platelets, also along the vertical plane (Figure 2-11a) On the other hand, along the horizontal plane, the general appearance of the structure is different. Silt grains exist in larger numbers while clay practical edges are less in comparison to the number of particle edges in the vertical plane. Also, larger clay plates exist in parallel to the horizontal plane (see Figure 2-11b). In both planes, the clay particles were observed to occupy most of the sample volume more than the non-clay particles, and seemed to be more apparent [11]. The above observation would lead to the conclusion that the intact samples show aggregated anisotropic structures with an allowance for high water content.

In the remolded samples, the researchers observed that the clay structure does not disperse by remolding and the interaggregate pores remain large with small intra-aggregate porosity [11].

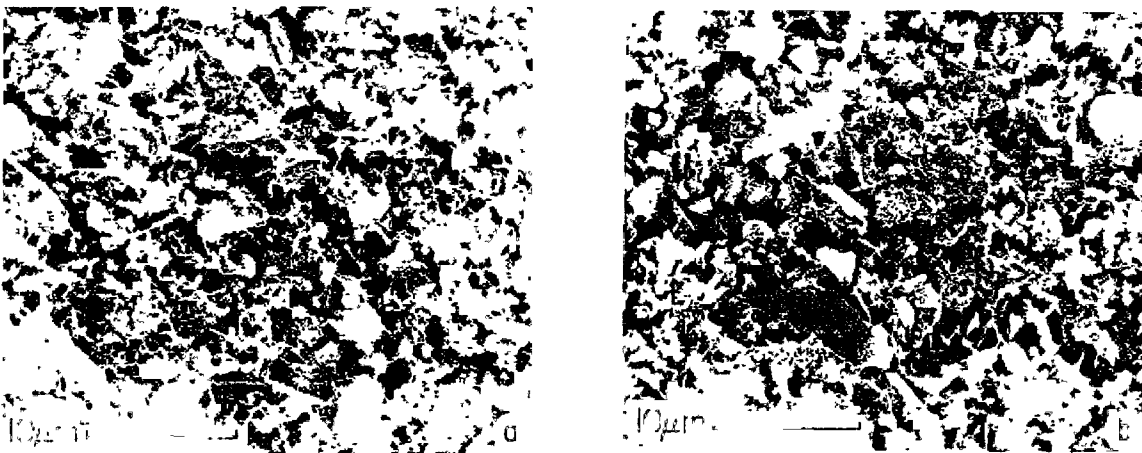


Figure 2-11: Photomicrographs of St. Marcel clay, on : a) vertical plane x 3000 ; b) horizontal [11]

However, the consolidation characteristics of the remolded samples defer very much from the intact samples and result in different consolidation parameters

2.2. Geotechnical properties of sensitive clays

This section presents an overview of the main mechanical properties of sensitive clays, including consolidation, compression and compressibility predictions, and stress-strain behavior. Also, this section presents the in-situ undrained shear strength (C_U) of sensitive clays.

2.2.1. Consolidation behavior and sensitive clay structure relationship

Leda clay is considered to have a “card-house” type of structure that would experience noticeable particle reorientation when consolidated. The relationship between soil structure (fabric) and anisotropic consolidation of sensitive clay was examined by [12]. The clay particle parallelism was measured by using undisturbed and remolded samples of Leda clay from Ottawa. It was confirmed that anisotropic consolidation of sensitive clay produces reorientation of the clay flakes in a plane perpendicular to the major principle consolidation pressures σ_1 and σ_3 (Figure 2-12)[12].

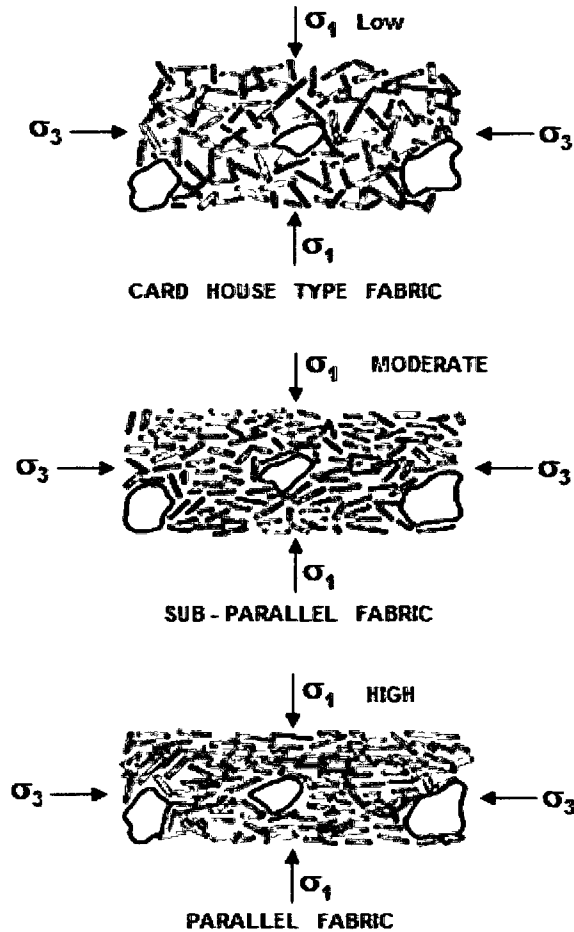


Figure 2-12: Fabric of marine clay at diff stages of anisotropic consolidation [12].

In the undisturbed samples, the researchers did not notice significant flake reorientation and parallelism before reaching the preconsolidation pressure. However, as illustrated in Figure 2-13, after reaching the preconsolidation pressure, there is sudden parallelism and rupture in the bonds between particles [12].

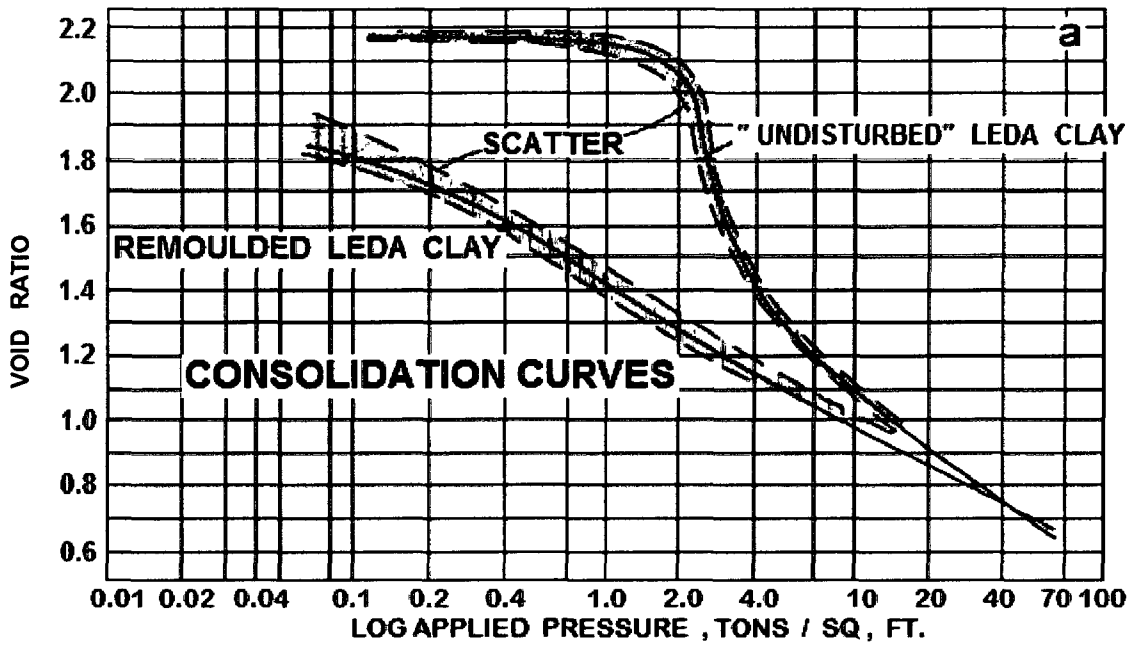


Figure 2-13: Clay platelet parallelism produced by anisotropic consolidation of "undisturbed" and remoulded samples of Leda clay [12].

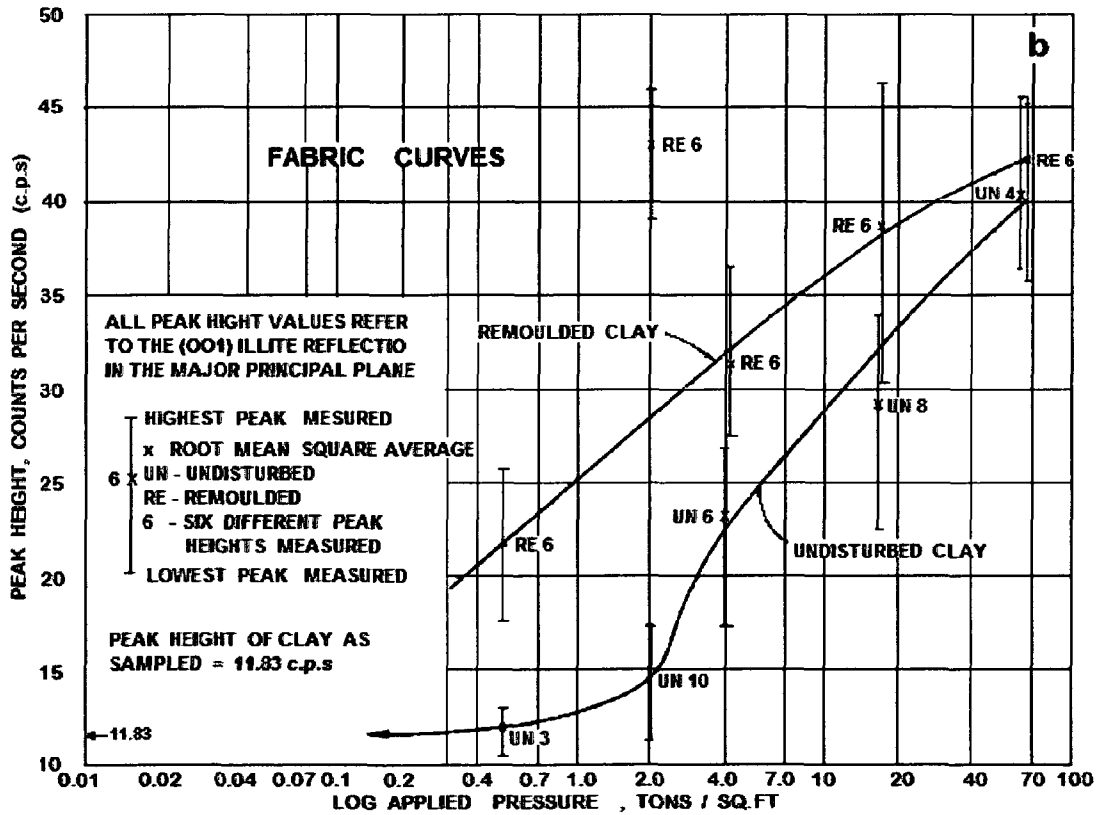


Figure 2-14: Clay platelet parallelism measurement [12].

Beyond the preconsolidation pressure, greater parallelism and flake reorientation is observed with a larger reduction in the void ratio on the remolded sample. This is due to the fact that remolding breaks the bonds between particles and rotates the particles more easily [12]. Figure 2-14 shows this behavior in which a greater degree of particle parallelism, measured from the amplitude of the (001) illite peaks, are observed for the remolded clay samples compared to the undisturbed samples. Figure 2-13 also shows that sample disturbance causes reduction in the slope of the consolidation curve and underestimation of the preconsolidation pressure. This could lead to errors in estimating the consolidation settlement [12].

Another result that can be drawn from the consolidation testing is that a slower consolidation rate implies a longer time is available for the sample to shear. This behavior is because a longer shearing time allows higher particle reorientation and parallelism. This conclusion is confirmed by other triaxial tests which indicate clay platelet parallelism along the failure surface [12].

Hong [13] completed research on the compression behavior of sensitive clay deposits around the Ariake Bay in Japan. Then, he developed a correlation between the compressibility and stress sensitivity of the sensitive clay deposits. This correlation depends on calculating the void index I_v , the intrinsic compression line (ICL) and the sedimentology compression line (SCL). The I_v was expressed by Burland [15] as

$$I_v = \frac{e - e_{100}^*}{e_{100}^* - e_{1000}^*}$$

Where e_{100}^* and e_{1000}^* are the void ratios corresponding to vertical effective stresses $\sigma'_v = 100$ kPa and 1000 kPa. Also, Burland [15] expressed the equation of the (ICL) as

$$I_v = 2.45 - 1.285x + 0.015x^3$$

The SCL is calculated by using an in situ void index, I_{vo} , versus the effective overburden stress, σ'_{vo} . I_{vo} was calculated in a similar manner to I_v except that it uses the in situ void ratio e_o [13].

To study the compression behavior, Hong [13] performed ten oedometer tests on the reconstituted Ariake clays and the resulting compression curves were compared to the (ICL) and the (SCL) (se figure 2-15) [13].

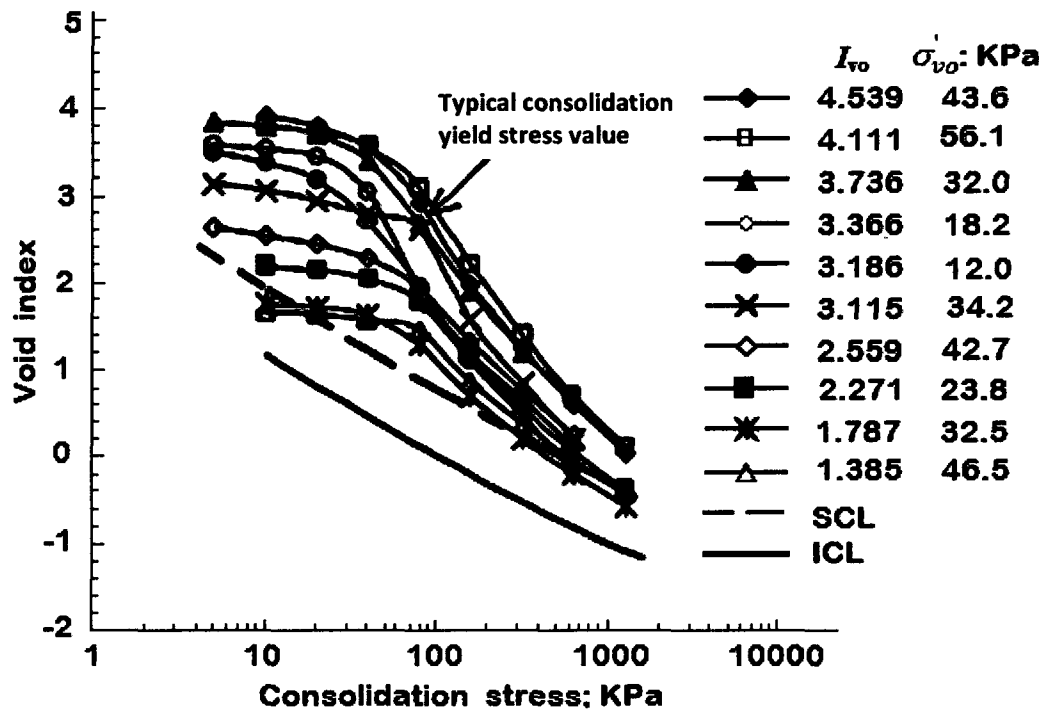


Figure 2-15: Representative compression curves of natural Ariake clays in $I_v - \log \sigma'_o$ plot [13].

Figure 2-15 shows how all of the compression curves of the undisturbed clay gradually converge towards the ICL and crosses the SCL as the consolidation stress increases. Furthermore, at stress levels higher than the consolidation yield stress, the compressibility significantly increases. In this post-yield state, the compressibility of natural Ariake clay is higher than that of the ICL [13].

Hong [13] mentioned in his research that there are three ways to correlate stress sensitivity with compressibility: stress sensitivity to yield stress ratio, stress sensitivity to differential I_v , and stress sensitivity to compression index (C_c).

Stress sensitivity can be obtained by measuring the clay yield stress σ'_{vy} from the oedometer test and calculating the ratio of σ'_{vy} to the corresponding stress on the ICL, σ^*_{ve} . Then, the stress sensitivity can be expressed as:

$$S_{ts} = \frac{\sigma'_{vy}}{\sigma^*_{ve}}$$

In addition, stress sensitivity can be obtained by measuring the differential value between I_{v0} and I_v on the ICL at the same overburden pressure, I_{vr} . This relationship can be expressed by the following equation of Figure 2-16:

$$S_{ts} = 1.89 \exp [1.735 X (I_{v0} - I_{vr})]$$

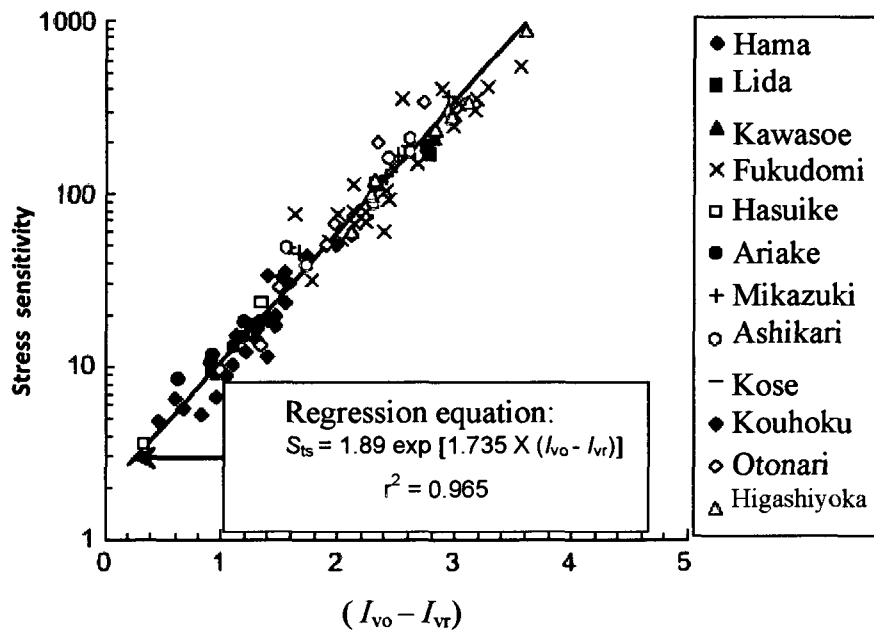


Figure 2-16 : Relationship between stress sensitivity and $(I_{v0} - I_{vr})$ [13].

Figure 2-16 clearly shows that the differential I_v is positively proportional to the stress sensitivity, and at a low I_{v0} , which is closer to the ICL, the stress sensitivity of Ariake clay tends to decrease.

Finally, the ratio of the measured C_c in the pre-yield and post-yield states, C_{CLB} and C_{CLR} , respectively, can also be used to obtain clay sensitivity (see Figure 2-17).

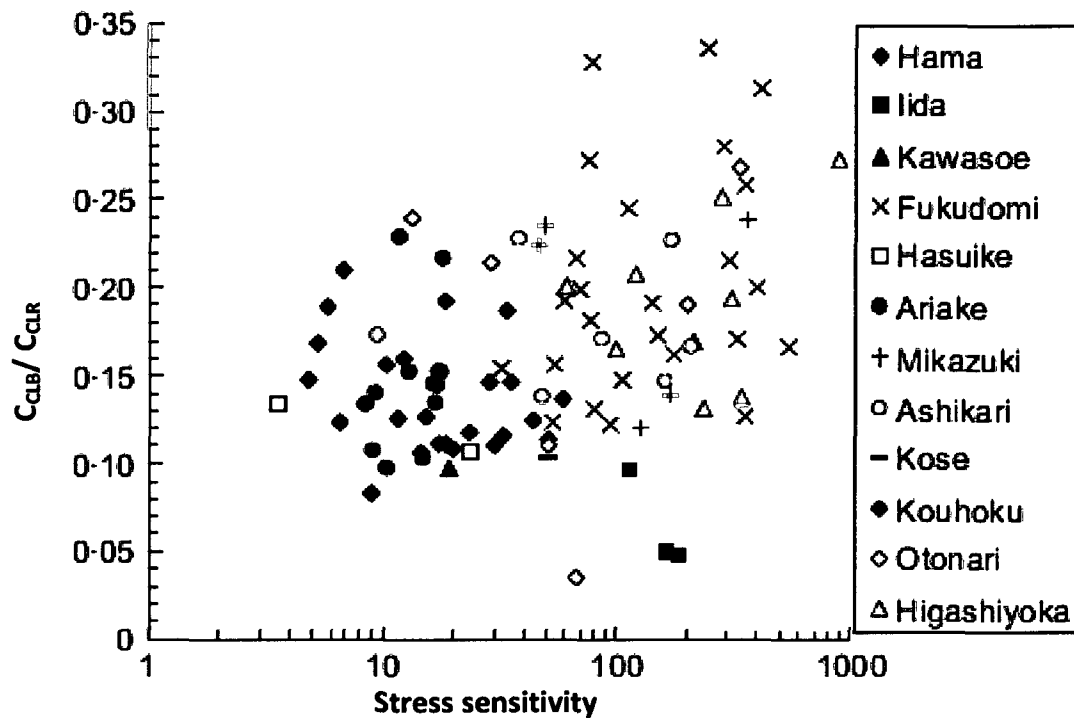


Figure 2-17: Relationship between C_{CLB}/C_{CLR} , where $C_{CLR} = 0.434 \ln \frac{1+e_{100}^*}{1+e_{1000}^*}$ [13].

Figure 2-17 shows that the C_{CLB} is the C_c in the pre-yield stage and can be measured from the plot: $[\ln(1+e) \text{ vs. } \ln \sigma'_v]$. The above figure clearly shows that the ratio (C_{CLB}/C_{CLR}) is positively proportional to the stress sensitivity.

The sensitivity versus I_v correlations can be tested on other kinds of sensitive clays around the world to expand the application of these useful correlations. However, more research has to be carried out on remolded clays.

Shibata and Nishihara [14] derived a relationship between the C_c of remolded Osaka sensitive clay and its liquid limit (LL). This relationship is obtained from several consolidation tests and equations to prove that the C_c vs. w_L relationship of Osaka clay is in agreement with Burland's [15] experimental tests on reconstituted natural clays. The derived C_c vs. w_L is expressed as:

$$C_c = 0.80 (w_L - 0.9) \quad (\text{see Figure 2-17}).$$

Figure 2-18 shows that the C_c line is in good agreement with Burland's [15] results of reconstituted natural clays [14],[15].

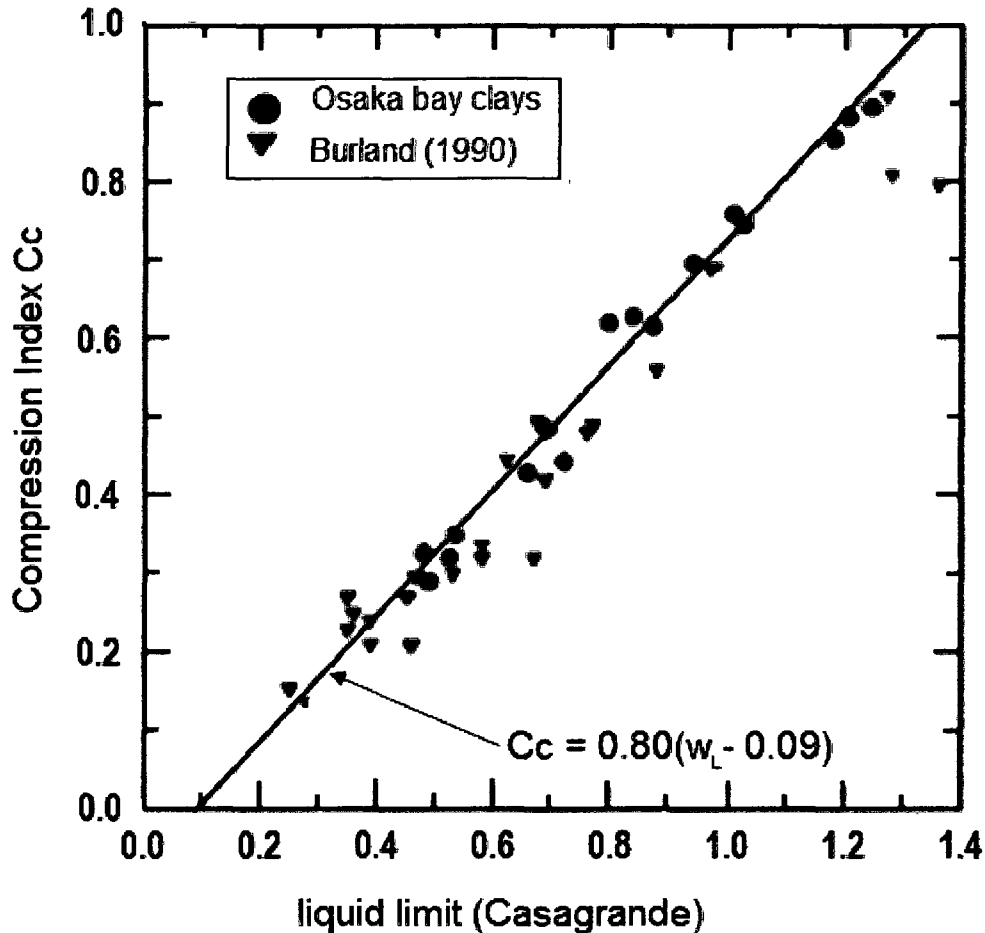


Figure 2-18: Relation between the C_c and LL [14].

2.2.2. Stress-strain behavior of sensitive clays

Dam et al. [16] implemented a series of direct shear tests (DSTs) on a selection of four types of sensitive marine clays around the world: Ariake (Japan), Bothkennar (United Kingdom), Champlain (Canada) and Drammen (Norway). The aim of these tests was to study the shear strength and deformation characteristics of these clays in the field state, overconsolidation undisturbed state, and in the normally consolidation after destruction state, which is the remolded state. The results of these tests are plotted for each clay and show the stress-displacement response in the two states (e.g. see Figure 2-19)[16].

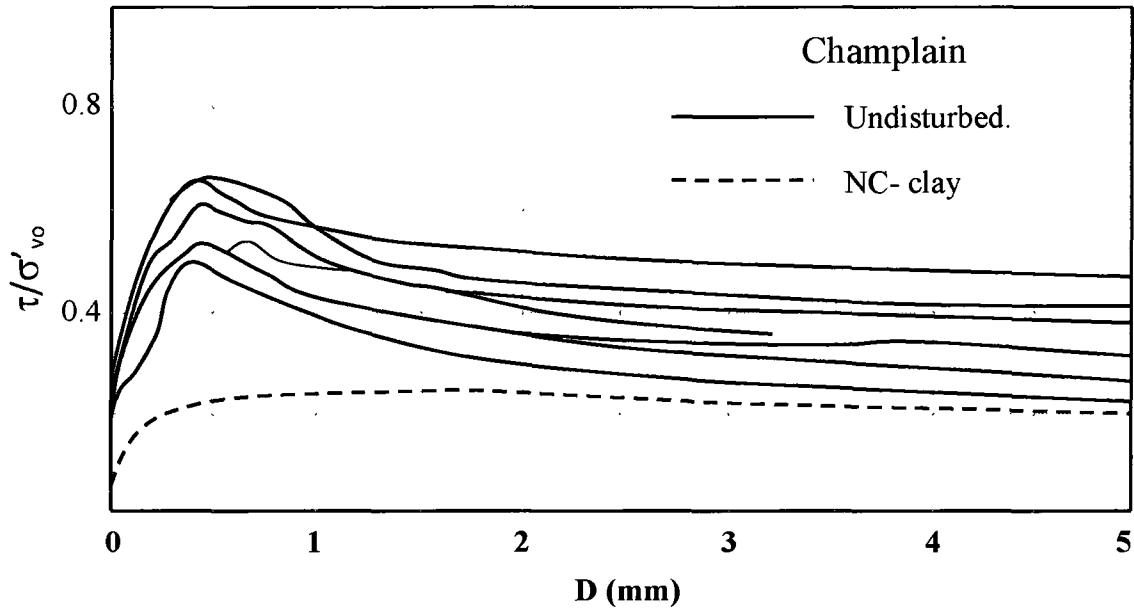


Figure 2-19: Normalized shear stress-displacement curves from DST [16].

The normalized stress-displacement curves in Figure 2-19 show that all clay samples demonstrate brittle behavior with higher shear stress at failure (τ_f) and with smaller displacement to failure (D_f) than those in the normally consolidated or remolded state. This behavior indicates that the marine clays had been structured very well, cemented strongly and overconsolidated.

Furthermore, to study the effect of overconsolidation on the brittleness of natural marine clays, Dam et al. [17] suggested that the brittleness should be presented by using three parameters: slope ratio (S_1/S_2), strength ratio (τ_f / τ_f), and area ratio, (A_1/A_2). The three ratios are illustrated in Figure 2-20.

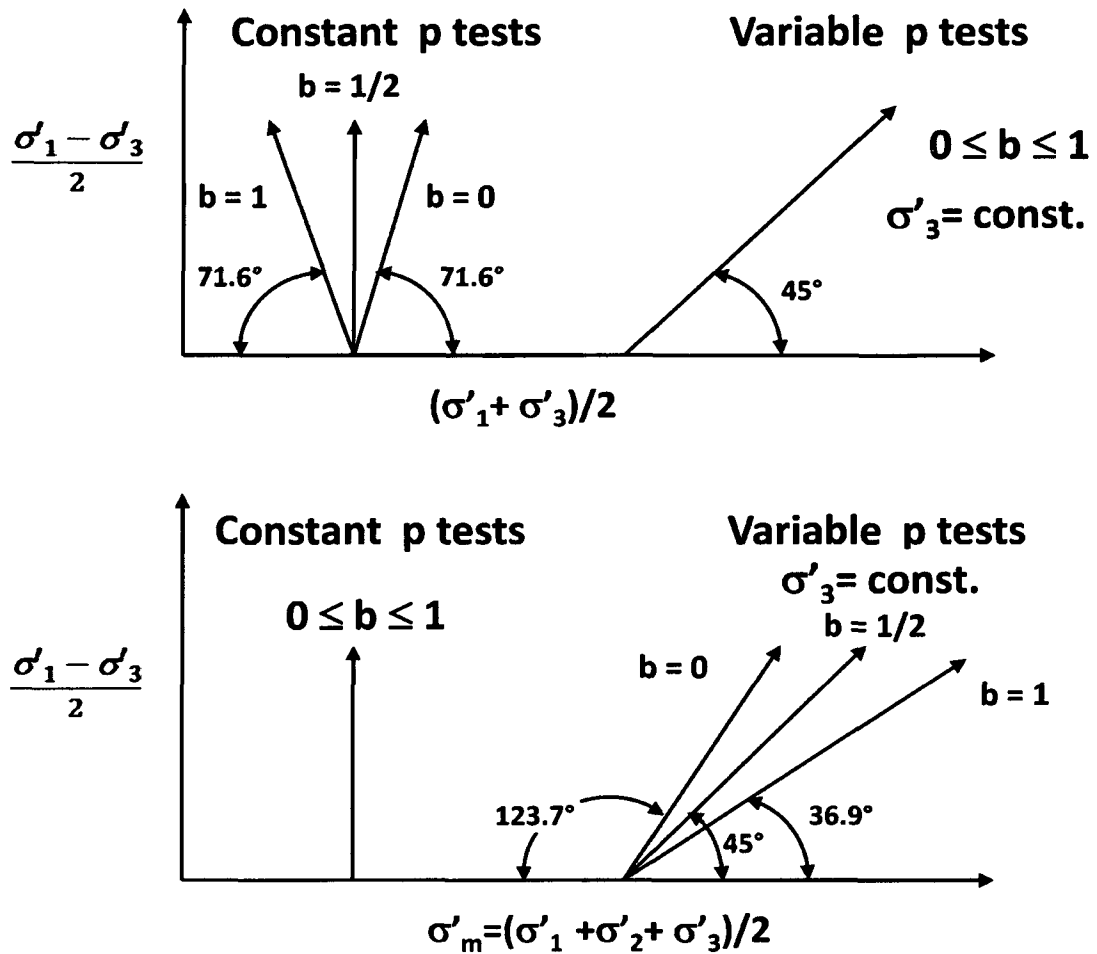


Figure 2-21: Stress paths in true-triaxial testing where $b = (\sigma'_2 - \sigma'_3) / (\sigma'_1 - \sigma'_3)$ [18]. (a) Shear stress- normal stress diagram. (b) Shear stress-mean stress diagram. Where $b = (\sigma'_2 - \sigma'_3) / (\sigma'_1 - \sigma'_3)$ [18].

In view of the research by La Rochelle and Lefebvre [19],[20] St-Louis sensitive clay was found to possess anisotropic characteristics that would make it improper to use Hooke's law to describe the complicated stress-strain behavior. Therefore, the researchers adopted Barden's [21] formula for anisotropic elastic material [18]. The stress-strain relationships are written as:

$$\varepsilon_1 = \frac{\Delta\sigma_1}{E_v} - \mu_{hv} \frac{\Delta\sigma_2}{E_h} - \mu_{hv} \frac{\Delta\sigma_3}{E_h} \quad \text{Equation 1 Barden [21] formula for anisotropic elastic material}$$

$$\varepsilon_1 = -\mu_{vh} \frac{\Delta\sigma_1}{E_v} + \frac{\Delta\sigma_2}{E_h} - \mu_{hh} \frac{\Delta\sigma_3}{E_h}$$

$$\varepsilon_1 = -\mu_{vh} \frac{\Delta\sigma_1}{E_h} - \mu_{hh} \frac{\Delta\sigma_2}{E_h} + \frac{\Delta\sigma_3}{E_h}$$

Yong and Silvestri [18] did several triaxial stress controlled and strain controlled tests and found that the stress strain curves obtained confirm the anisotropic character of St-Louis sensitive clay. This can be observed from a strength comparison between the vertically and horizontally trimmed samples. Figure 2-22a defines the inclination angle of clay samples whereas Figure 2-22b shows the strength comparison [18]

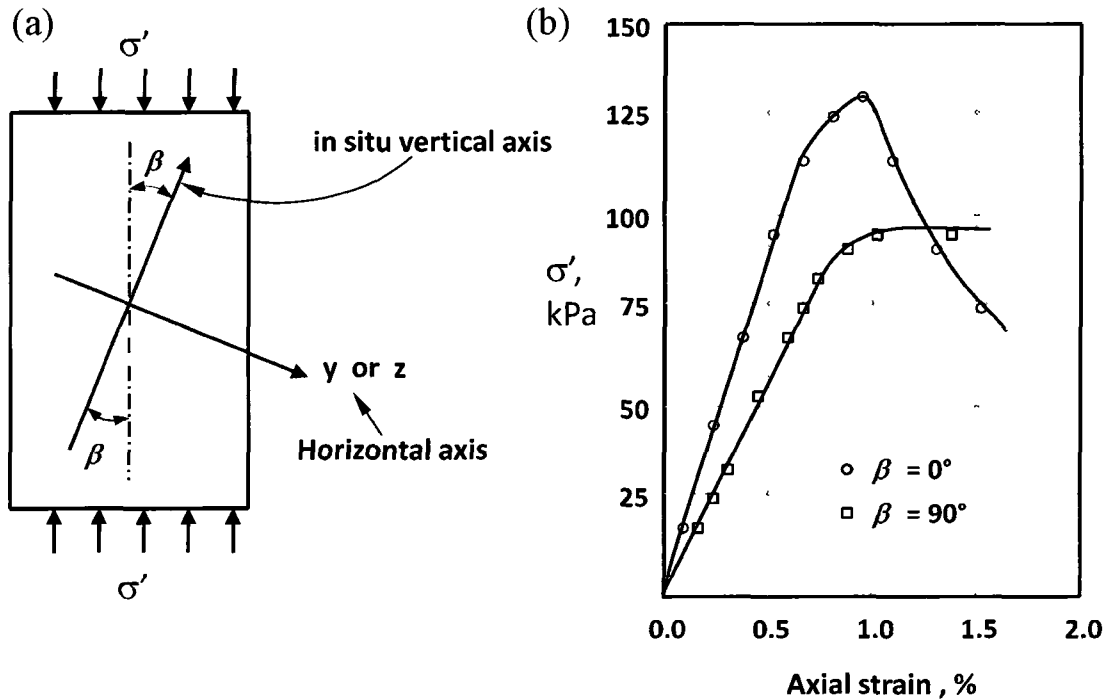


Figure 2-22: a) Inclination of clay sample; b) Unconfined compression stress-strain curves of standard triaxial tests [18].

The above figure shows that the strength of vertically trimmed samples ($\beta = 0^\circ$) is 1.41 times the strength of the horizontally trimmed samples ($\beta = 90^\circ$) and E_h of ($\beta = 0^\circ$) is 1.6 times E_v of ($\beta = 90^\circ$). In addition, stress strain curves of both standard and true triaxial tests indicated nonlinear behavior beyond the failure stresses of the clay (see figure 2-23) [18]

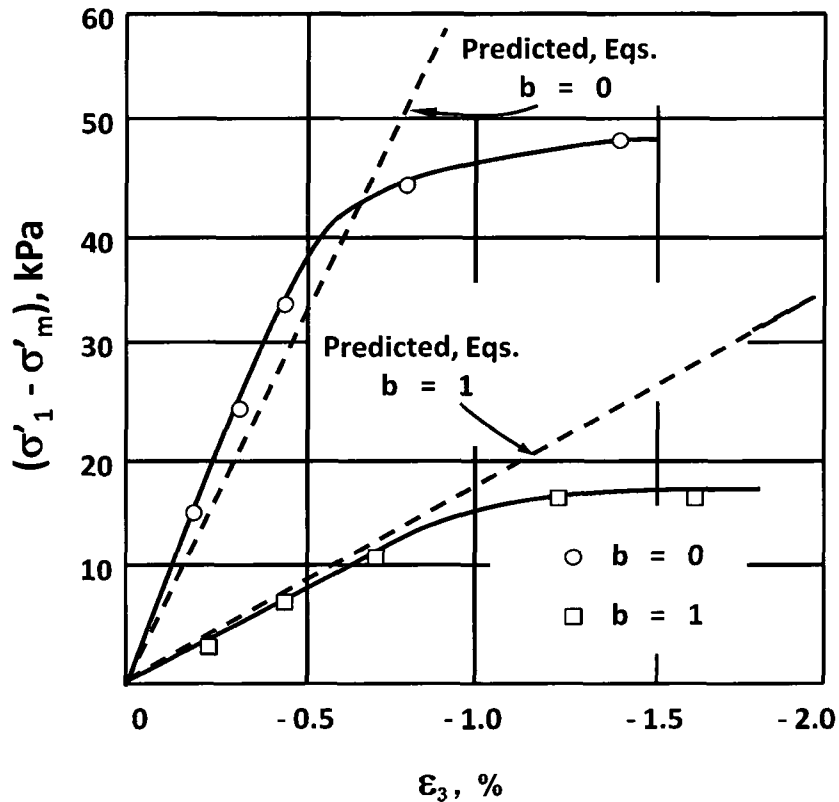


Figure 2-23: Stress-strain curves for strain controlled tests [18]

In order to model anisotropic stress-strain behavior, Yong and Silvestri [18] predicted the parameters of Barden's [21] formula. By applying confining pressures less than the preconsolidated clay stress and using experimental stress-strain curves, the stress-strain parameter μ_{hh} , μ_{hv} and μ_{vh} was obtained. Within an elastic range, the results showed that Barden's [21] formula predicted well the anisotropic stress-strain behavior of St-Louis clay. However, after reaching the peak failure stress, the formula results do not match with the experimental results at all (see Figure 2-24). This conclusion is obvious since the calculated parameters are only good for an elastic range.

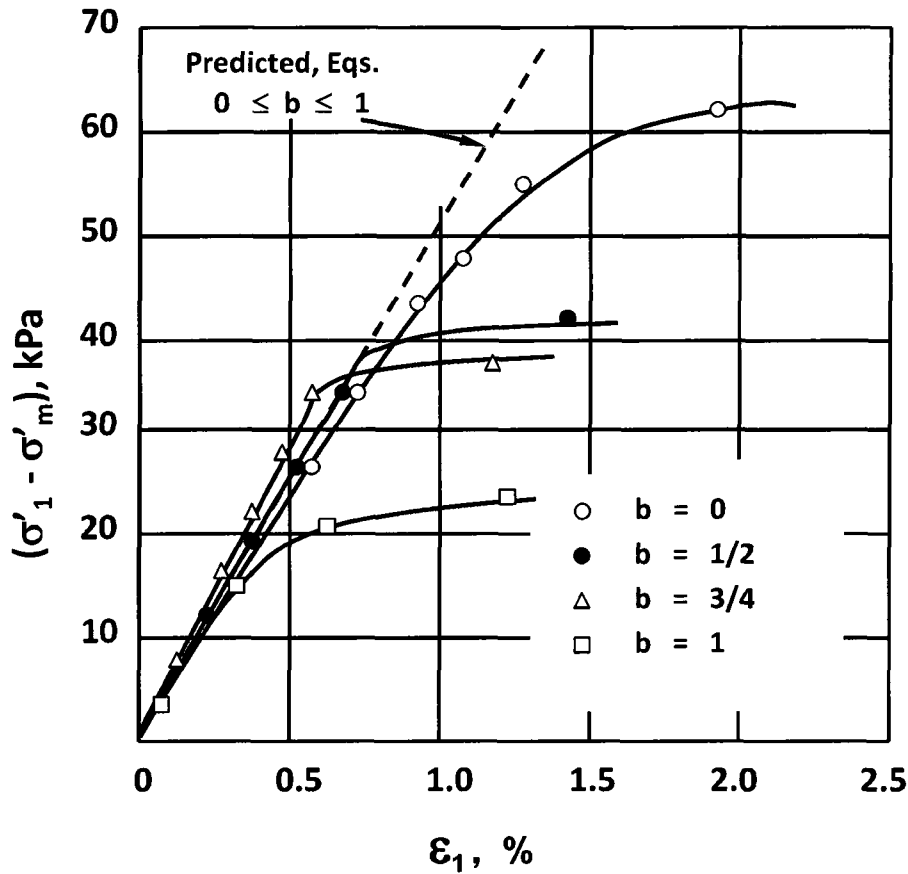


Figure 2-24: $(\sigma'_1 - \sigma'_m)$ vs. axial strain: $\sigma'_m = 69$ kPa [18].

Rodríguez et al. [22] studied the undrained stress-strain behavior of undisturbed sensitive clay in Mexico City. Using the conventional triaxial cell, the researchers studied the effect of subjecting Mexican sensitive clay to different strain rate levels to explore its stress-strain behavior. Furthermore, they studied the effects of yield stress increase of soil on the stress-strain behavior by subjecting the soil to different ratios of yield stress and imposed effective confining pressures, i.e. (σ'_y / σ'_o) . The researchers used $\sigma'_y / \sigma'_o > 1$ to represent overconsolidated specimens and $\sigma'_y / \sigma'_o < 1$ to represent destructured normally consolidated specimens [22].

The strain rates that were used in the undrained testing are $\dot{\epsilon} = 1\%$, 5% , 100% and 800% with ratios $(\sigma'_y / \sigma'_o) = 2.4, 1.2, 0.6,$ and 0.32 and showed that strength

increases when the applied confining effective pressure σ'_o and the strain rate are increased (see Figures 2-25 a, b).

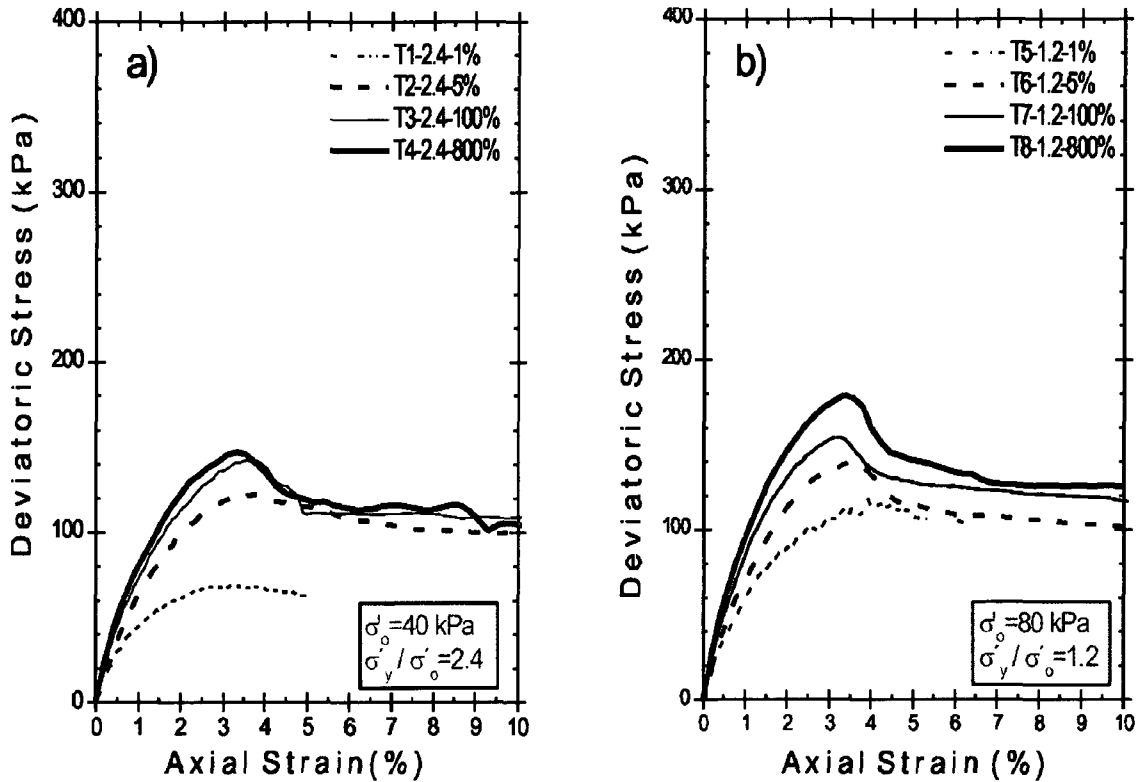


Figure 2-25: Stress-strain curves for triaxial comparison tests on Mexico City soil: a) Confining stress = 40 kPa, OCR=2.4. b) Confining stress = 80 kPa , OCR = 1.2 [22].

However, at peak deviator stresses, the axial strain at failure ($\dot{\epsilon}_f$) showed independent behavior from the strain rate changes. Also, higher $\dot{\epsilon}_f$ values were recorded in destructured normally consolidated specimens in contrast to structured overconsolidated specimens (see Figures 2-26 c, d) [22].

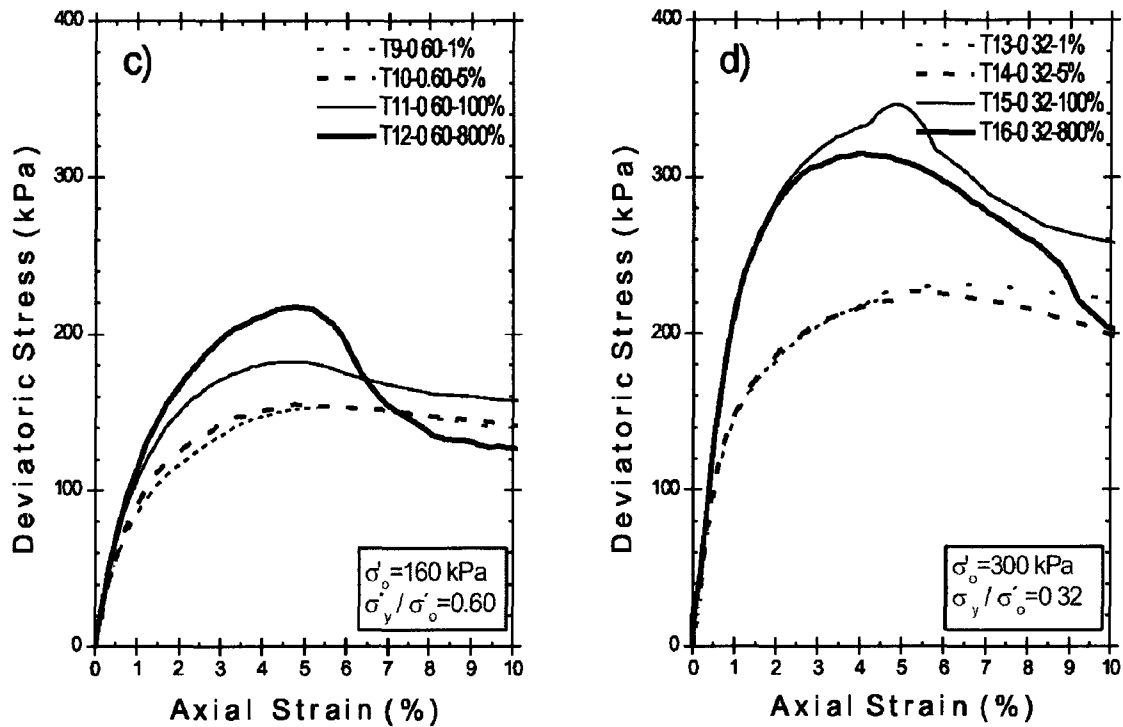


Figure 2-26: Stress-strain curves for triaxial compression tests on Mexico City soil: a) Confining stress = 160 kPa, OCR=0.6. b) Confining stress = 300 kPa, OCR = 0.32[22].

2.2.3. In-situ undrained shear strength

In a paper presented to the ASTM International Symposium on Laboratory and Field Vane Shear Strength Testing, Lefebvre et al [23] showed a comparison of field vane (FV) and laboratory C_U strengths in soft sensitive clays. Their research provided a valuable analysis on the reliability of FV in-situ testing on sensitive clay deposits in northern Quebec [23].

The research provided the results of three laboratory tests on the Quebec sensitive clay: the consolidated undrained triaxial compression, extension, and direct simple shear tests. The shear stresses obtained from these tests were normalized by a measured preconsolidation stress σ'_p . The plot data from the triaxial compression shows a high peak strength at low shear strain ($\gamma = 0.5\%$), and then a dramatic loss of strength with large strain softening. In contrast, the peak strength is reached at larger strains with small strain softening. Also, the research noted that the triaxial

compression strength is larger than that of the direct shear and triaxial extension tests (see Figure 2-26) [23].

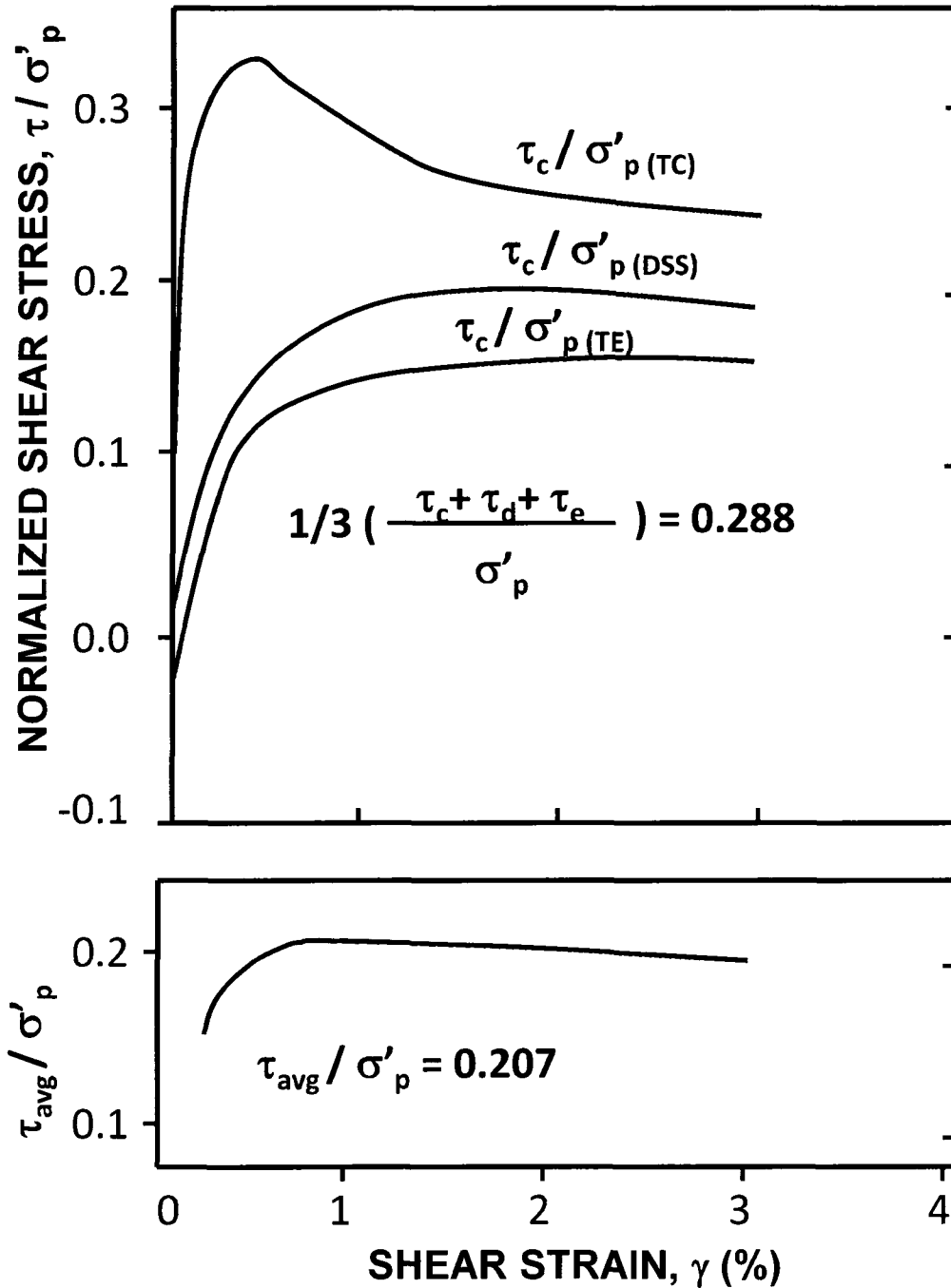


Figure 2-26: Stress strain data after the application of the strain compatibility technique. Average of peak strength defines a maximum $1/3 (\tau_c + \tau_d + \tau_e / \sigma'_p) = 0.288$. In comparison, maximum average strength τ_{avg} is less by 10% i.e =0.207[23].

For the purpose of a field and laboratory comparison, the FV strength and laboratory C_u shear strength profiles are graphed together throughout the sensitive clay layer, which extends from 5.5 to 12.5 m in depth [23] (see Figure 2-27) [23].

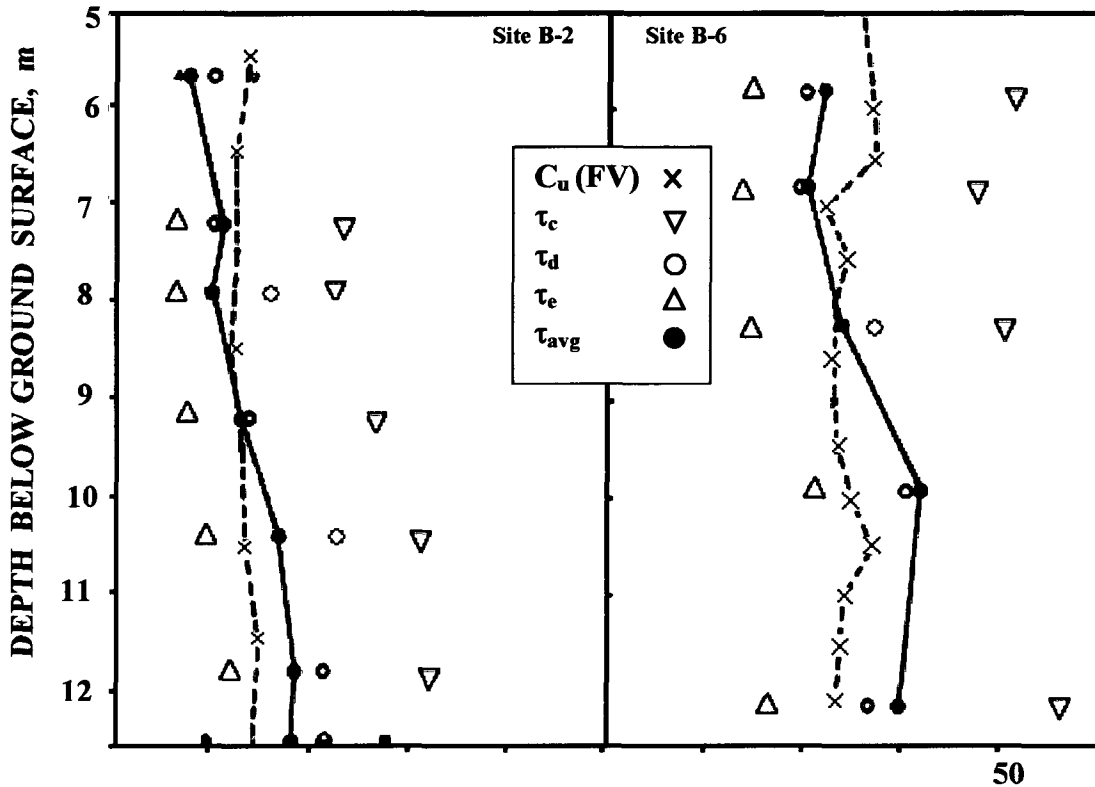


Figure 2-27: Comparison of FV and laboratory C_u strength[23].

The above figure shows that the FV strength C_u is much lower than the peak strength in a triaxial compression test, τ_c . However, the FV strength C_u is much higher than the peak strength in a triaxial extension test, τ_e . In spite of this behavior, the laboratory normalized average shear strength, for the two deposits, are found to be identical to the normalized FV shear strength after strain compatibility was imposed (see Table 2-4) [23].

Table 2-4: Mean values of normalized undrained shear strengths at B-2 and B-6[23].

Site	$C_u (FV) / \sigma'_p$	τ_c / σ'_p	τ_d / σ'_p	τ_e / σ'_p	τ_{ave} / σ'_p
B-2	0.185	0.27	0.215	0.14	0.185
B-6	0.225	0.345	0.23	0.17	0.225

2.3. Background information on the interface problem

2.3.1. Interface definition

The soil-structure interface is the soil zone in immediate contact with structural material at which a shearing mechanism occurs as a result of the load reaction transition between soils and substructures. The interface zone is the medium of interaction between structures and soils (eg. see fig 2-28).

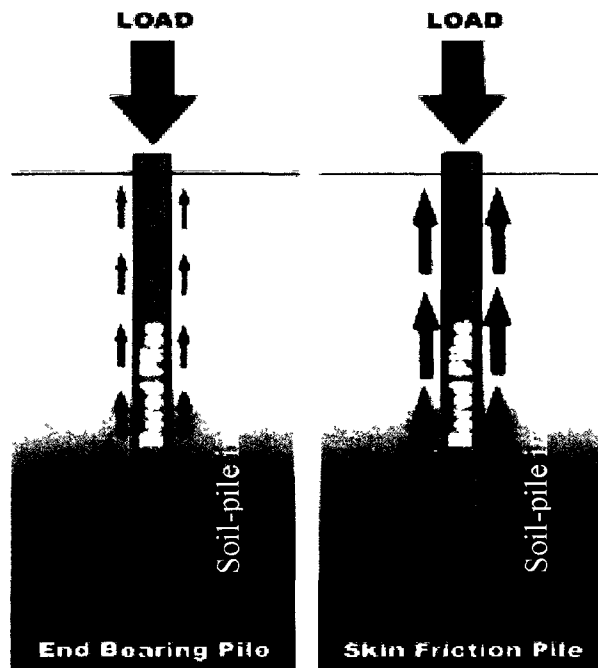


Figure 2-28: Soil-pile interface interaction. (Source: Bored Piled Foundation)

Therefore, the interface problem has been of major concern in the design retaining walls, deep foundation, cutoff walls and earth reinforcements [24].

2.3.2. Interface thickness

Several researchers investigated the thickness of the shear zone of granular soils in contact with structural material. Tracking the particle behavior near the interface using close-up photographs and revealed the development of a shear zone having a thickness of $5D_{50}$ within the sand mass along the rough interface [25]. Other researchers observed a shear zone thicknesses ranging from $2D_{50}$ to $10D_{50}$ [26][27]. The D_{50} is the mean particle size that partially determines the shearing mechanism of a particle, whether it is interlocking, dilation or translation along the interface [28]. In comparison with the past researches on granular interfaces, to the best of author's knowledge, the interface shear zone in cohesive soil has not been established very well in past cohesive soils interface researches.

2.3.3. Interface roughness

Surface roughness is important parameters that affects the frictional and shear behavior between soils and construction material. Several methodologies have been carried out to quantify the surface roughness. Japanese researchers employed a value called R_{max} to measure the surface roughness of construction material. R_{max} equals the distance from the highest peak to the lowest trough within a gage length $L=0.2$ [29].

Gokhale and Drury [30] introduced a surface roughness parameter called R_s calculated from the measurements performed on the vertical section fracture profiles as follows

$$R_s = R_L \cdot \psi$$

$$\text{Where } R_L = \frac{\text{Profile length}}{\text{Projected Length}}$$

$$\psi = \int_0^\pi \sin \theta \int_0^\pi |\cos(\theta + \pi/2 - \alpha)| \cdot f(\alpha) d\alpha d\theta$$

Where α is the angle between the tangent to an arc element on the fracture profile and the vertical axis and θ is a dummy variable of integration

Uesugi, and Kishida [31] adopted the relative interface roughness value R_n to quantify the surface roughness:

$$R_n = R_{\max}/D_{50}$$

R_{\max} is the vertical distance between the highest peak and the lowest trough and D_{50} is the mean grain size.

Dove and Jarrett [32] quantified the surface topography using a Taylor Hobson S3F stylus profilometer. The surface topography was described using Peak to valley asperity height (R_t), average mean line spacing (S_m), root spacing (S_r), average asperity angle (Δ_a), and nominal asperity angle (see fig 2-29).

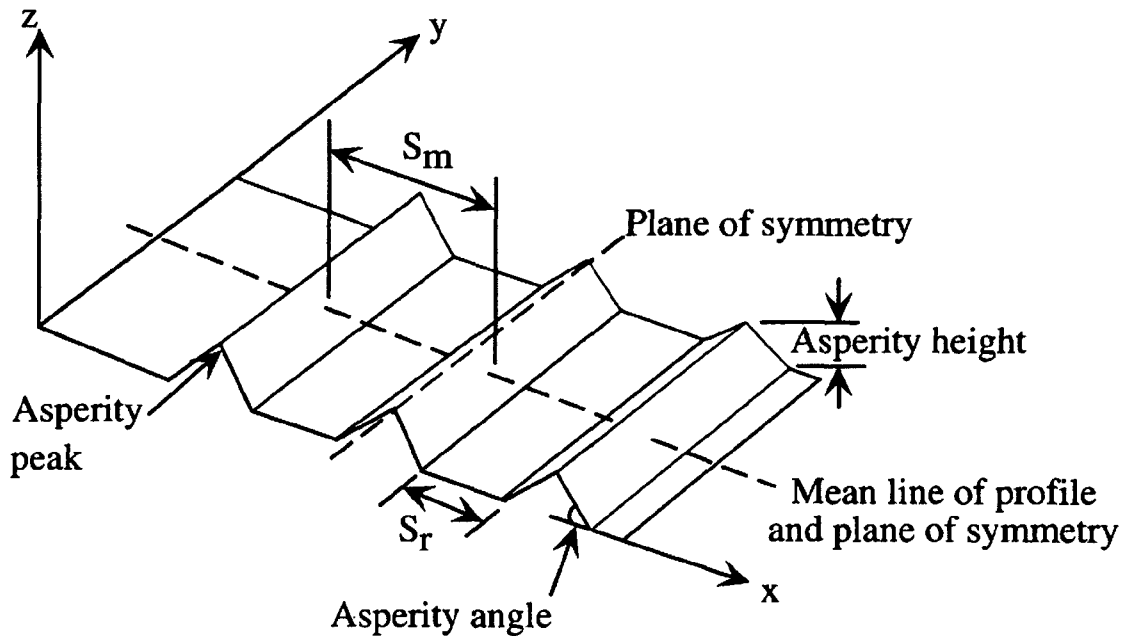


Figure 2-29: interface profile parameters [32].

Unlike the 2D R_{\max} used by kishada and Uesugi[31], the value R_t in Dove and Jarrett[32] research is based on the entire surface profile and gives better estimation to the effect of a varying surface roughness on the interface shear behavior. However, this method is limited by the rough assumption that asperities have triangular shape.

The review of the above methodologies indicates that there is a paucity of real representation and measurement of the entire 3D surface profile i.e., the real topography modeling of the interface materials. This would result in the fact that surfaces may have the same measured roughnesses but different topography. This could affect the interface shear response.

2.3.4. Shear strength parameters

In geotechnical engineering, the direct shear apparatus has been used to obtain the shear strength parameters of cohesive and non-cohesive soils. One of the shear strength parameters is stress dependent component and is called the angle of internal friction, symbolically denoted as ϕ . The other component is related to the intrinsic cohesion of soil and is represented by the symbol c [10]. These two components are used to determine the shear strength of soils as defined by Coulomb's equation:

$$\tau_f = \sigma \tan \phi + c$$

Where τ_f : The shear strength of the soil

σ : The applied normal stress

The above relation can be graphed as a straight line plot (see fig 2-30)

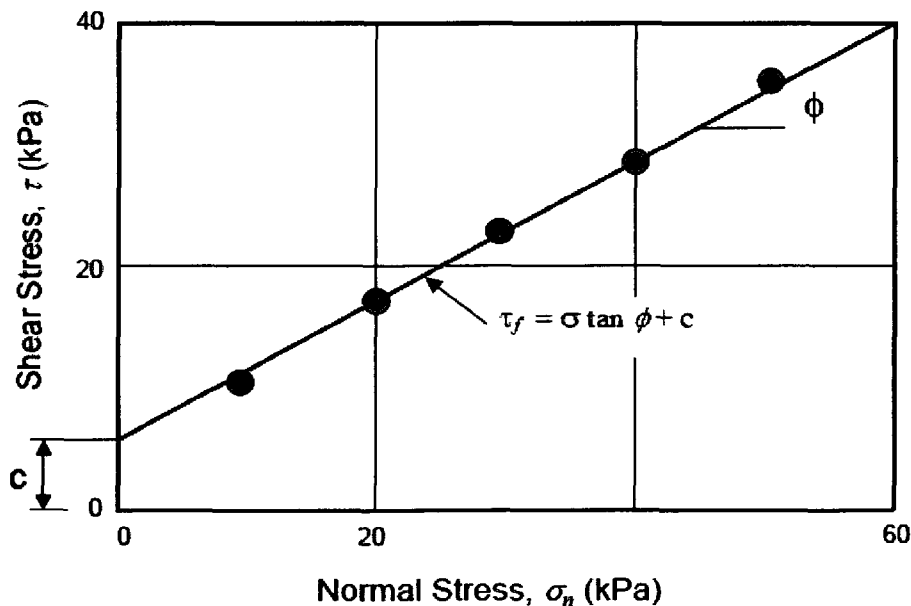


Figure 2-30: Typical graph of Coulomb strength equation[34].

A typical consolidated drained Mohr failure envelopes for remolded clays and normally consolidated undisturbed clay shows cohesion intercept as zero (see Figure 2-31) [10]

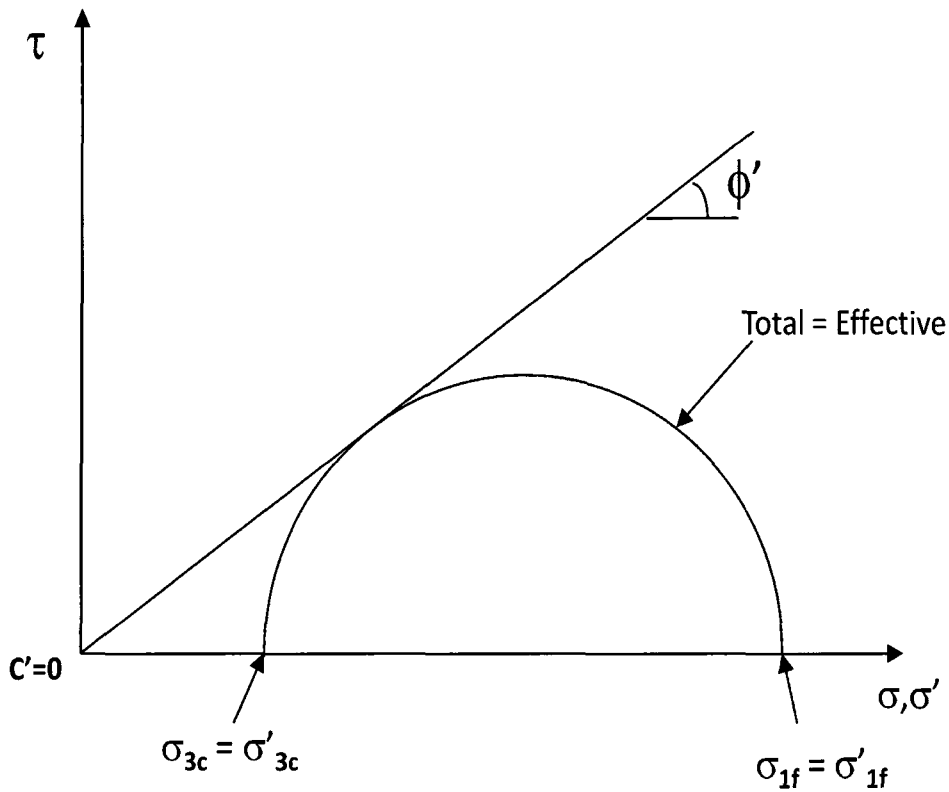
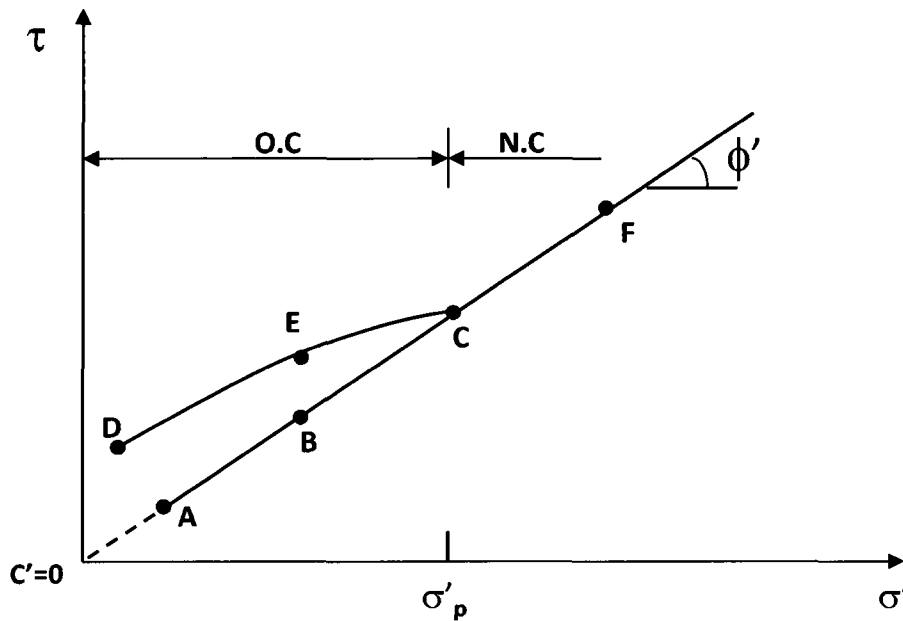


Figure 2-31: Typical mohr failure envelope for a normally consolidated clay in drained shear test [10].

The c' value of Mohr Coulomb's failure envelope for overconsolidated calys is greater than zero Also, the overconsolidated portion of the shear strength envelope lies above the normally consolidated envelope [10] (see figure 2-32)



2-32: Mohr failure envelope (DEC) for overconsolidated clay [10].

Direct shear tests on sensitive marine clays revealed that the cohesion intercept for normally consolidated specimens goes to zero while a significant cohesion intercept was recorded for overconsolidated marine clays [16][33].

In the interface problem, the shear strength parameters are: (i) the interface friction angle δ , and (ii) The adhesion value α . These parameters are the main components of the interface Mohr-Coulomb failure envelope.

$$\tau_f = \sigma \tan \delta + \alpha$$

the above mentioned relationship has been widely accepted as a failure criterion for interface shear testing [34].

2.4. Summary and conclusion

The chapter 2 presents a review on the sensitive marine clays from two main perspectives (i) geological, and (ii) geotechnical

Leda clay is sensitive soft marine clay It is the product of sedimentation in proglacial lake basins which existed between 18000 and 6000 years BP The sensitivity of Leda clay is governed by its geochemical and mineralogical properties The main clay mineral that exists in the sensitive clays is illite Other minerals include quartz, feldspar and chlorite The dominant cation in Leda clay is Na at all depths Other cations may include K, Mg, Ca, Mn, Sr and Fe

Leda clay structure plays an important role in its mechanical behavior The intact clay structure is anisotropic, characterized by small platelets and separated by regular porous networks These regular porous networks define a large interaggregate porous medium that could hold water contents beyond the liquid limit Therefore, Leda clay is considered to have a “card-house” type of structure that would experience noticeable particle reorientation when consolidated The anisotropic consolidation of sensitive clay produces reorientation of the clay flakes in a plan perpendicular to the major principle consolidation pressure Beyond the preconsolidation pressure, greater parallelism and flake reorientation was observed with larger reduction in void ratio

The review on the mechanical behavior of sensitive marine clays included the results of direct shear tests, triaxial drained and undrained tests and in-situ vane shear tests The direct shear tests on a selection of four sensitive marine clays indicated that all undisturbed clay samples demonstrate brittle behavior with higher (τ_f) and smaller D_f than those in the normally consolidated or remolded state The drained triaxial tests conducted on undisturbed sensitive marine clay confirmed its anisotropic structure and indicated a nonlinear beyond the failure stresses of the clay The undrained triaxial tests using different strain rates have shown that the strength increases with an increase in the applied confining effective pressure and the strain rate The comparison between field vane and laboratory undrained shear testing

revealed that the field vane shear strength is much lower than that obtained from the triaxial comparison. However, the field vane strength is much higher than that obtained from triaxial extension. On the other hand, the laboratory normalized averaged shear strength were found identical to that of the field vane shear strength after strain compatibility treatment.

Finally, chapter 2 presented an overview on the interface definition, thickness, roughness measurement and shear strength parameters.

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Chapter 3

Technical Paper I-

Shear behaviour of sensitive marine clay- steel interfaces

A. Taha , M. Fall

ABSTRACT

Many soil-structure interaction problems require the knowledge of the shear resistance and behavior between the soil and construction materials. Although sensitive marine clay deposits are widely found in Canada (Leda clay) and many regions in the world (e.g., Scandinavia), and steel is a common construction material for many civil engineering structures, our understanding of the interface shear behavior between sensitive marine clay and steel is still limited. This paper presents the results of an experimental study on the interface shear behavior between Leda clay and steel. In this research, direct shear tests (DTSs) are conducted to investigate the interface shear strength parameters and behavior between Leda clay and steel, and the effect of several factors (e.g., steel surface roughness, properties of the Leda clay) on the interface shear behavior and parameters. All tests have been carried out with a standard DST apparatus at normal loads which range from 250 to 450 kPa. The results show that the Leda clay interface shear behavior can be significantly affected by the steel surface roughness, the Leda clay's OCR, degree of saturation, dry density, and salt content. The results presented in this paper will contribute to a more cost-effective design of geotechnical structures in Leda clay.

3.1. Introduction

Sensitive marine clays are found in many countries and regions around the world, such as Canada and Scandinavia. The sensitive marine clays of Canada and Scandinavia are young deposits. They have resulted from sedimentation of glacially ground rock flour in basins that had been isostatically depressed by glacial ice sheets. They have been subsequently uplifted above sea level and significant post-depositional changes in properties have occurred, primarily as a result of the leaching of salt from the pore water and soil profile development in the surface zone (Kondo and Torrance, 2005). The leaching resulted from the fact that these clays can have high sensitivity and exhibit quick clay behavior in many cases (e.g., Rosenqvist 1953, Crawford 1963). The most striking feature of these marine clays is their tendency to considerably lose their strength when they are disturbed. Therefore, these clays are problem soils and represent a serious geotechnical hazard. They are responsible for many geotechnical problems, such as landslides (e.g. Scöld et al. 2005 and Okamoto et al. 2004) and foundation damages (e.g. Gregersen and Loken 1979) in the aforementioned regions.

The sensitive marine clays are often called "Leda" clays (also sometimes called Champlain Sea clays) in the Canadian geotechnical engineering literature. Thick deposits of Leda clays cover large areas of the provinces of Quebec and Ontario. The existence of the Leda clays in these areas has made infrastructure expansion challenging for geotechnical engineers. Since the aforementioned areas are fairly heavily populated and industrialized, the importance and behavior of Leda clays as a foundation soil have led to several investigations of their geotechnical properties (e.g., Crawford 1963, Leroueil 1999). At the locations where the Leda clays exist, many foundation engineers have adopted the use of pile foundation as subsurface supporting (heavy) structures to ensure the safe transmission of loading deep into a harder stratum (e.g., moraine, rock). Long end bearing piles, driven through the Leda clay to the deep underlying harder stratum, are often used when heavy loads must be supported (Roy et al. 1981). In many cases, short friction piles in the clay would

provide much less expensive foundation solutions. However, their use has been relatively limited despite their lower cost (Roy et al. 1981). This is mainly due to the many unknown aspects of their geotechnical behavior in the Leda clays. In particular, the interface shear behavior between pile materials (steel, concrete, and wood) and Leda clay is not sufficient known. To design reliable and cost-effective pile foundation (especially friction piles) structures, it is crucial to understand the mechanical behavior at the interface of the pile-Leda clay. This is especially true with regard to interface strength properties used in stability analyses.

Several studies have been conducted to understand the interface shear behavior between pile construction material and cohesionless soils (e.g., Yoshimi and Kishida 1981; Evgin and Fakharian 1996) and the factors that affect it. Compared to the great number of research carried out to study the interface behavior of granular soils, only a few researchers (e.g., Tsubakihara et al. 1993, Tsubakihara and Kishada 1993, Zimnik et al. 2000; Hammoud and Boumekik 2006, Tan et al. 2008, Tariq and Gerald 2009) have conducted studies on the interface shear behavior between cohesive soils and structure. The previous research findings have significantly contributed to better understanding the interface shear behavior of soil-structure. They revealed that the surface roughness, water content, soil composition and structure, and the intensity of the normal load have significant influence.

However, since Leda clay is a special soil and different from the “normal” cohesive soils, the results of the aforementioned studies cannot be directly applied to the Leda clay-structure interface. Thus, there is a need to gain knowledge about the interface shear behavior of Leda clay-structure for a safe and cost-effective design of pile foundation in these soils. Moreover, this understanding is not only important for pile design, but also crucial for the cost-effective design and accurate performance predictions of other relevant civil engineering structures (e.g., retaining walls, reinforced earth, shallow foundations, buried structures (e.g. lined tunnel)). However, no studies have addressed the interface shear behavior between Leda clay and structure.

In consideration of the facts mentioned above, a research program has been conducted at the University of Ottawa to study the interface shear behavior between Leda clay and steel. The objective of the study is to present and discuss the results of shear tests performed at the interface of Leda clay-steel. Furthermore, the effects of steel surface roughness, initial saturation degree, OCR, dry density, and salt content of Leda clay on the interface shear behavior will be presented discussed.

The paper is organized as follows. The next section presents the experimental setup and materials. Then, the experimental results will be presented and discussed. Finally our conclusions are given.

3.2. Experimental programs

3.2.1. Testing apparatus

The interface shear behavior of soil-steel can be tested by using one of a number of tests (e.g., simple shear test, the torsion or ring shear test). The advantages and disadvantages of each method have been summarized by Uesugi and Kishida (1987) and Paikowsky et al. (1995). Despite some inherent problems (e.g., principal stress rotation, stress non-uniformity, and failure plane definition), the DST apparatus is a commonly used device for interface testing due to its simplicity and well suitability for interface testing (Miller and Hamid, 2004). Hence, a standard DST machine was used to conduct the shear tests on the Leda clay-steel interface. This DST apparatus uses a microprocessor controlled drive system and keyboard entry which provides the machine with a wide range of features that include pause and speed changes from 0.00001 to 9.99999 mm/minute. Normal loads can be applied to the specimen by adding weights to the weight hanger. The changing shearing stresses are recorded by a mounted loading cell that is horizontally fixed to a shear box. The vertical deformation is recorded through an LVDT that is vertically mounted on top of the loading yoke. The horizontal displacement is recorded through an LVDT that is horizontally fixed to a shear box. All recorded data were gathered by using a

computerized data logging system. The results were automatically monitored and saved by using the LabVIEW software.

3.2.2. Material used in the experimental programs

3.2.2.1. Leda clay

The undisturbed Leda clay material used in the present study is collected from the Ottawa area. The samples were recovered from a depth that ranged from 8 to 12 meters. The mineralogy of the Leda clay was dominated by quartz, feldspars, illite, chlorite and amphibole (Kondo and Torrance, 2005), which is typical of Leda clays. However, it should be mentioned that the mineralogical composition of Leda clay is dependent on the geographical location (Law and Bozozuk, 1988). Standard laboratory geotechnical tests were carried out on the clay samples to obtain their basic geotechnical characteristics. Table 3-1 summarizes the main properties of the Leda clay used whereas the relevant soil index parameters are discussed below.

The Atterberg limits test was carried out per ASTM D 4318. The LL of the used Leda clay is 66% and the plastic limit is 25%. These results match with the findings of Leroueil (1999) who stated that the LL of eastern Canada clays are less than 83% and the plastic limit is between 17% and 35%. The calculated liquidity index is 1.4. This value is in accordance with the findings of Law and Bozozuk (1988), who mentioned that Leda clay usually has a liquidity index in excess of 1.0; this gives the clay a quick condition when remolded (Law and Bozozuk, 1988). The water content analysis showed that the water content of all samples is beyond their LL values. This can be attributed to the open fabric structure of Leda clay which can hold a high content of water without flowing and undergoes sudden collapses upon remolding. A particle size analysis of Leda clay was performed per ASTM D422. As shown in Figure 3-1, an 84% fraction at 2 μm in particle diameter indicates high clay fraction in this Leda clay. Its activity, calculated as 0.47, falls within the Eastern Canadian activity range that was defined by Leroueil (1999) to be between 0.25 and 0.75. Based on the above testing results, the used Leda clay can be classified with respect to the Unified Soil Classification System (USCS) as inorganic clay of high

plasticity (CH). This result is in agreement with that obtained by Locat et al. (1984) who examined the physical properties of sensitive clays in Ontario, Quebec and British Columbia, and found that almost all of the samples are classified above the “A” line (Bell, 2000). The determination of the specific gravity was based on ASTM D845. The specific gravity of the samples computed as 2.74 was found to fall within the specific gravity range defined by Leroueil (1999), i.e. between 2.7 and 2.8. The soil-water characteristic curve (determined by using the WP4 Dewpoint potentiometer) of the Leda clay is illustrated by Figure 3.2. The air entrained value of the studied soil is around 400 kPa (Figure 3-2). One-dimensional consolidation tests in accordance to ASTM D 2435 were carried out to obtain the pre-consolidation pressure, OCR and time required to reach the 50% consolidation of the Leda clay. An average pre-consolidation pressure and C_c of 150 kPa and 2.44, respectively, were obtained. Leroueil (1999) developed a relationship between the natural void ratio of Leda clay and the C_c . According to this relation, the C_c can be expressed in the following equation as:

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

The calculated C_c is 2.34 which matches very well with the C_c obtained from the consolidation test. The obtained value of the OCR ratio (1.1) of the Ottawa Leda clay is close to the previous finding of Leroueil’s (1999) research which indicated that the OCR of the Champlain Sea clay varies between 1.2 and 5. The vane shear tests were carried out per ASTM D 4648 – 05. The laboratory C_u is 9.7 kPa, the remolded shear strength is 1.7 kPa, and the calculated sensitivity is 6. This sensitivity matches well with the test results of the sensitivity of Ottawa’s Leda clay at a depth of 12 m, as indicated in Burn and Hamilton (1868). Moreover, in Roy et al.’s (1981) undrained FV tests in Quebec, the C_u Champlain Sea clay is recorded at 10 kPa.

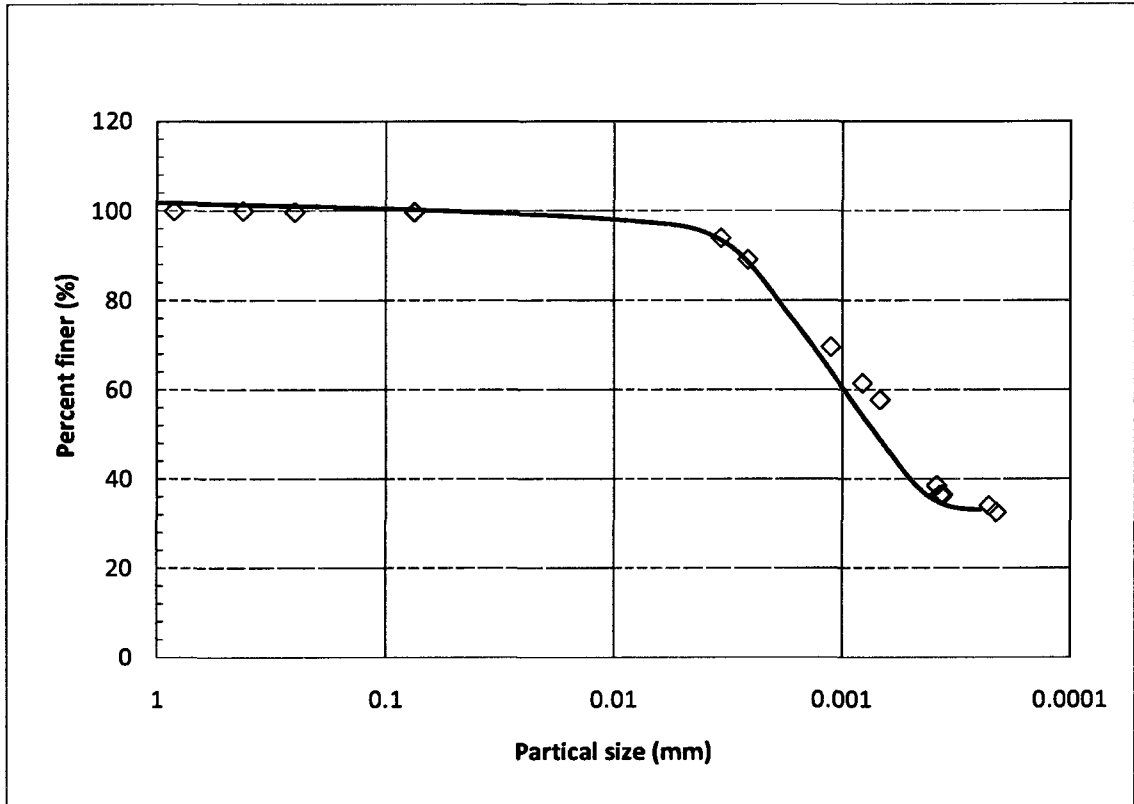


Figure 3-1: Grain size distribution of the Ottawa Leda clay used

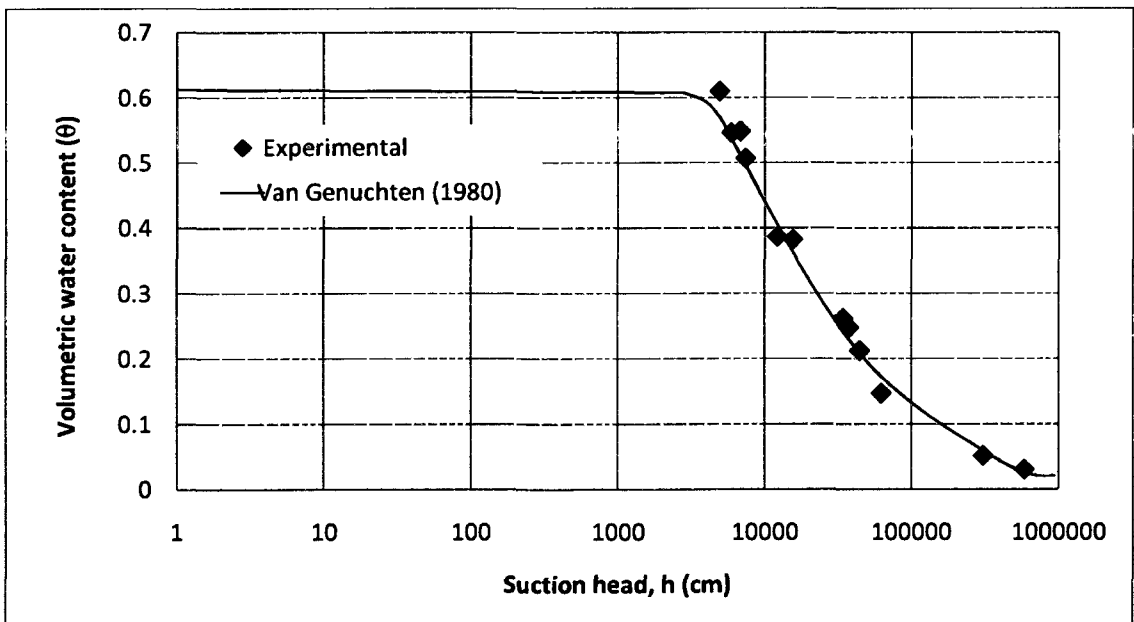


Figure 3-2: Soil water characteristic curve of the Leda clay used

Table 3-1: Summary of the main characteristics of the Leda clay used

Property	Value/Classification
Classification	CH
Water content (%)	82
Liquid limit (%)	66
Plastic limit (%)	25
Plasticity index (%)	40
Liquidity index	1.4
Clay fraction (%)	84
Activity	0.48
Laboratory vane shear strength (kPa)	9.7
Sensitivity	6
Pre-consolidation pressure (kPa)	150
Natural void ratio	2
OCR	1.1
Sodium chloride concentration	3-4 g/l
Optimum water content, Dry unit weight at optimum	22%, 1.53 g/cm ³

3.2.2.2. Steel material

The steel plates were fabricated from milled steel rods, which made up the shear box with inner dimensions of 60x60 mm and a thickness of 5 mm. Table 3-2 summarizes the main mechanical properties of the steel. The surface of each steel plate was finished to a specific surface roughness. The roughness was measured by using a high precision LVDT that has a strain sensitivity of $527 \times 10^{-6}/\text{mm}$. The LVDT together with the data logging system recorded the interface profile. The surface roughness was evaluated based on a method proposed by Yoshimi and Kishada (1981), and Uesugi and Kishada (1986). The average maximum surface roughness (R_{max}) was 20 μm , 5 μm and 1 μm .

Table 3-2: The properties of the steel plates (at temperature = 25° C)

Properties	Value
Density ($\times 1000 \text{ kg/m}^3$)	7.7-8.03
Poisson's ratio	0.27-0.30
Elastic modulus (GPa)	190-210
Tensile strength (Mpa)	394.7
Yield strength (Mpa)	294.8
Hardness (HB)	111
Impact strength (J)	123.4

3.2.3. Test specimen preparation

Undisturbed Leda clay samples directly obtained from Shelby tubes were cut from a vertical profile into squares of approximately 60 x 60 x 12 mm in size to fit the top half of the shear box. These samples are used to study the interface shear behavior of normally consolidated Leda clay-steel. They are also used to investigate the effects of steel surface roughness and OCR on the interface shear behavior. Prior to placement in the shear box, the square samples were covered with saturated filter papers and placed on top of porous stones. The porous stones were kept submerged in distilled water in a humid room until they reached complete saturation. Trial studies have shown that specimens attain saturated conditions within a period of 24 hours. As well, no volume increase of the soil sample was measured. However, to study the effect of the initial saturation degree, density and salt content of the Leda clay on the interface shear behavior, further preparation of the clay samples is necessary as described below.

In the preparation of the clay samples to study the effects of various initial saturation degrees or matrix suctions, some of the undisturbed Leda clay samples were first saturated and then subjected to different increments of air-drying to produce variations in the degree of saturation from specimen to specimen and soil suction (i.e., the soil suction in the samples increase as the drying time increases). The air dried soil specimens were covered and placed in an airtight plastic bag and left until they attained equilibrium conditions with respect to water content and suction

throughout the specimen. The dried samples did not show any significant shrinkage during air drying. The water content and the total density of the air dried soil specimens were measured and used in the calculation of basic phase relations. Thus, samples with 96%, 90% and 60% initial saturation degrees were prepared. These saturation degrees correspond to initial suction values of 500 kPa, 670 kPa and 1180 kPa, respectively. The suctions of the prepared specimens for different values of degree of saturation were estimated from the soil-water characteristic curve data that was determined in the previous section.

In the preparation of clay samples to study the effects of different dry densities or dry unit weights, remolded Leda clays were left to completely air dry and then crushed. After that, they were mixed with specific volumes of water and then statically compacted in a mold of known volume. In this test series, three dry density or unit weight points are selected from the curve: two on the dry side and one at the optimum. After that, compacted clay samples with the same dry densities or unit weights and water contents of the selected points were prepared. This is done by adding the required amount of dry soil and water per the basic phase relations, and then by statically compacting the wet clay until achieving the required dry densities. The samples were then used for the interface shear testing.

To examine the effects of increasing the sodium chloride (NaCl) content in Leda clay, the undisturbed specimens were left to absorb the NaCl on top of porous stones which were submerged in salt solutions with two different concentrations (10 g/l, 20 g/l) for a minimum of 15 days. These concentrations are higher than the original salt content (~3 g/l) of the undisturbed saturated Leda clay. Thus, the samples were prepared and used in the interface shear test. An inductively coupled plasma atomic emission spectroscopy (ICP-AES) analysis allowed us to determine and verify the salt content of the prepared samples.

3.2.4. Test procedures

A series of consolidated drained (CD) and undrained (CU) direct shear tests were carried out to study the shear behavior of the interface between Leda clay and steel.

3.2.4.1. Consolidated drained interface shear tests

A series of direct shear CD tests were carried out to examine the interface shear behavior of normally consolidated Leda clay sheared with respect to varying steel surface roughness. CD tests were also performed to study the effect of OCR on the Leda clay interface shear behavior. This CD interface testing series is carried out in accordance with ASTM D 3080. This procedure is selected to simulate the effect of slow loading after pore water pressure dissipation. Interfaces for shear testing were constructed by first placing the specified steel plate into the base of the shear box. The prepared Leda clay specimen was placed in the top halve of the shear box in such a way that the interface between the Leda clay and steel was located exactly between the two halves of the shear box. The prepared test samples in the shear box were submerged into water. To conduct CD on normally consolidated Leda clay, the test samples were first left to consolidate (against the interface steel plate) completely at normal stresses that are high above their pre-consolidation stress. The normal stresses (applied during the shear test) were maintained at 250, 350 and 450 kPa based on previous experimental work (e.g., Dam 1999). Furthermore, these stresses encompass a realistic range of operational stresses in geotechnical structures (Dejong and Westgateb, 2005). After the completion of the primary consolidation, the samples were sheared at a slow rate of 0.0185 mm/min in order to minimize pore-pressure development during shearing. This shearing rate is selected based on the time required to reach 50% consolidation, i.e. the t_{50} , at a 5 mm displacement to failure, and also previous work (e.g., Tsubakihara et al. 1993a). In a review of the previous work of Tsubakihara and Hiskeda (1993b), a shearing rate of 0.03 mm/min was used in their interface CD tests on Kawaski marine clay to avoid changes in pore pressure during shearing. This indicates that 0.0185 mm/min is a reasonable shearing speed.

To conduct CD tests on Leda clay with various OCRs, the samples were left to completely consolidate at stresses two to three times higher than these three normal stresses: 250, 350 and 450 kPa. After the completion of the primary consolidation, the samples were slowly unloaded to the desired normal stresses to produce three OCR values of 1, 2 and 3. The samples were then sheared according the procedures described above.

3.2.4.2. Consolidated undrained interface shear tests

A series of consolidated undrained (CU) direct shear tests were carried out to examine the interface shear behavior of disturbed and undisturbed Leda clay with different initial saturation degrees as well as on Leda clay samples with different dry densities. The soil specimens were left to consolidate under 250, 350 and 450 kPa normal stresses prior to shearing in a conventional direct shear apparatus. ASTM D 3080 is applied in this series of Leda clay-steel interface tests. After the completion of the primary consolidation, the samples were sheared at a fast rate of 1.25 mm/min. The details of the test procedure have been previously described by Lane et al. (2001). The shearing rate was selected to simulate the CU condition or rapid shearing. It can be assumed that the specimens were sheared under undrained conditions due to the relatively rapid strain rate used for shearing and the low coefficient of permeability of the soil specimens (Vanapalli and Lane, 2002). Furthermore, this is a reasonable assumption for fine-grained soils because shearing of the specimen was completed over a short period of time (i.e., 5 to 10 minutes). Vanapalli and Fredlund (2000) used similar assumptions for analyses of shear strength test results on a silty soil. The test results can be analyzed based on the assumption that there is no significant change in suction of the soil specimen during the shearing stage.

3.3. Results and discussion

In this section, typical DST results for different Leda clay-steel interfaces are presented and discussed. In general, the shear test results are presented by three types of graphs: shear stress–relative shear displacement curves, vertical displacement–shear displacement curves, and shear strength envelopes, which give the interface shear strength parameters (cohesion or adhesion, friction angle).

3.3.1. Shear behaviour of the interface between normal consolidated Leda clay and steel

Figure 3-3 shows typical shear stress versus relative shear displacement for the interface between Leda clay and steel with a surface roughness of 5 μm . Typical stress–relative shear displacement curves from DSTs that were performed on the Leda clay alone are also presented in Figure 3-3 for comparison. Typical shear stress–vertical displacement and shear stress-normal stress curves for various steel surface roughness will be presented in the next section to avoid repetition and keep the paper at a reasonable length. From Figure 3-3, it can be observed that at a normal stress of 250 kPa, the shear resistance of the normally consolidated Leda clay is higher than that of the Leda clay-steel interface. However, at normal stresses higher than 250 kPa, the shear resistance of Leda clay becomes almost close to that of the interface shear resistance. This behavior can be attributed to the remolding in Leda clay structure at higher normal stresses. It can be seen that the Leda clay-steel interface exhibits distinct peak and postpeak shear stress behavior, which is not true for the curves of the Leda clay. In other words, the Leda clay-steel interfaces exhibit postpeak displacement-softening behavior, whereas the normally consolidated Leda clay has approximately postpeak plastic behavior. However, it should be mentioned that the residual state was not reached in any of these tests (interface residual shear strength could not be achieved with the conventional DST apparatus). Figure 3-3 also shows both the interface shear strength and the corresponding shear displacement increase with applied normal stress. This is due to the fact that an increase in normal stress on

the interface shear plane results in higher frictional resistance between the Leda clay and the steel, leading to increased interface peak shear strength.

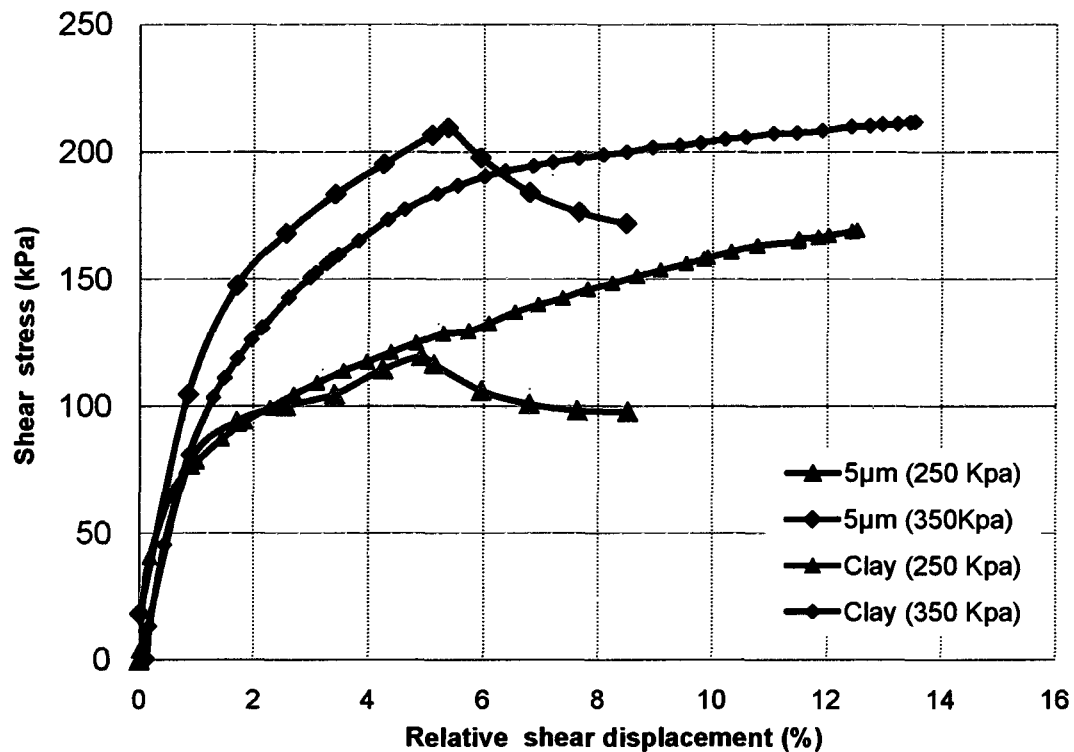


Figure 3-3 : Shear stress vs. relative shear displacement of Leda clay-steel interface and Leda clay alone

3.3.2. Effect of steel roughness on the interface shear behavior

Typical results of the effects of steel surface roughness on shear stress-relative shear displacement curves, vertical displacement-shear displacement and shear stress-normal stress curves are presented in Figures 3-4 to 3-6, respectively. Figure 3-4 shows a comparison between the shear stress and relative shear displacement curves of the Leda clay interfaces with various values of roughness at 250 kPa and 350 kPa normal stresses. It can be noted that an increase in the interface surface roughness results in an increase in the interface shear resistance.

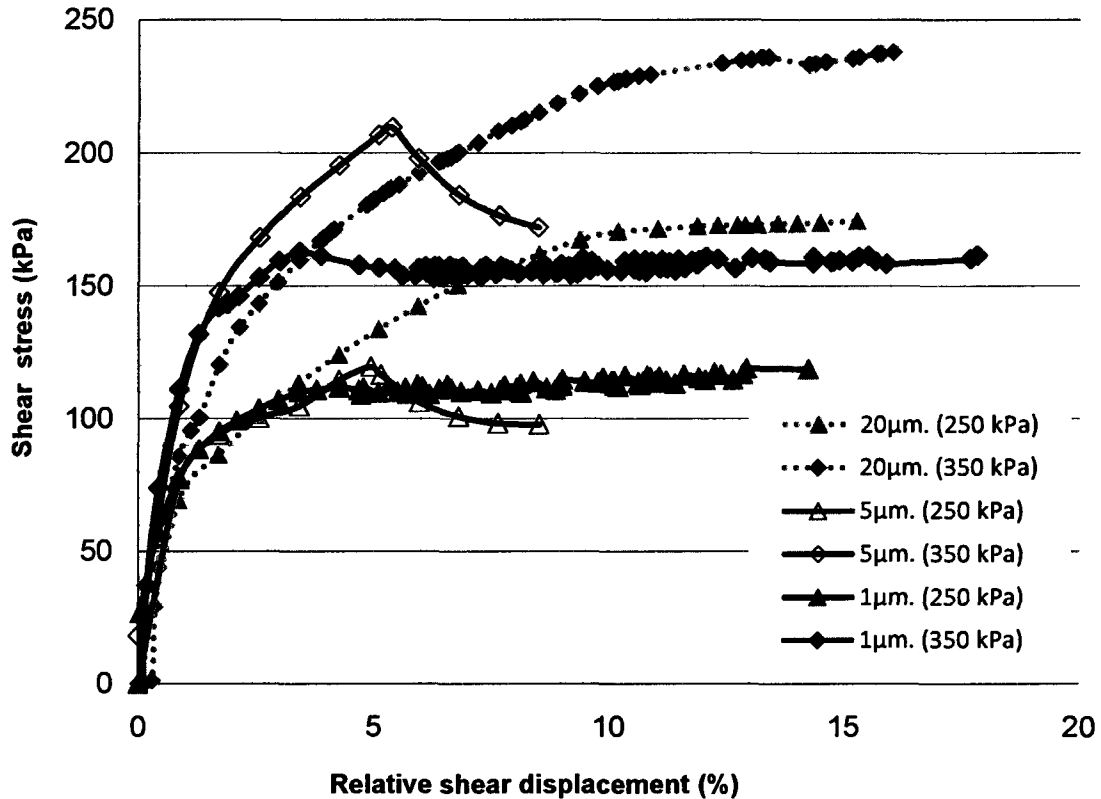


Figure 3-4: Shear stress vs. relative shear displacement of the Leda clay interface for different surface roughness.

For example, it can be seen that at a normal stress of 350 kPa, the peak shear stress of the 20µm-interface is around 12.5% higher than that of the 5µm interface. On the other hand, at the same normal stress, the peak shear stress of the 20µm interface is around 32% higher than that of the 1µm-interface. This behavior is due to the increase in the interlocking forces and contact area between the asperities of the Leda clay and steel as the surface roughness increases. This observation is similar to that made by Tsubakihara and Kishada (1993b) interface research on Kawasaki marine clay. Their research indicated an increase in the peak shear stress with the increase in steel roughness (Tsubakihara and Kishada . 1993b).

Furthermore, Figure 3-4 revealed that, at 250 kPa normal stress, the peak shear stress of the 5 μm -interface is around 8% higher than that of the 1 μm -interface. However, at a normal stress of 350 kPa the difference between the two peaks reaches around 22%. This behavior can be attributed to the fact that high normal stresses cause more parallelism and particle reorientation of the Leda clay than low normal stresses (Quigley and Thompson 1966). Therefore, at low normal stress, the open clay fabric structure is partially reserved and possessed less resistance to the interlocking forces between the clay and steel asperities. However, the dense clay structure at high normal stress possessed higher interlocking forces and hence, higher shear resistance.

Figure 3-5 shows typical examples of the impact of steel surface roughness on the vertical deformation behavior of the Leda clay interface. From this figure, it can be observed that a higher roughness is associated with a higher final contraction at the interface. This behavior can be attributed to the fact that surfaces with higher roughness have higher asperities. This induces more remolding and collapse of the clay structure during the shearing process. Tsubakihara and Kishada (1993b) recorded a similar behavior in their interface tests on Kawasaki marine clay. They observed that the volumetric strain increases as the surface roughness increases.

Also, Figure 3-5 indicates that from the beginning of shearing and beyond the peak shear displacement the vertical contraction of the 5 μm and 1 μm interfaces are almost the same. This behavior indicates that small difference in the surface roughness does not lead to significant difference in the vertical contraction of the interface. Similar conclusions were obtained by Tsubakihara and Kishada (1993b) who conducted interface shear tests on Kawasaki clay-sand mixture. They observed that the vertical contraction of the 3 μm and 10 μm interface were similar.

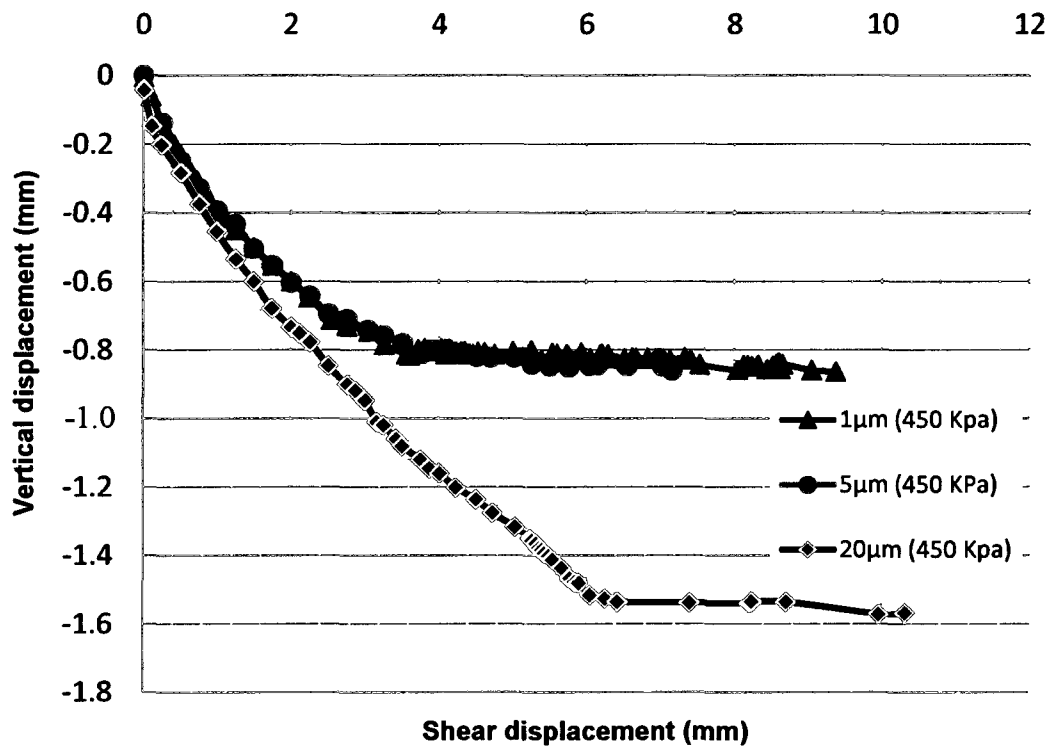


Figure 3-5: The effect of the surface roughness on the vertical deformation at 450 kPa.

Furthermore, it can be noted from figure 3-5 that the maximum vertical interface contraction reached around 13 % of the original sample height. This behavior results from the deformation of the interface and also compressibility of the Leda clay. The open fabric structure of Leda clay and the high natural void ratio ($e = 2.0$) allowed high contraction during the interface shear testing. This is supported by the results of past researches that recorded high vertical displacements during shearing as a characteristic of sensitive marine clays. Tsubakihara and Kishada (1993b) research on Kawasaki marine clay recorded a high volumetric strain of around 9.5 % from a simple shear test.

Figure 3-6 shows the typical shear strength envelopes for the Leda clay interface over various steel surface roughness. The shear strength envelope for Leda clay alone is also presented for comparison purposes. These envelopes are obtained by fitting linear regression lines through each set of interface shear stress vs. normal stress data. For all regression lines, R^2 is approximately between 0.96 and 0.99. A

Mohr–Coulomb type equation was used to obtain values of the interface friction angle (ϕ) for the range of normal stresses. From Figure 3.6, it can be seen that the angle of internal friction increases as the surface roughness increases. Furthermore, as the roughness of the surface increases, the interface friction angle approaches the internal friction coefficient of the Leda clay, which suggests that failure will occur in the Leda clay mass rather than at the interface. It is noteworthy to mention that similar observations were made by Hamid and Miller (2009) from the results of their interface DSTs on a low-plasticity fine-grained soil.

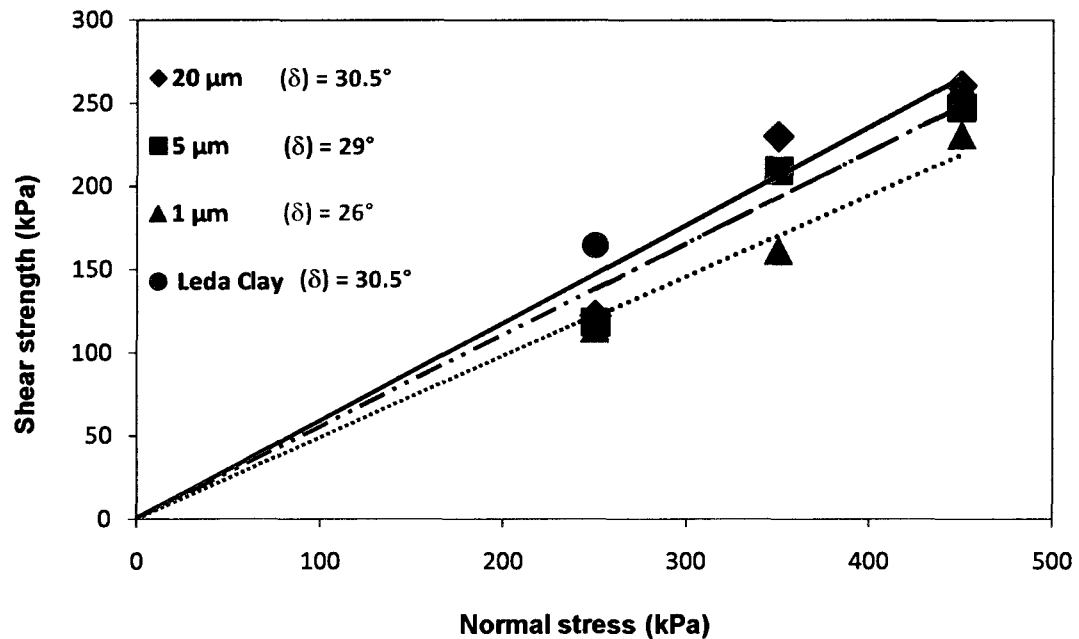


Figure 3-6: Shear stress vs. normal stress for Leda-clay steel interface and Leda clay alone.

3.3.3. Effect of Leda clay's OCR on the interface shear behavior

The results obtained have shown that the OCR has a significant effect on the interface shear behavior of Leda clay. Typical result samples are presented in Figures 3-7 to 3-10.

Figure 3-7 shows representative examples of the effect of OCR on the shear stress-relative shear displacement curves of the interface shear behavior under normal

stresses of 250 kPa. An analysis of Figure 3-7 reveals that the peak interface shear stress is a function of the OCR. A higher OCR leads to a higher peak shear stress. This is attributed to the fact that an increase in an OCR ratio results in more densification of the porous medium of Leda clay and stronger clay asperities with higher interlocking forces that resist shearing deformation. However, it was found that the magnitude of the OCR ratio effect on the interface shear strength depends on the applied normal stress. A higher normal stress implies a lower magnitude of the OCR ratio effect. This is illustrated by Figure 3-8 which shows the shear stress-relative shear displacement curves of the interfaces of Leda with OCR = 3 and OCR = 1 for two normal stress levels, 250 and 450 kPa. From this figure, it can be noted that in a comparison of OCR 1 and 3 curves, the peak shear stress of the sample with OCR = 3 is around 35% higher than that of the sample with OCR = 1 at 250 kPa normal stress. However, for a normal stress of 450 kPa, the shear strength of the OCR 3 sample is only 7% higher. This implies that the benefits of OCR decrease as the magnitude of the normal stress increases. Similar observations were made by DeJonga and Westgateb (2005). The observed reduction of the effect of OCR as the normal stress increases could be explained by the damage of the original clay structure and remolding of clay particles under 450 kPa normal stress.

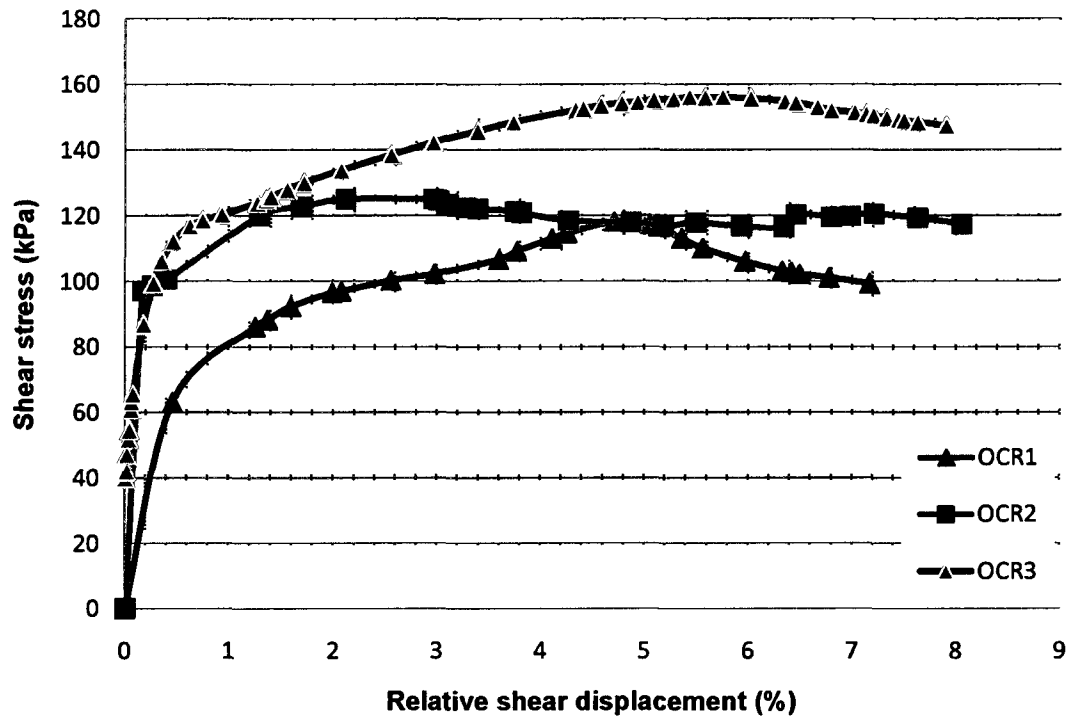


Figure 3-7: Shear stress vs. horizontal shear displacement of the Leda clay interface for OCRs 1, 2 and 3 at a normal stress of 250 kPa ($R_{max} = 5 \mu\text{m}$).

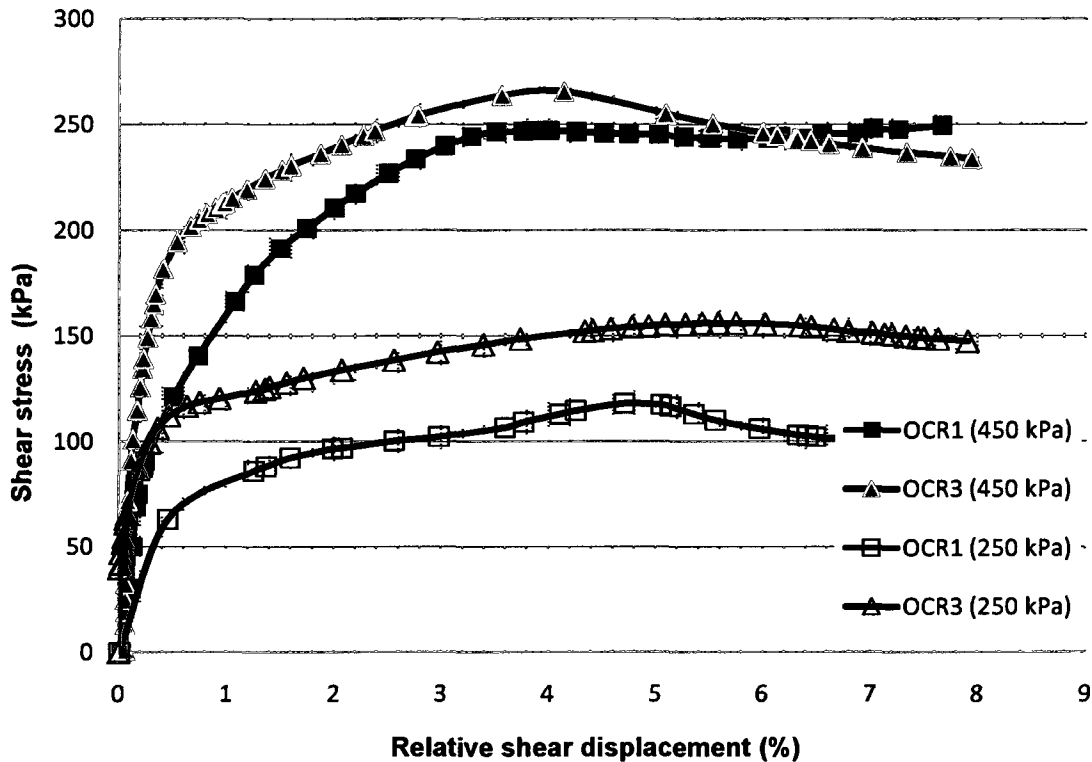


Figure 3-8: Shear stress vs. relative shear displacement of the interface of Leda clay with OCR = 1 and 3 at 250 and 450 kPa normal stress ($R_{max} = 5 \mu m$).

Figure 3-9 displays the shear displacement vs. vertical deformation behavior of the Leda clay interface at 250 kPa normal stress for the three OCRs. From this figure, it is clear that increasing the OCR of Leda clay leads to a reduction of the interface and soil compressibility. This behavior can be attributed to the fact that the compressibility of overconsolidated soils is much less than that for the same soil in normally consolidated condition (Venkatramaiah, 2006). Also, overconsolidation produces a dense clay structure and leave less space for contraction during the shearing process. Moreover, the OCR 3 curve shows a dilation behavior at the beginning of shearing. This can be attributed to the rise of dense clay asperities out of their embedment location within the steel asperities. As the shearing propagates, and after passing the peak stress zone, crushing the clay asperities causes an interface contraction with a level less that that observed at OCRs 1 and 2. DeJong and

Westgate (2005) observed that the volumetric behavior of Unimin sand was changed towards a dilative behavior as the OCR value increases

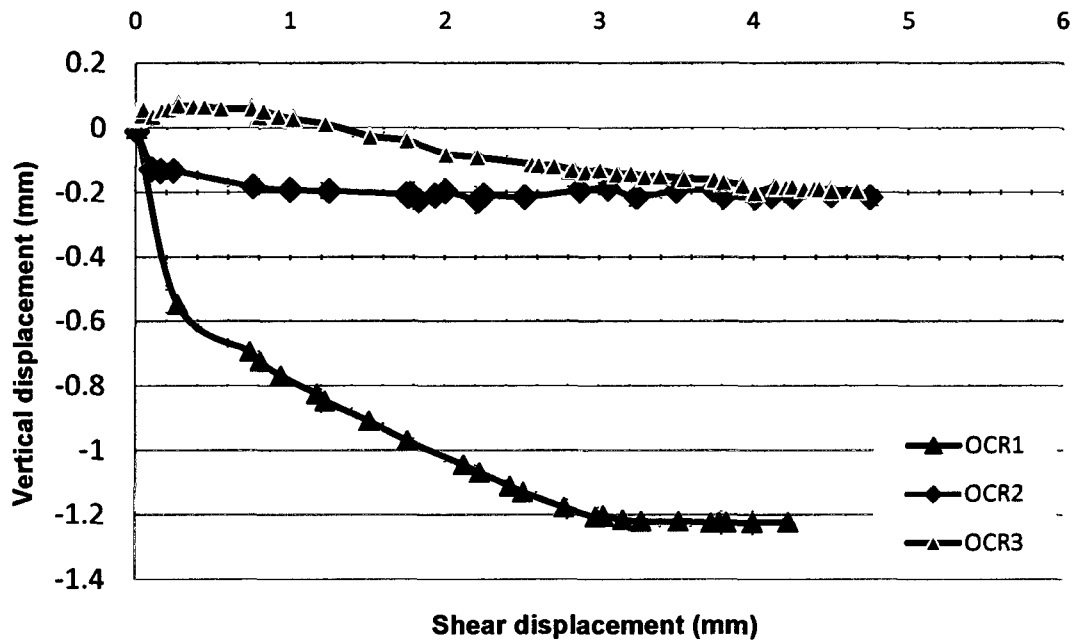


Figure 3-9: The effect of OCR on the interface vertical displacement at a normal stress of 250 kPa ($R_{max} = 5 \mu\text{m}$).

In Figure 3-10, the Mohr-Coulomb diagram indicates that the interface angle of friction increases as the OCR increases. Similar observations were recorded by DeJong and Westgate (2005) with granular soil interface tests. Their research results shown that the interface coefficient increases as the OCR value increases. This behavior can be attributed to the densification of soil asperities due to overconsolidation and the increase in interlocking forces that led to higher shear resistance at the steel interface. This suggests that, from an engineering practical point of view, the preloading of Leda clay soil would enhance its interface shear behavior.

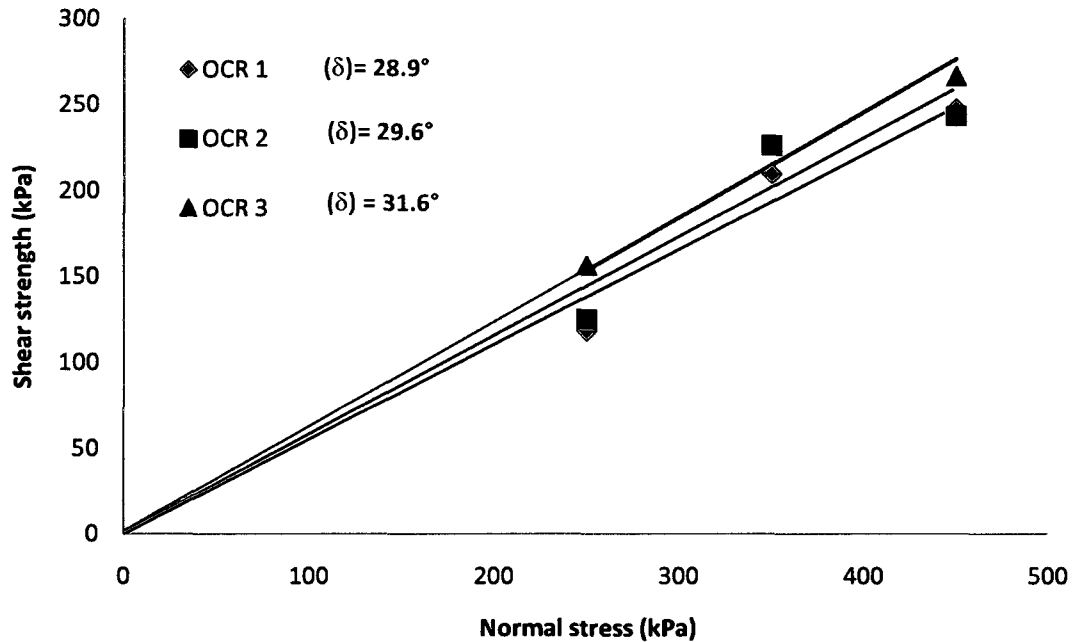


Figure 3-10: Interface shear envelopes of the Leda clay-steel interface for various OCRs ($R_{max} = 5 \mu\text{m}$).

3.3.4. Effect of initial saturation degree on the interface shear behavior

Unsaturated soils are frequently found in nature. It is well known that suction can significantly affect geotechnical behavior (e.g., shear strength, hydraulic conductivity) of unsaturated soil (e.g., Fredlund and Rahardjo 1993). Unsaturated interface conditions are commonly encountered in the design of several geotechnical structures, such as piles, retaining walls, and shallow foundations. Thus, the study of the effect of unsaturated conditions on the interface shear of Leda clay is important to improve our understanding of the behavior of structures that come in contact with unsaturated Leda clay, and ultimately to improve the engineering of these geotechnical systems. Hence, the effect of the initial degree of saturation or suction on the interface shear behavior of Leda clay has been studied.

Typical results of the effect of the initial degree of saturation on the shear stress-relative displacement curves of the Leda clay interfaces are shown in Figures 3-11 and 3-12 for normal stresses of 250 kPa and 450 kPa, respectively. Based on the

range of initial saturation degrees (60% to 96%) or suctions (1180 kPa to 500 kPa) tested and by keeping all other test parameters the same, it can be observed that the interface shear strength significantly increases as the initial saturation degree decreases or as the initial suction increases. The results are consistent with expected trends in behavior in that the shear strength increases with increasing net normal stress and matrix suction for the interface tests (Hamid and Miller, 2009). Similar observations have been made in other studies on unsaturated soil interface behavior (e.g., Fleming et al. 2006; Hamid and Miller 2009).

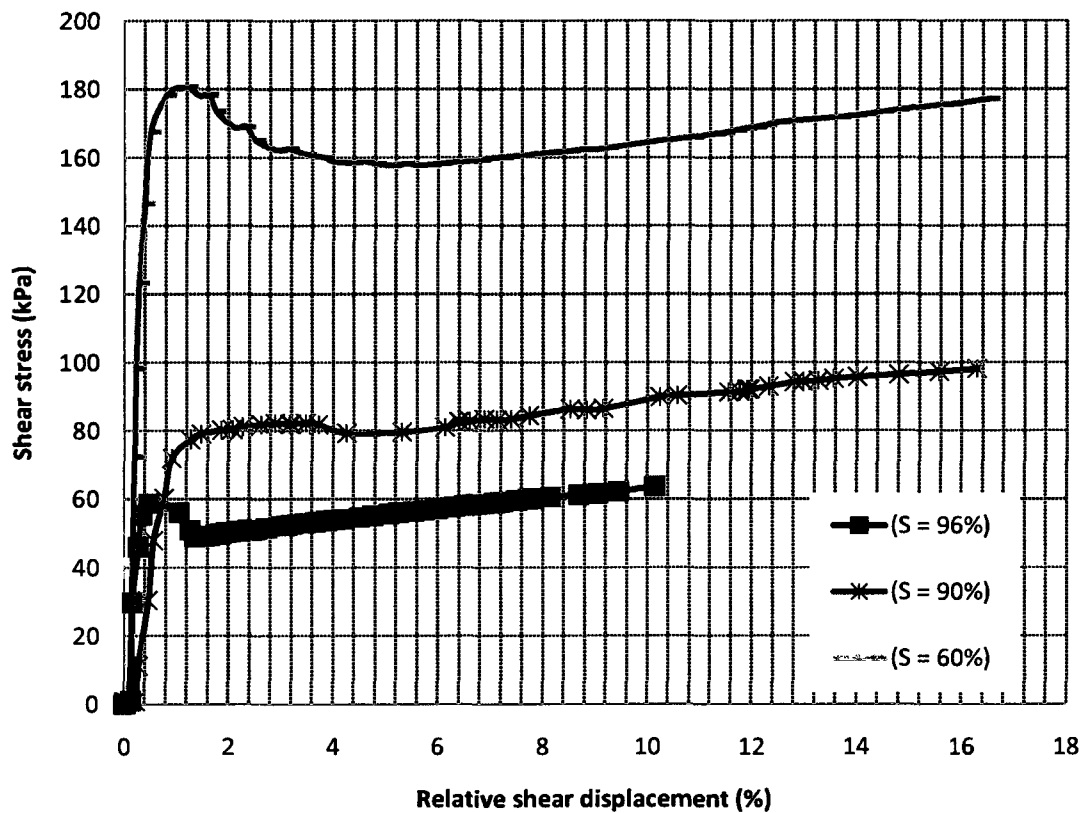


Figure 3-11: Typical shear stress vs. relative shear displacement of the Leda clay interface at normal stress of 250 KPa for 96%, 90% and 60% initial degree of saturation (S) ($R_{max} = 5 \mu m$).

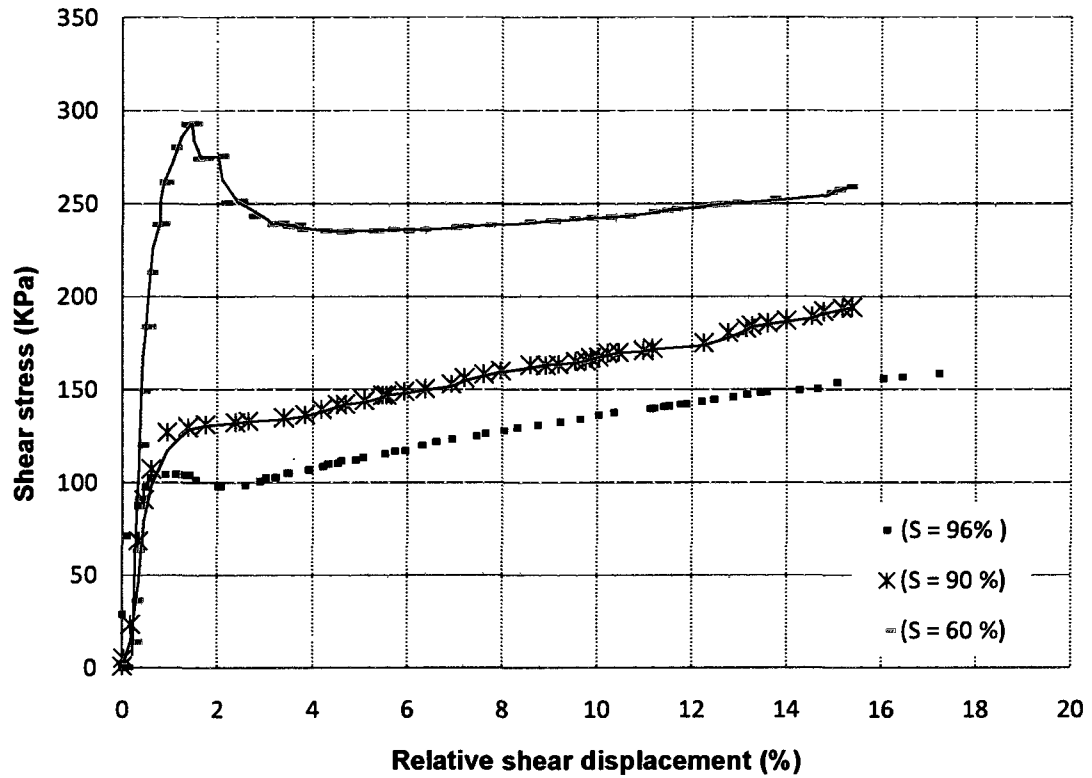


Figure 3-12: Shear stress vs. relative shear displacement at normal stress of 450 KPa for 96%, 90 % and 60 % initial degree of saturation (S) ($R_{max} = 5 \mu m$).

Typical results of vertical displacement versus shear displacement of the interface between Leda clay and steel are shown in Figure 3-13. It can be observed that for the studied shear displacement and normal stress ranges, the Leda clay interface exhibits contraction behavior. Also, Figure 3-13 indicates that the sample with 60% saturation underwent less interface contraction than that with 90% saturation. The reason for this behavior can be explained by the fact that the amount of compression decreases as the suction significantly increases (Fredlund and Rahardjo 1993). Moreover, at high suction and low saturation degree and high suction, the clay fabric changes into more rigid structure that can resist further volume changes during shearing. Similarly, Hamid and Miller (2009) observed that the interface volume behavior Oklahoma Minco silt changes towards reduction in contraction and dilation as the matric suction increases. However, Figure 3-13 shows that the sample with 96 % saturation underwent less interface contraction than that

with 90% saturation. The reason for this smaller difference can be related to the changes in saturation during shearing and the fact that the two samples were prepared at saturation degrees relatively close to each other. Also, it can be seen that the 60% degree of saturation sample has low contraction and almost no contraction occurs after the deformation that corresponds to the peak stress. The reason for this behavior can be related to the fact that a low initial saturation degree or high initial suction leads to the formation of a rigid clay structure which has undergone less contraction.

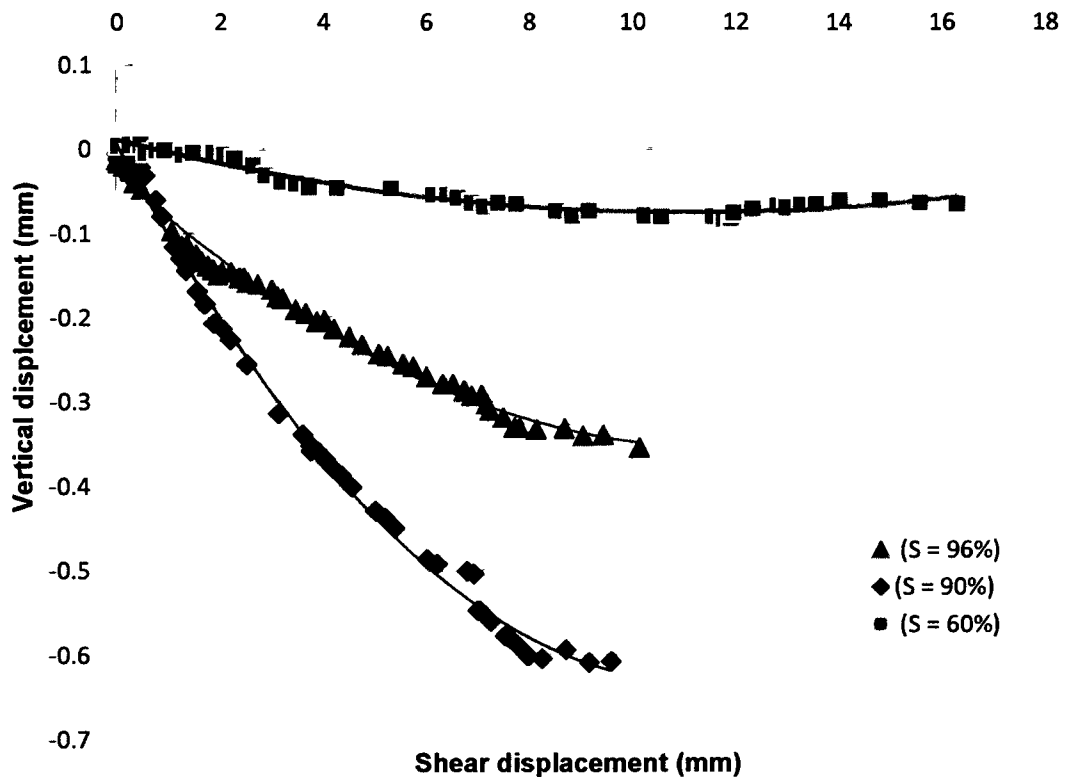


Figure 3-13a: Vertical displacement vs. relative shear displacement comparison at 250 KPa for 96%, 90 % and 60% initial saturation degree (S) ($R_{max} = 5 \mu m$).

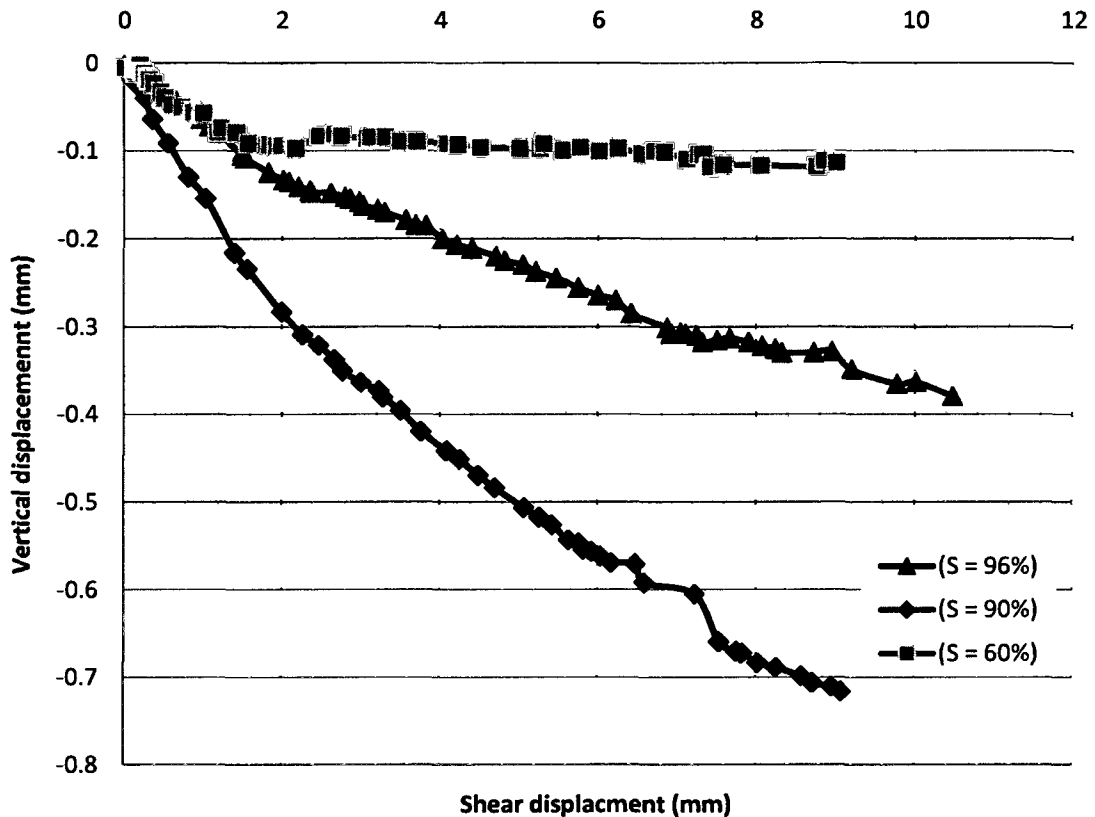


Figure 3-13b: Vertical displacement vs. relative shear displacement comparison at 450 KPa for 96%, 90 % and 60% initial saturation degree (S) ($R_{max} = 5 \mu m$).

3.3.5. Effect of Leda clay's dry density on the interface shear behavior

Figures 3-14 and 3-15 illustrate representative examples of the effect of compaction or initial dry densities of the Leda clay on the shear vs. relative shear displacement and vertical displacements vs. shear displacement curves of the interfaces. Over the range of densities tested, the interface shear strength is found to increase as the dry density of the compacted Leda clay increases (Figure 3-14). It can be noted that under a normal stress of 250 kPa, the peak shear stress of the sample prepared at a dry density of 1530 kg/m^3 is around 13% higher than that of the sample prepared at a dry density of 1500 kg/m^3 . On the other hand, under the same normal stress, the peak shear stress of the sample prepared at 1530 kg/m^3 is around 14% higher than that of the sample prepared at 1400 kg/m^3 . This behavior can be attributed

to the densification of the clay's asperities that lead to higher clay-steel interlocking forces. Furthermore, at higher dry density values, more clay particles are in contact with the surface of the steel, resulting in increased contact area, and therefore, increased interface shear resistance. These results are in agreement with those of DeJong and Wetstgate (2009) on cohesionless soils. The authors found that dense specimens have higher interface shear resistance than loose specimens. These results suggest that the compaction of Leda clay would result in an enhancement of the shear strength at the interface between Leda clay and steel structure. From Figure 3-15, it can be observed that in a pre-peak shear deformation range, the interface with clay compacted at optimum (dry density = 1530 kg/m^3) undergoes dilatation, while the interfaces with clay compacted on the dry side show contraction behavior. This dilative response can be related to the stronger asperities of the clay compacted at dry optimum. The sliding of clay asperities above the steel asperities leads to expansion at the interface. However, in the post-peak deformation region, all interfaces undergo compression behavior. This can be attributed to the destruction of clay asperities, the remolding and collapse of the clay structure that result in contraction behavior.

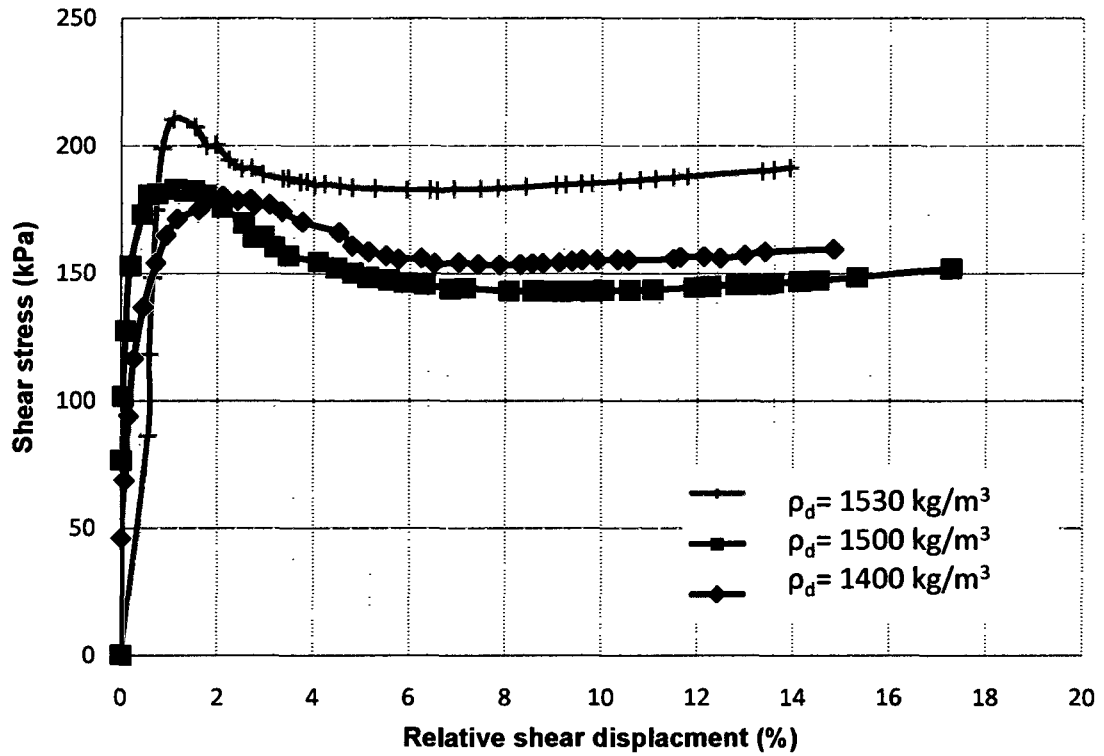


Figure 3-14a: Shear stress vs. relative shear displacement at 250 kPa for different dry densities of 1530, 1500 , 1400 kg/m^3 ($R_{\text{max}} = 5 \mu\text{m}$).

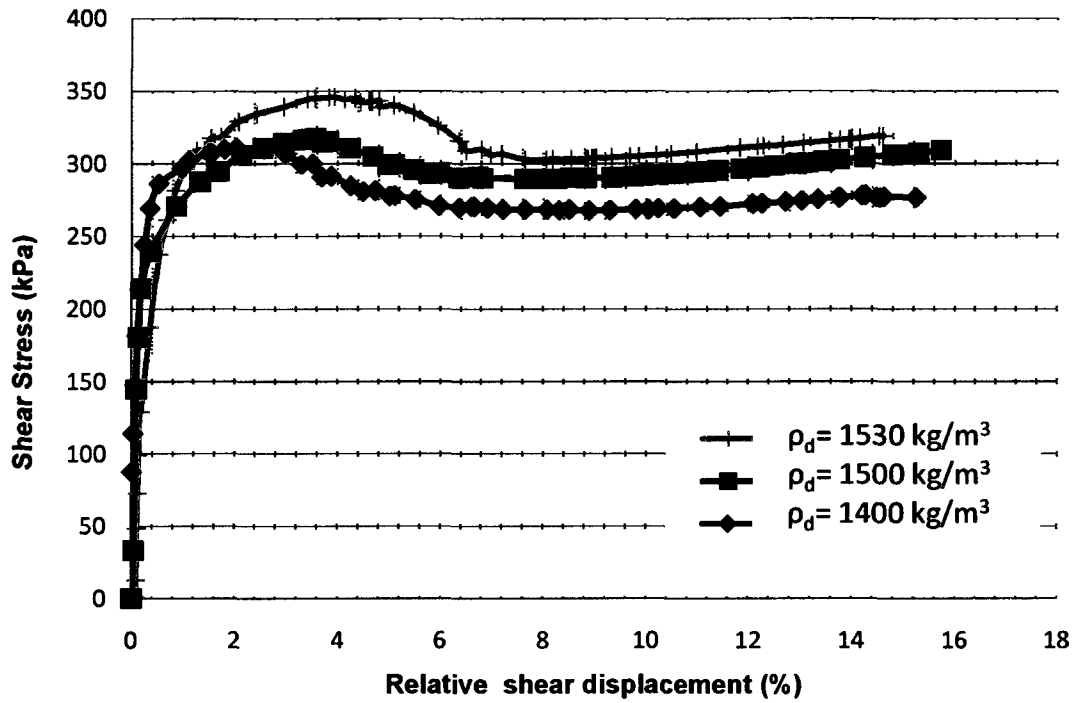


Figure 3-14b: Shear stress vs. relative shear displacement at 450 kPa for different dry densities of 1530, 1500 , 1400 kg/m³ ($R_{max} = 5 \mu\text{m}$).

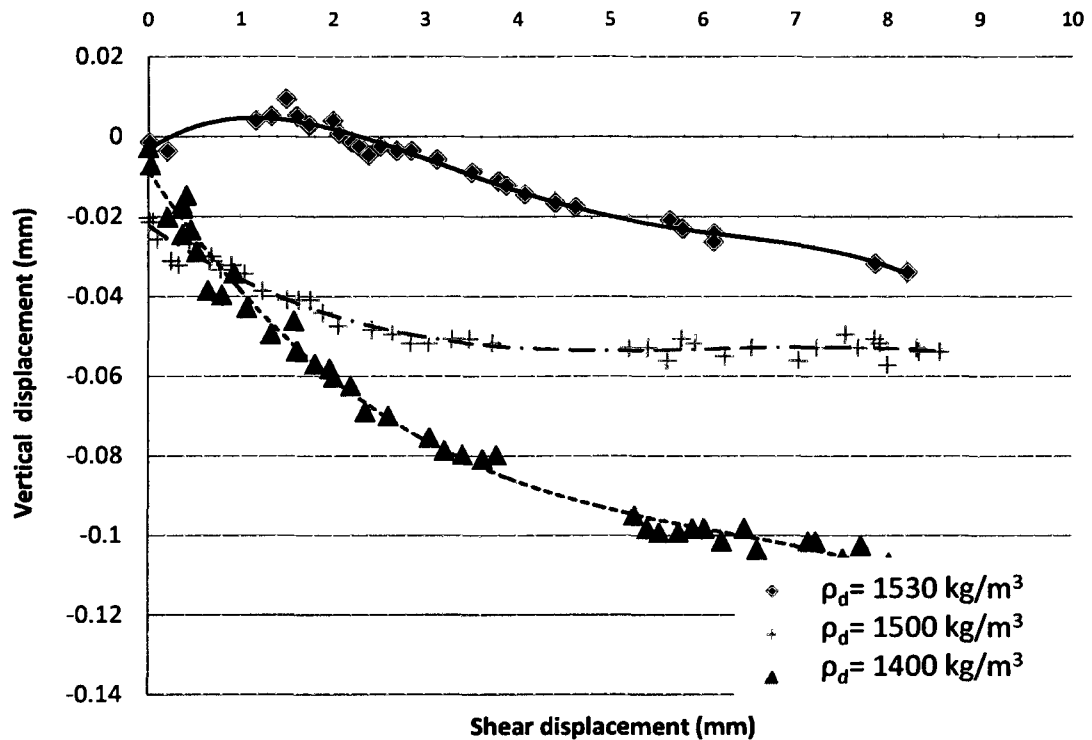


Figure 3-15: Vertical displacement vs. relative shear displacement at 250 KPa for different dry densities of 1530, 1500 , 1400 kg/m³ ($R_{max} = 5 \mu\text{m}$).

3.3.6. Effect of Leda clay's salt content on the interface shear behavior

It is widely recognized that geotechnical properties (sensitivity, shear strength) of marine clays can be significantly affected by the nature and concentration of salt within the clay. While there is a strong relationship between the salt content and the sensitivity of Scandinavian marine clays, this relationship is not entirely clear or has not always existed for all Leda clays. Penner (1965) noted that the relationship between salinity and sensitivity found in Scandinavian sensitive marine clays do not apply for Canadian marine clays. However, the Canadian marine clays located in Chelsea (in Quebec, near Ottawa) have been found to show response to salinity changes similar to that of the Norwegian marine clays (Moum et al. 1968, 1971, 1972). Hence, the effect of salt content on the interface behavior of Leda clay has been studied. Typical results of the effect of salt content on the shear stress-relative displacement and shear displacement-vertical displacement curves of the Leda clay interfaces are illustrated by Figures 3-16 and 3-17, respectively.

Figure 3-16 reveals that the shear resistance at the interface significantly increases as the salt concentration increased from 3g/l to 10 g/l. A salt content higher than 10 g/l has no significant influence on the shear strength. This behavior can be explained by the outcome of the research carried out by Torrance (1975) who studied the effects of added salt on the remolded shear strength and the liquid sensitivity limits of two Ottawa area clays. Torrance (1975) found that the sensitivity of these Ottawa clays decreases as the salinity increases, while the undrained shear strength increases as the salinity increases. This argument is also supported by the results of laboratory vane shear tests carried out on Leda clay samples with various amounts of initial salt content (3g/l, 10g/l) in this study. The results showed that the initial sensitivity (6, see table 1) of Leda clay samples is reduced by around 50% as salinity was increased from 3 g/l to 10 g/l.

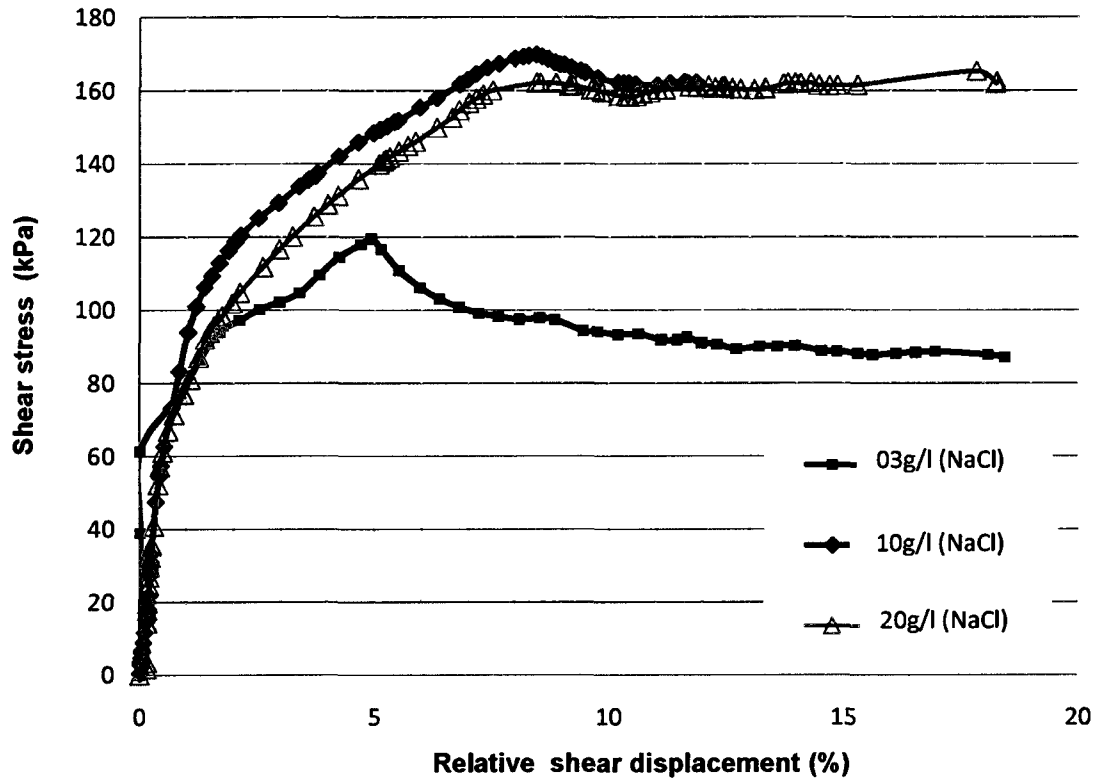


Figure 3-16a: Shear stress vs. relative shear displacement at 250 kPa for NaCl concentrations of 3 g/l , 10 g/l and 20 g/l ($R_{max}=5\mu m$).

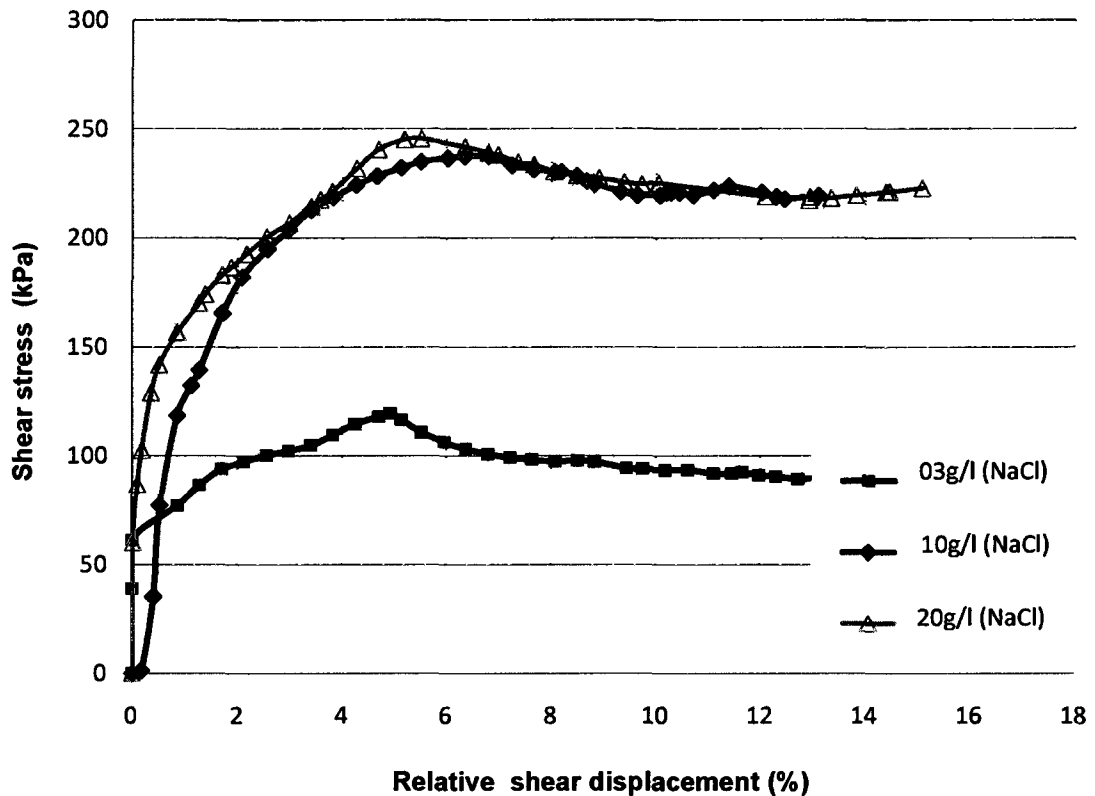


Figure 3-16b: Shear stress vs. relative shear displacement at 450 KPa for NaCl concentrations of 3g/l , 10 g/l and 20 g/l ($R_{max} = 5\mu m$).

Figure 3-17 shows that samples with high salt content (10 g/l, 20 g/l) demonstrate less contraction than samples with low salt content (3 g/l). This lower contraction can be attributed to the salt-induced enhancement of the bonds between the clay particles, thereby resulting in higher resistance to volume deformation at the interface. This explanation is also supported by Torrance (1975) research who noticed that high concentration of ions enhances the interaction between clay particles and increases the bonds between those particles.

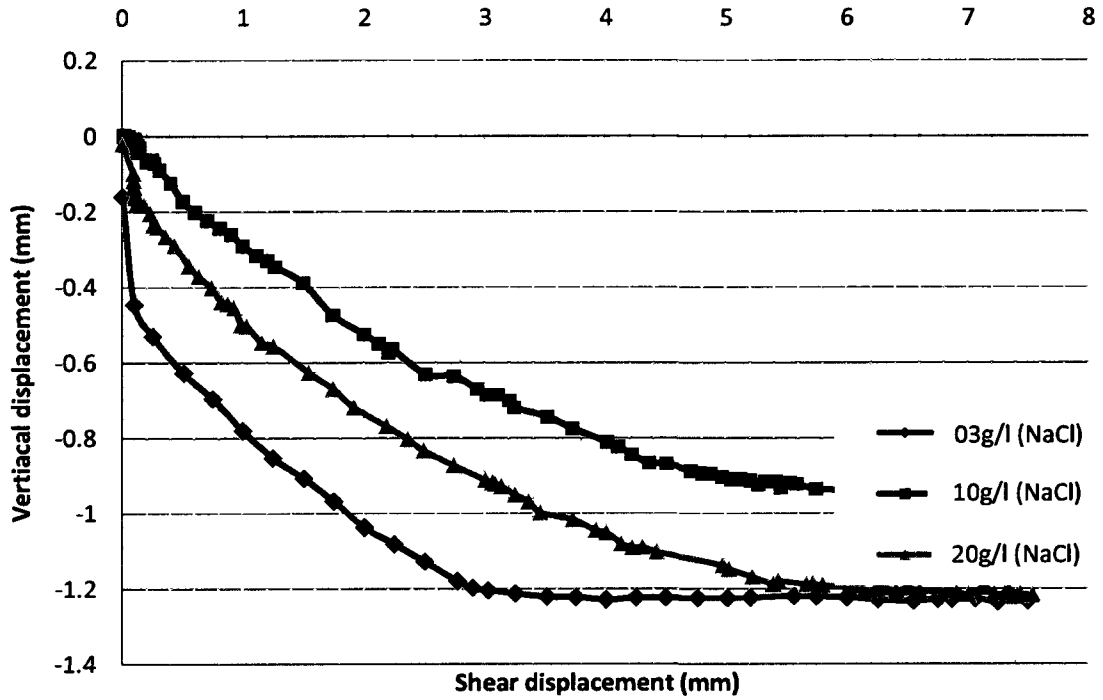


Figure 3-17: Vertical displacement vs. relative shear displacement at 250 KPa for NaCl concentrations of 3g/l, 10g/l and 20 g/l ($R_{max} = 5\mu m$).

3.4. Conclusion

This paper has addressed the understanding of Leda clay-steel interface shear behavior and how several factors, such as steel surface roughness, the degree of saturation in clay, OCR, dry density and salt content, influence the shear behavior of the interface. The following points summarize the paper's main findings and conclusions:

- increasing the Leda clay-steel interface surface roughness increases the interface friction angle. Leda clay-steel interfaces with a steel surface roughness of 20 μm show a similar frictional angle as those of Leda clay alone. However, increasing the surface roughness will result in more interface contraction. Thus, the use of steel with rough surfaces will enhance the interface shear strength of the geotechnical structure that is in contact with Leda clay;
- the Leda interface shear strength increases as the OCR ratio increases;
- a lower saturation degree in Leda clay is associated with higher interface shear resistance and lower contraction during shearing;
- increasing the dry density of Leda clay on the dry side of optimum increases the interface shear strength and reduces the interface contraction; and
- increasing the pore water salinity of Leda clay enhances the friction resistance and reduces contraction at the interface. This suggests that chemical treatment could be a suitable means to improve the interface shear strength of Leda clay.

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Chapter 4

Technical Paper II-

Shear Behaviour of Sensitive Marine Clay- Concrete interfaces

A. Taha , M. Fall

ABSTRACT

Understanding the interface shear behavior between sensitive marine clays and concrete structures is significantly important for safe and cost effective designs of several geotechnical structures (e.g., friction piles, retaining walls, and anchors). This paper presents the results of an experimental study on Leda clay-concrete interface shear behavior. The interface tests are carried out by using a direct shear apparatus at normal loads which range from 250 to 450 kPa. Three types of concrete surfaces are prepared to investigate the effects of surface roughness on the interface shear behavior. Leda clay specimens with different degrees of initial saturation, dry densities, and salt content are used to study the influence of these parameters on the interface shear behavior. The results indicate that: (i) shearing within clay dominates the interface shear behavior except at very low degrees of saturation; (ii) interface shear resistance increases as the concrete surface roughness increases; (iii) interface shear resistance increases as the saturation degree of Leda clay decreases; (iv) increasing the dry density of remolded Leda clay enhances its interface shear resistance; and (v) a higher salinity of Leda clay pore water results in higher interface shear strength.

4.1. Introduction

Sensitive marine clays occupy large areas of geological stratigraphies around the world. These clays are widely present in Canada, Scandinavia, and other countries and regions at high latitudes. Canada's sensitive clays are the products of the crustal rebound of postglacial lake sediments which existed 18,000 and 6,000 years BP (Quigley, 1980). These sediments consist of both freshwater and marine clays. The micro-structure of these types of clays is similar to a card house structure that collapses and flows upon remolding and excessive wetting (Quigley and Thompson, 1966).

The existence of Leda clay underneath massive lands which cover the Canadian provinces of Ontario and Quebec makes it challenging for geotechnical engineers to arrive at safe and cost effective foundation designs, such as concrete piles and deep earth retaining walls. At locations where thick deposits of Leda clay exist, many engineers used to adopt pile designs to support heavy superstructures. The limited understanding of Leda clay's interface interaction with concrete foundation structures and the lack of true interface design parameters have led to overly conservative pile foundation designs in Leda clay. Furthermore, the understanding of Leda clay interface shear behavior is also essential for the cost-effective design of several other geotechnical structures, such as concrete anchors, soil reinforcements, and retaining structures.

Despite the large number of research carried out on the interface shear behavior between cohesionless soils and concrete e.g., (Zong-Ze et al. 1995; Kulhawy and Peterson, 1979) only a few researchers have focused on the cohesive soils-concrete interface problem. For example, Shakir and Jungo (2009) conducted experimental tests on the interface shear behavior between clay and concrete with different surface roughness. They concluded that shear failure occurs within the body of the clay samples for rough concrete surfaces when the clay water content is greater than 16%. Otherwise, interface shear sliding dominates interface shear displacement behavior. They also found that as the water content ratio increases, the interface shear strength

also increases. Moreover, Hammoud and Boumekik (2006) focused their research on the effect of changing the surface roughness of concrete and steel on the interface shear behavior of cohesive soils. Their results indicated that the surface roughness and the average diameter of particles have a significant effect on the interfacial shear strength at a given normal stress level. They also found limiting values of relative roughness that depend on the soil characteristics, which influence the modes of failure. When the material surface is smoother than a lower value, a sliding shearing mode will occur at the interface. On the other hand, when the relative roughness surpasses an upper value, a shearing mode within the soil will instead take place.

In a review of previous interface research, the interface shear properties of Leda clay-concrete have never been previously studied to the best of the author's knowledge. Furthermore, since Leda clay is different from conventional clays or cohesive soils, the results obtained from previous studies on cohesive soil-concrete interface behavior cannot be directly applied to a Leda clay interface. Hence, a research program has been carried out at the University of Ottawa to investigate the shear behavior of Leda clay-concrete interface. The objective of this research is to study: (i) the shear behavior of the interface between Leda clay and concrete of different surface roughness; (ii) the influence of the salinity of Leda clay pore water on its interface shear behavior; (iii) the interface shear behavior of Leda clay with different initial saturation degrees; and (iv) the influence of the dry density of Leda clay on its interface shear behavior.

4.2. Experimental Program

4.2.1. Testing apparatus

Several interface research projects have been carried out by using the direct shear box (e.g., Yoshimi and Kishida 1993a,b; Zimnik et al. 2000; Dejong and Westgate 2005; Tan et al. 2008; Hamid and Miller 2009). Despite some of the disadvantages of using a direct shear box in soil testing, such as stress concentration, a predefined failure plane, and principle stress rotation, the direct shear apparatus has been widely

used in interface testing. This is mainly because it can easily simulate the loading and shearing mechanisms established in real interfaces. Moreover, it was proven by several researchers that for plastic soils, the uniformity of stress and strain in the apparatus are acceptable up to the peak shear stress (DeGroot et. al, 1994). In this research, the interface shear tests are carried out by using a standard digital direct/residual shear apparatus. This apparatus uses a microprocessor controlled drive system and keyboard entry that provides a wide range of features, including pause and speed changes from 0.00001 to 9.99999 mm/minute. The vertical and horizontal deformations are recorded through an LVDT that is vertically and horizontally mounted to the shear box. The resistance to shearing is recorded by using a loading cell that is horizontally mounted to the shear box. The recorded data are gathered by a data logging system and analyzed by a laboratory data analysis software Labview.

4.2.2. Leda clay samples

Undisturbed Leda clay samples were collected from the Tenth Line Road in Ottawa, from a depth that ranged from around 8 to 12 meters. Several laboratory geotechnical tests were conducted in accordance with ASTM standards to determine the basic geotechnical properties of the sampled Leda clay. The basic properties of the Leda clay samples are presented in Table 4-1.

The Atterberg limits of Ottawa's Leda clay matches with Leroueil's (1999) findings which stated that the LL of eastern Canada clays is less than 83% and the plastic limit is between 17% and 35%. The open fabric structure of Leda clay causes the water content of all samples to exceed their LL values. This is due to that fact that the clays with open fabric structures can hold a high water content without flowing and undergo sudden collapses upon remolding.

Table 4-1: Summary of the main characteristics of the Leda clay used

Property	Value/Classification
Classification	CH
Water content, %	82
Liquid limit, %	66
Plastic limit, %	25
Plasticity index, %	40
Clay fraction, %	84
Activity	0.48
Laboratory vane shear strength (kPa)	9.7
Sensitivity	6
Preconsolidation pressure (kPa)	150
Natural void ratio	2
OCR	1.1
Sodium chloride concentration	3-4 g/l
Optimum water content, Dry density at optimum	22%, 1.53 g/cm ³

It was noticed that in the particle size analysis, there is a high clay fraction of 84% (Figure 4-1). The Leda clay activity, calculated as 0.47, falls within the eastern Canadian activity range that was defined by Leroueil (1999), between 0.25 and 0.7. In addition, the soil-water characteristic curve (determined by using the WP4 Dewpoint potentiometer) indicates an air entrained value (a_v) of 400 kPa (Figure 4-2).

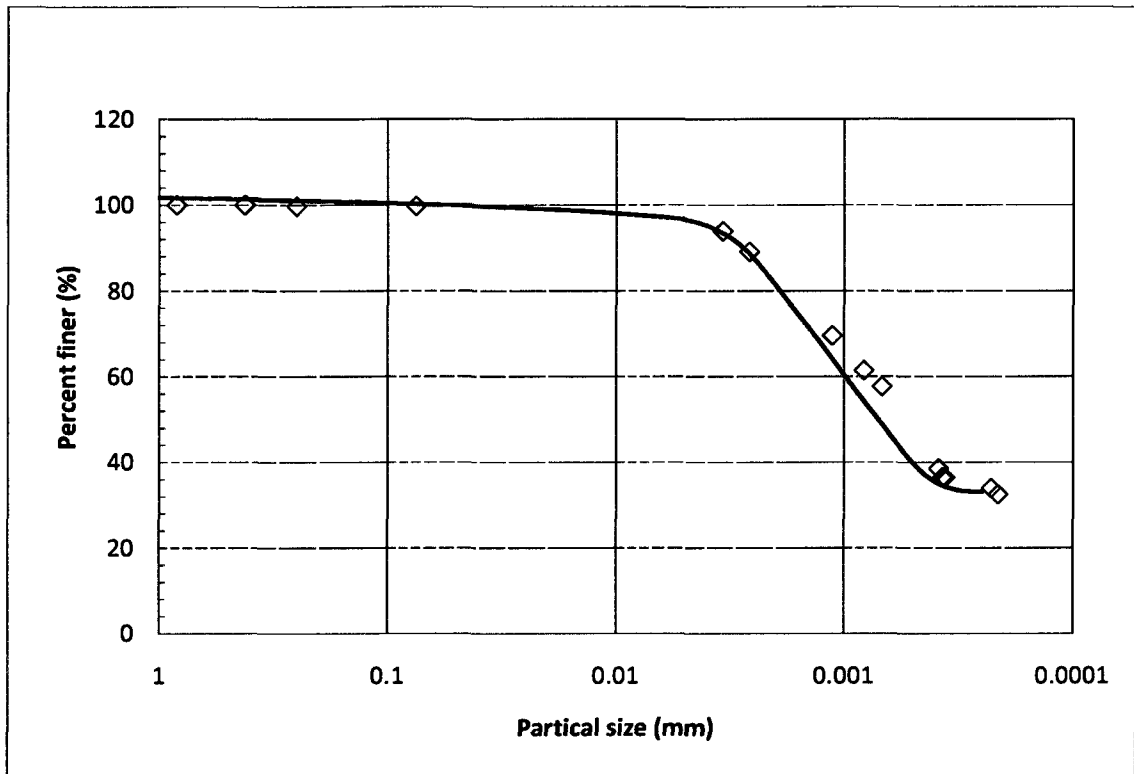


Figure 4-1: Grain size distribution of the Leda clay used.

According to the USCS, the Leda clay that is used in this study can be classified as inorganic CH. This result agrees with that of Locat et al. (1984) who investigated sensitive clays in Ontario, Quebec, and British Columbia, and found that almost all of the samples are classified above the “A” line. A one-dimensional consolidation test indicated that the preconsolidation pressure is 150 kPa and the C_c is 2.44. The laboratory C_u indicated that the undisturbed shear strength is 9.7 kPa, remolded shear strength is 1.7 kPa, and calculated sensitivity is 6. This sensitivity value is close to those determined by Burn and Hamilton (1968) on Ottawa’s Leda clay at a depth of 12 m.

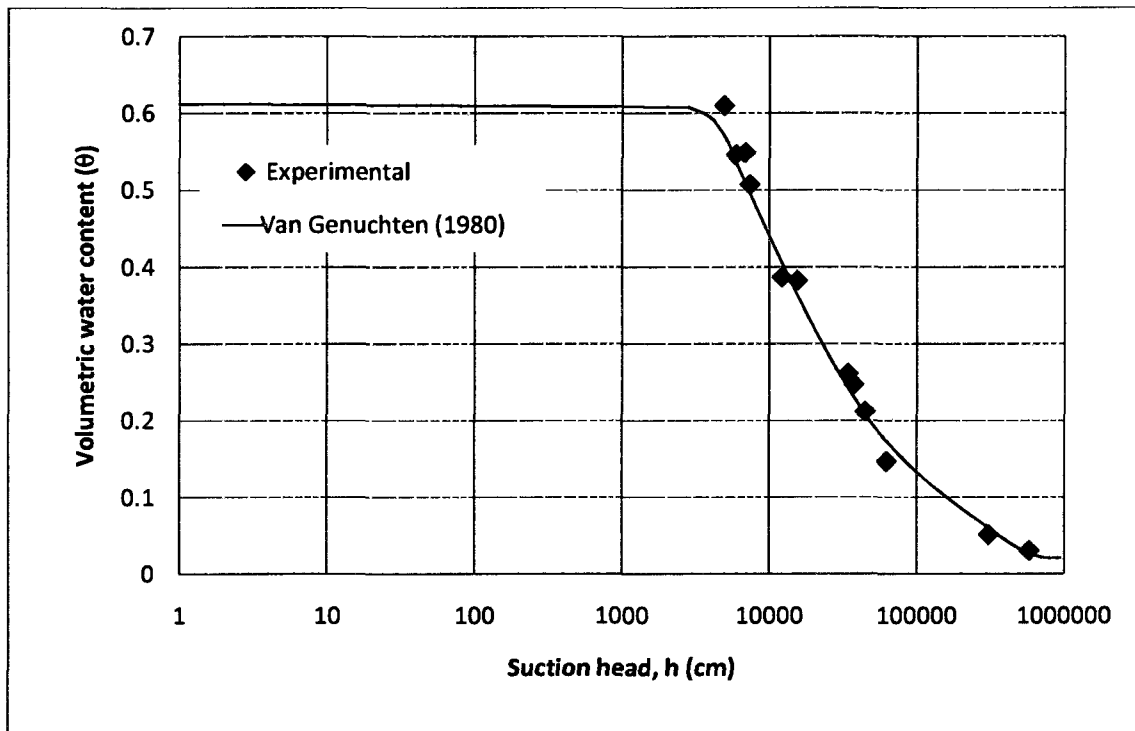


Figure 4-2: Soil water characteristic curve for the Leda clay used

4.2.3. Concrete samples

Around 60 concrete cubes were prepared by using a structural mix design of small aggregate sizes and then casted into 60 x 60 x 15 mm styrofoam molds. The cubes were cured in a standard humid room for at least 90 days. The compression strength of the cubes reached 18 MPa. To generate interfaces with different roughness, some cubes were grit blasted to generate a rough surface, others were poured against a smooth styrofoam surface to generate the smoothest surface possible, and the rest were disc grinded to produce a “medium” rough surface.

The interface roughness measurements were carried out by using concrete cubes with different profile roughness. The cubes were fixed onto the bottom half of the shear box, and then the automated direct shear equipment was used to run an LVDT over the interface surface. The LVDT together with the data logging system recorded vertical deformations of the interface profile. The profile roughness (R_{max}) was

evaluated based on the maximum recorded height over a gage length of 0.2 mm according to the method proposed by Yoshimi and Kishada (1981), and Uesugi and Kishada (1986). The three (R_{max}) values which were determined are 20 μm , 10 μm and 6.5 μm .

4.3. The shear tests

A series of consolidated drained (CD) and undrained (CU) direct shear tests (DSTs) were carried out to study the interface shear behavior of Leda clay and concrete. In addition, the influence and effects of the following factors on interface shear behavior is also examined:

- interface surface roughness,
- pore water salt content in Leda clay,
- degree of saturation of Leda clay, and
- dry density of Leda clay.

Furthermore, CD tests were also performed on Leda clay alone for comparison purposes.

4.3.1. Consolidated drained interface tests

A series of direct shear CD tests were carried out to examine the interface shear behavior of normally consolidated Leda clay sheared with respect to varying concrete surface roughness. Also, the CD tests were carried out to examine the effect of pore water salinity of Leda clay on its interface shear behavior.

At the beginning of the testing program, undisturbed Leda clay samples were obtained from Shelby tubes and cut from a vertical profile into squares of approximately 60 x 60 x 12 mm in size to fit into the top half of the shear box. Prior to placement in the shear box, the square samples were placed on top of porous stones. The porous stones were kept submerged in distilled water and covered with saturated filter paper in a humid room to reach complete saturation during a minimum time period of 24 hrs. Trial studies have shown that specimens attain full saturation during a period of 24 hrs. No significant volume changes of the samples were noted. On the

bottom half of the shear box, the concrete cubes were fitted into place by keeping their flat surface flushed with the top shearing surface. The concrete cubes with three different types of roughness were used to study the effect of roughness on interface shear behavior.

To examine the effect of the salinity of Leda clay on its interface shear behavior, undisturbed specimens were placed on top of a porous stone submerged in a NaCl solution for a minimum of 15 days. Three salt concentrations were used: 4g/l, 10 g/l and 20 g/l. An ICP-AES analysis was performed to determine the salt concentration of the Leda clay samples (Table 4-1).

This interface testing series is carried out per ASTM D 3080. This procedure is selected to simulate the effect of a slow loading rate after pore water pressure dissipation. The Leda clay samples were placed into the shear box which was submerged in water and left to completely consolidate at normal stresses high above their preconsolidation stress (Table 4-1). The normal stresses were maintained at 250, 350 and 450 kPa based on the previous experimental work of Dam et al. (1999) who examined the normal consolidation behavior of Canadian sensitive clays at normal stresses equal to or higher than 200 kPa. Moreover, these stresses cover a realistic range of operational stresses in geotechnical structures (DeJonga and Westgateb 2005). After the completion of the primary consolidation, the samples were sheared at a slow rate of 0.0185 mm/min. To minimize pore water pressure development during shearing, this shearing rate is selected based on the time required to reach 50% consolidation, i.e. the t_{50} , at a 5 mm displacement to failure. Moreover, in a review of the previous work of Tsubakihara et al (1993a), a CD shearing rate of 0.03 mm/min was used in their interface tests on Kawasaki marine clay, which indicates that our shearing rate of 0.0185 mm/min is a reasonable speed.

4.3.2. Consolidated undrained interface tests

A series of direct shear consolidated undrained (CU) tests were carried out to examine the interface shear behavior of disturbed and undisturbed Leda clays with different initial saturation degrees. ASTM D 3080 is applied in this series of Leda

clay-concrete interface tests. Prior to shearing, the soil specimens were left to consolidate under 250, 350 and 450 kPa normal stresses in a conventional direct shear apparatus. After the completion of the primary consolidation, the samples were sheared at a fast rate of 1.25 mm/min. This shearing rate is selected to simulate the CU condition or rapid shearing (Vanapalli and Lane, 2002). It can be assumed that this shearing rate would induce undrained conditions due the low coefficient of the permeability of clayey soils and shearing in a relatively short period of time (i.e., 5 to 10 minutes).

The undisturbed samples, prepared for studying the effect of different saturation degrees, were left for various increments of air drying. Thus, samples with saturation degrees of 96%, 70% and 5% were prepared. These saturation degrees correspond to the initial suction values of 500 kPa, 1000 kPa and 12,000 kPa, respectively.

To study the effect of different dry densities, remolded Leda clay was left to completely air dry, crushed, mixed with measured volumes of water, and statically compacted in a mold of known volume. In this test series, three dry density points are selected: two on the dry side and one at the optimum. The prepared clay samples have the same dry densities and water content of the selected points by adding the required amount of dry soil and water per the basic phase relations (Figure 4-3).

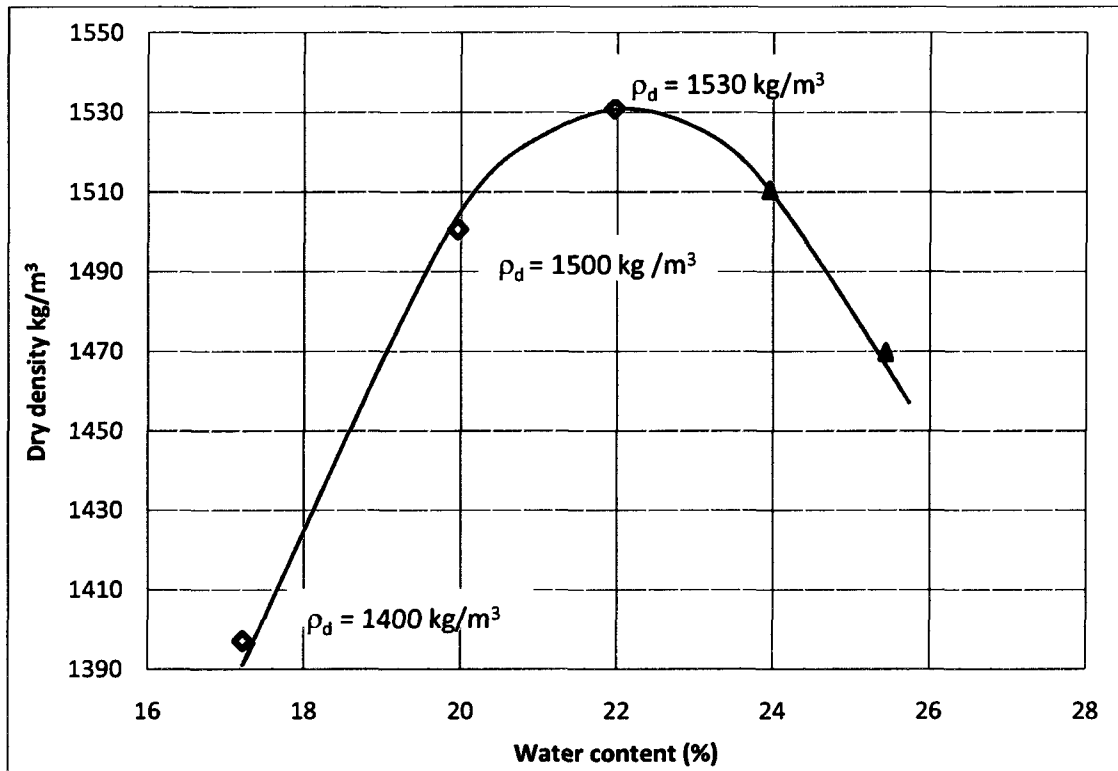


Figure 4-3: Leda clay compaction curve showing the three points selected for the interface tests

4.3.3. Direct shear tests on Leda clay

In addition to the interface shear tests, DSTs were performed on some Leda clay samples alone for comparison purposes. Undisturbed samples of Leda clay were placed on top of a porous stone submerged in distilled water for a minimum of 24 hrs to reach complete saturation. These samples have a thickness of 30 mm and dimensions equal to the inner width and length of the shear box. Then, after saturation was completed, the samples were fixed inside the shear box so that each sample could be sheared at the mid height between the two halves of the box. After completing primary consolidation under the desired normal loads (250, 350 and 450 kPa), the samples were sheared at a slow rate of 0.0185 mm/min. ASTM D 3080 is applied in this Leda clay-Leda clay DST.

4.4. Results and discussion

In the following sections, typical interface shear tests results will be presented and discussed. Furthermore, typical direct shear results on Leda clay alone will be presented for comparison purposes.

4.4.1. Shear behaviour of Leda clay alone and clay- concrete for various interface surface roughness.

Figure 4-4 and 4-5 show typical interface shear behavior of Leda clay-concrete with different surface roughness, and the shear behavior of pure Leda clay specimens at 450 and 250 kPa normal stresses, respectively.

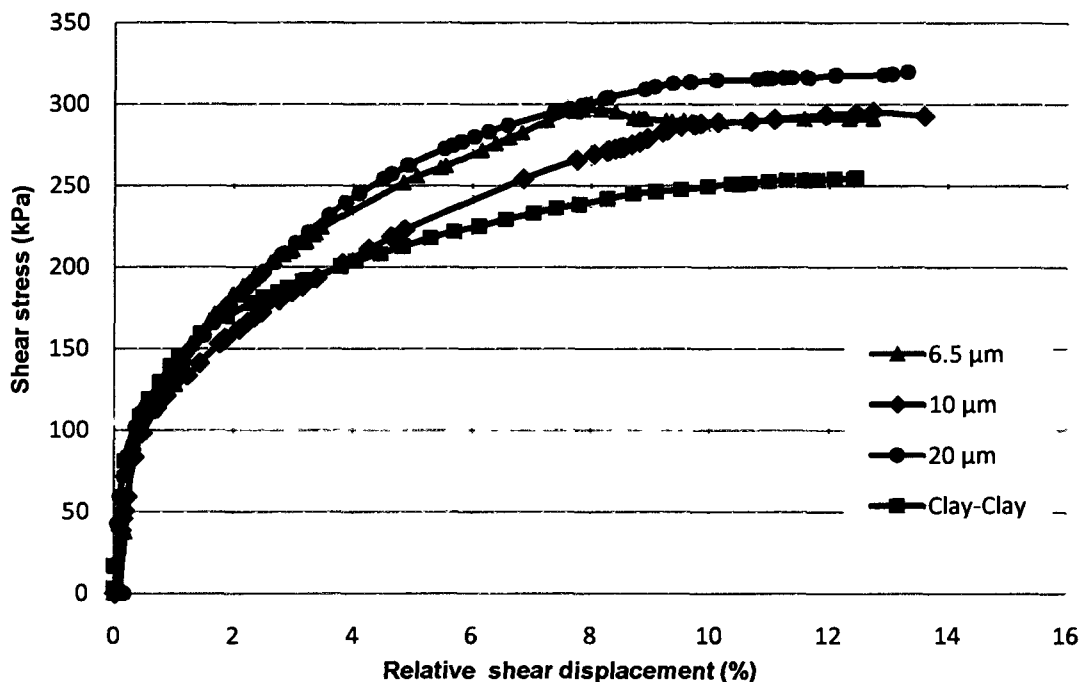


Figure 4-4: Shear stress vs. relative shear displacement for clay-clay and clay-concrete at 450 kPa normal stress.

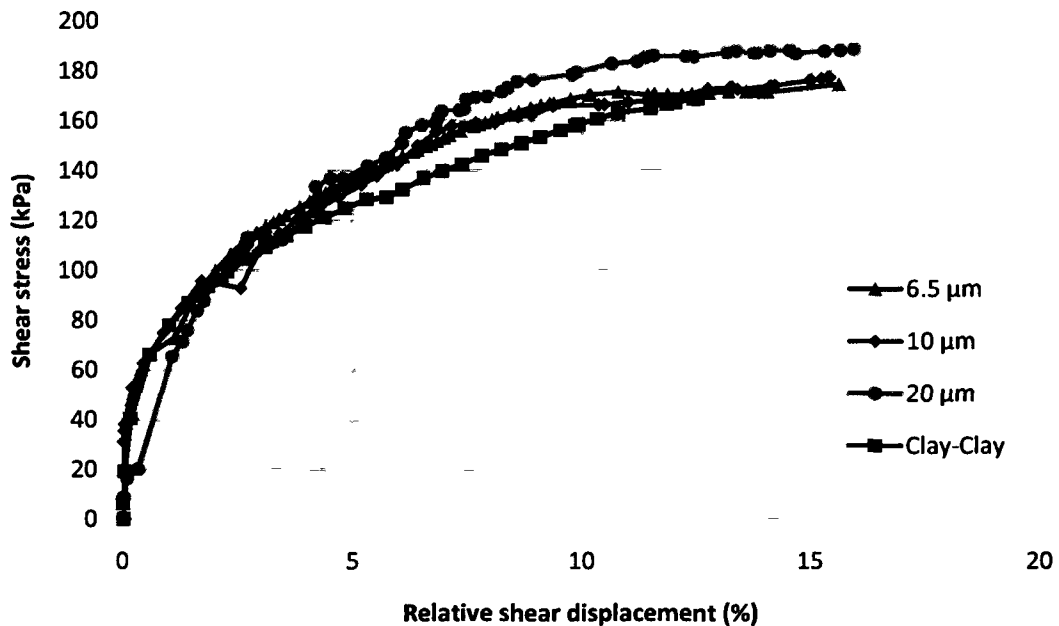


Figure 4-5: Shear stress vs. relative shear displacement for clay-clay and clay-concrete at 250 kPa normal stress.

It can be seen from Figure 4-4 that in the beginning of the shearing, the interface shear stress vs. relative displacement curves fit relatively well with that of the pure clay curve. This could suggest that the dominant shearing mechanism is shearing within the clay above the interface surface. However, after around 2% relative displacement, the interface stress increases as the surface roughness increases. Furthermore, Figure 4-4 indicates that the peak shear stress of the 20 μm -interface is around 6.4 % higher than that of the 10 μm -interface. In comparison, the peak shear stress of the 20 μm -interface is around 8.5 % higher than that of the sample sheared at the 6.5 μm -interfaces. This result indicates that the shear strength of the 10 μm -interface is close to that of the 6.5 μm -interfaces. This insignificant difference can be attributed to : (i) the accuracy of the measuring LVDT's ; (ii) the fact that the methodology of calculating the surface roughness value that has some limitations related to the lack in modeling the full topography of the interface; (iii) the possibility of having different R_{max} values for two surfaces sharing similar topographies. Also, this behavior may be attributed to the increase in the interlocking forces between clay

asperities as the concrete asperities increase and the remolding of the clay structure at the interface as shearing progresses.

Figure 4-5 shows similar behavior at 250 kPa normal stress. It can be noted that, in the beginning of shearing, the interface shear stress vs. displacement curves undergo similar shearing behavior compared to that of the clay-clay specimen. Also, the effect of the increase in the interface surface roughness on shear stress is noticeable after 4% relative displacement. However, a comparison between Figures 4-4 and 4-5 indicates that the difference between the interface curves and the clay-clay curve is less pronounced under the 250 kPa load. This can be attributed to two factors: i) the interface shearing occurs within clay and above the interface surface; and ii) shearing at a normal stress that is near the preconsolidation stress means that the original clay structure above the interface would be partially reserved. Therefore, at low normal stress, the concrete interface shearing mechanism becomes relatively similar to that of normally consolidated clay. Figures 4-4 and 4-5 also reveal that the interface shear strength increases as the concrete surface roughness increases significantly. This behavior can be attributed to the increase in the interlocking forces and contact area between the Leda clay particles and concrete asperities as the surface roughness increases. Furthermore, from Figure 4-5 it can be seen that a small increase in the interface roughness (from 6.5 μm to 10 μm) doesn't significantly affect the interface shear strength.

Moreover, from Figures 4-4 and 4-5, it can be observed that for the studied concrete surface roughness, the interface shear strength of the Leda clay interface is higher than that of the Leda clay alone. This can be attributed to the fact that shearing Leda clay against concrete asperities requires higher shearing stresses to overcome the interlocking forces at the clay-concrete interface which is comparable to the less harder clay-clay interface.

Figure 4-6 shows typical vertical displacement behavior with various surface roughness at 250 kPa normal stress. It can be observed that the contraction curves of the interface with the three surface roughness are close to the pure clay curve. This can be attributed to the same two factors mentioned before and that at low normal stress, the concrete interface vertical displacement mechanism becomes relatively similar to that of normally consolidated clay.

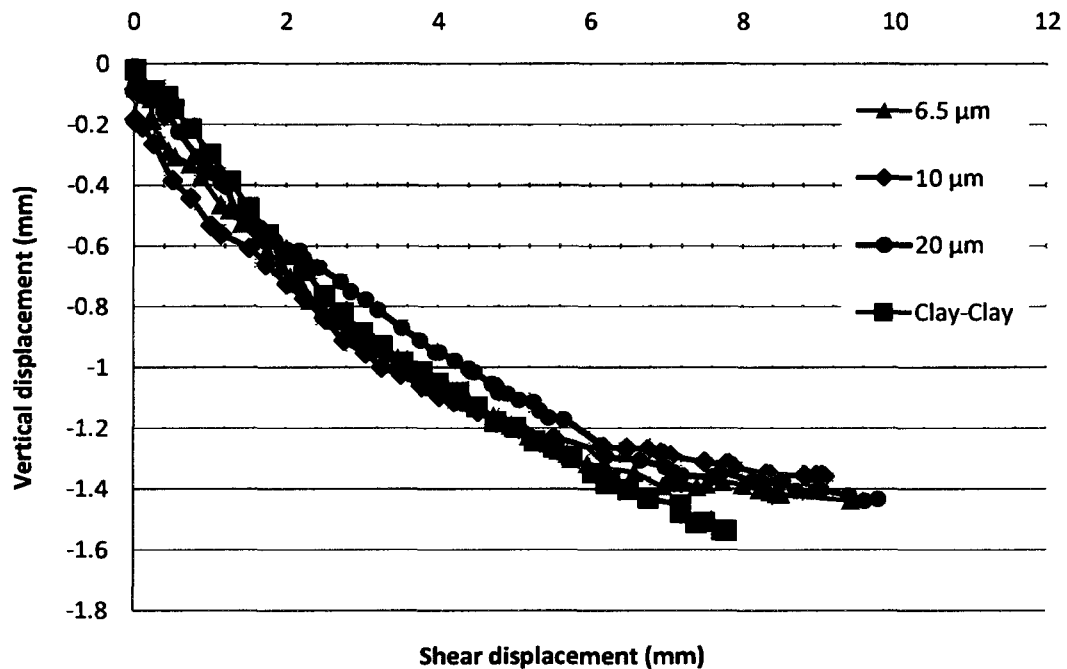


Figure 4-6: The effect of the surface roughness on the interface vertical displacement at 250 kPa.

Figure 4-7 compares the vertical deformation curves of the 10 μm Leda clay-concrete interface to those of Leda clay alone.

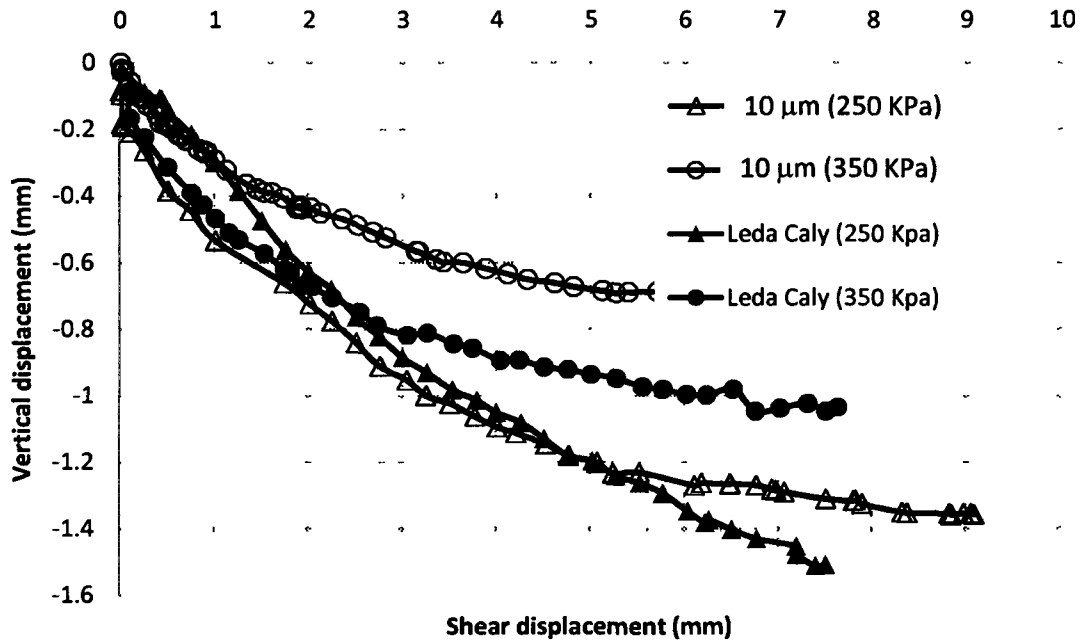


Figure 4-7: Comparison between the vertical displacement behavior of the normally consolidated Leda clay with the vertical displacement of Leda clay-concrete interface at 250 and 350 kPa.

It can be noted from Figure 4-7 that the concrete-clay interface undergoes less contraction than that of pure Leda clay at high increments of shear deformation. This can be due to the effect of remolding by concrete asperities that cause significant flake reorientation and more particle parallelism with further reduction in voids. Tsubakihara and Kishada (1993b) recorded the same observation from their steel interface tests of Kawasaki marine clay under a constant normal load.

Figure 4-8 shows a comparison between the normalized shear stress vs relative displacement curves at 250 and 450 kPa normal stresses with a 20 μm interface roughness.

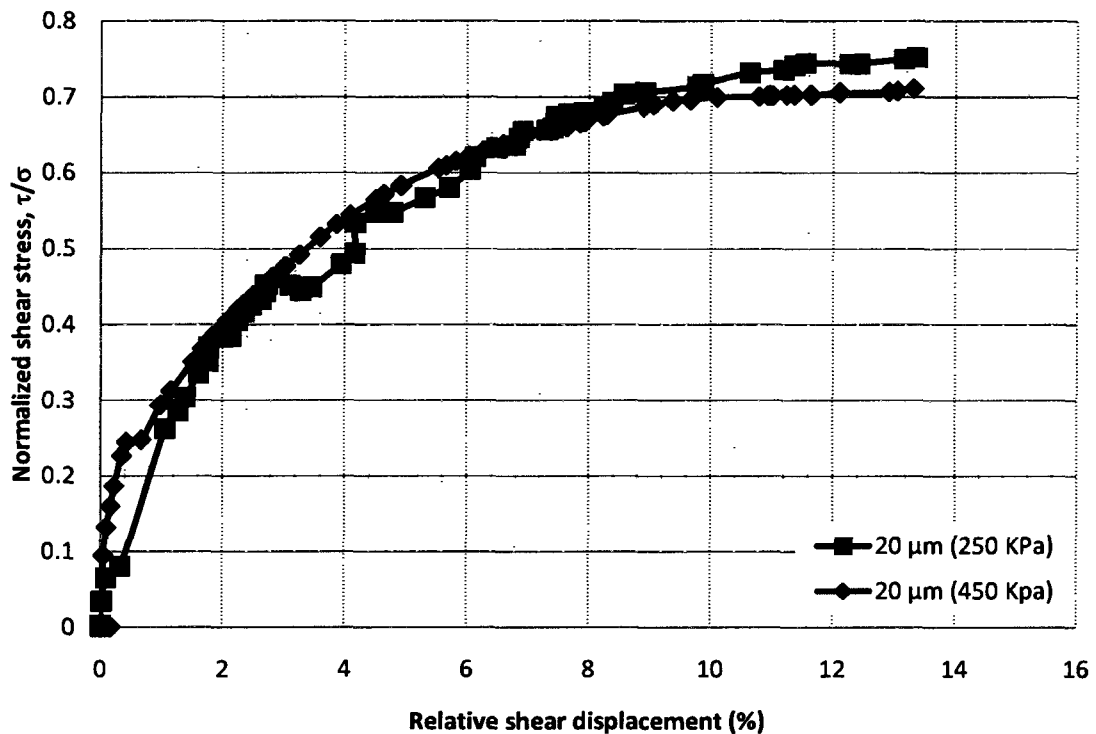


Figure 4-8: Normalized shear stress vs. relative displacement comparison at 250 and 450 kPa normal loads with 20 μm roughness.

Figure 4-8 reveals that the normalized shear stress (normalized by the consolidation pressure) vs. relative displacement behavior is independent of the consolidation pressure. This characteristic is similar to the one in normally consolidated clays. Tsubakihara and Kishada (1993b) also came across the same observation in their steel interface tests.

Figure 4-9 illustrates the shear strength envelopes for the interface tests as well as the clay-clay shear test. These envelopes are the results of fitting regression lines through every set of interface shear vs. normal stress data. A Mohr-Coulomb type equation is used to obtain the values of the interface friction angle (δ) for the range of normal stress

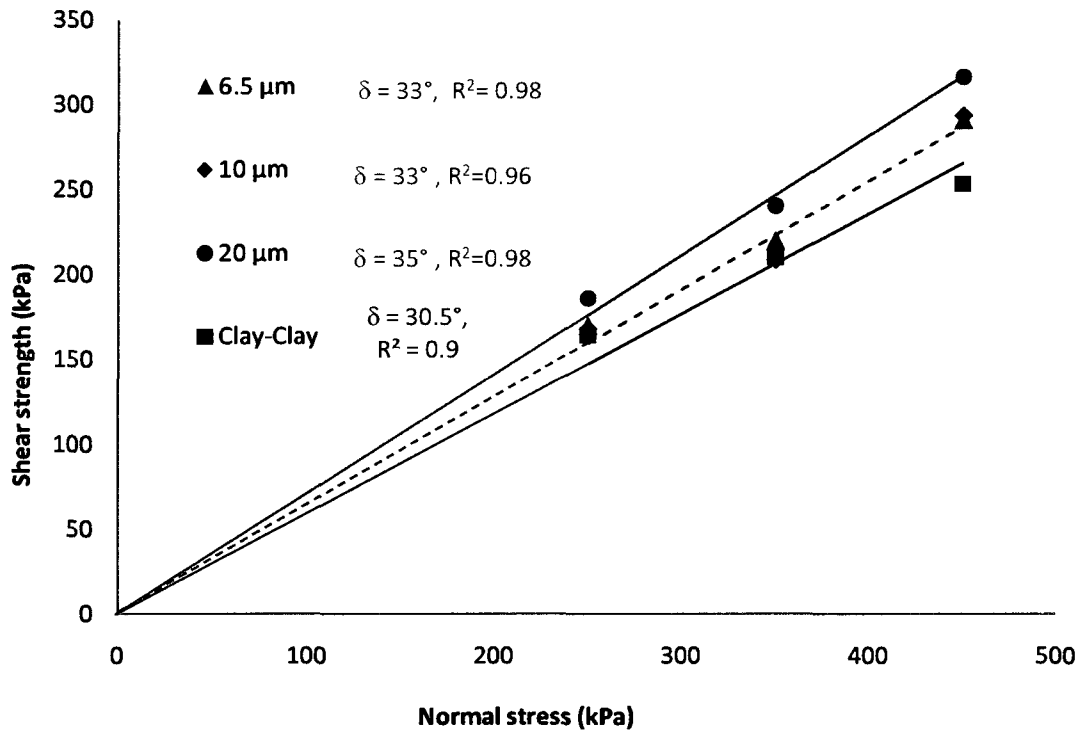


Figure 4-9: Mohr- Coulomb diagram comparison between concrete with different surface roughness and Leda clay.

Figure 4-9 indicates that the angle of interface friction increases as the surface roughness significantly increases. However, the interface friction of the 10 μm and 6.5 μm surfaces is almost equal. This is an indication that the concrete interface friction angles are insensitive to minor changes in the surface roughness. Also, it can be noted that the interface friction angles are higher than that of pure clay. This can be attributed to the fact that clay asperities face higher interlocking forces when sheared against concrete asperities.

4.4.2. Effect of salt content on the Leda clay-concrete interface shear behavior

Hayens and Quigley (1976) carried out testing on four boreholes in Leda clay in Hawkesbury, Ontario, to study their geochemical and mineralogical profiles. By assessing the minerals that were in the Leda clay, they concluded that sensitivity is inversely proportional with its salinity (Quigley, 1980). Moreover, Torrance (1975)

reported that the interaction between clay particles is influenced by pore-water salt concentrations. Therefore, it should be expected that a change in the salt concentration within the pore water of Leda clay would have a significant effect on the Leda clay-concrete interface shear behavior.

Figures 4-10 and 4-11 show typical shear stress vs. relative shear displacement and vertical displacement vs. relative displacement curves of Leda clay interfaces with various NaCl content (4g/l, 10 g/l, 20 g/l) under 450 kPa normal stress.

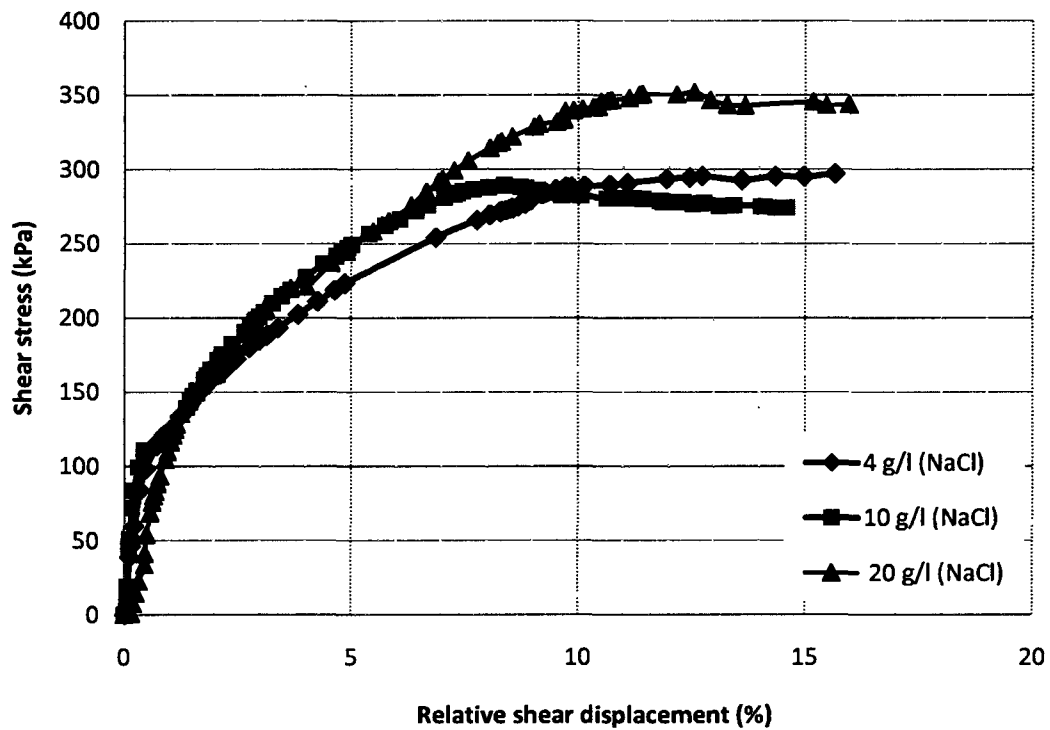


Figure 4-10: Shear stress vs. relative shear displacement at 450 kPa for NaCl concentrations of 4g/l , 10 g/l and 20 g/l ($R_{max} = 10 \mu m$).

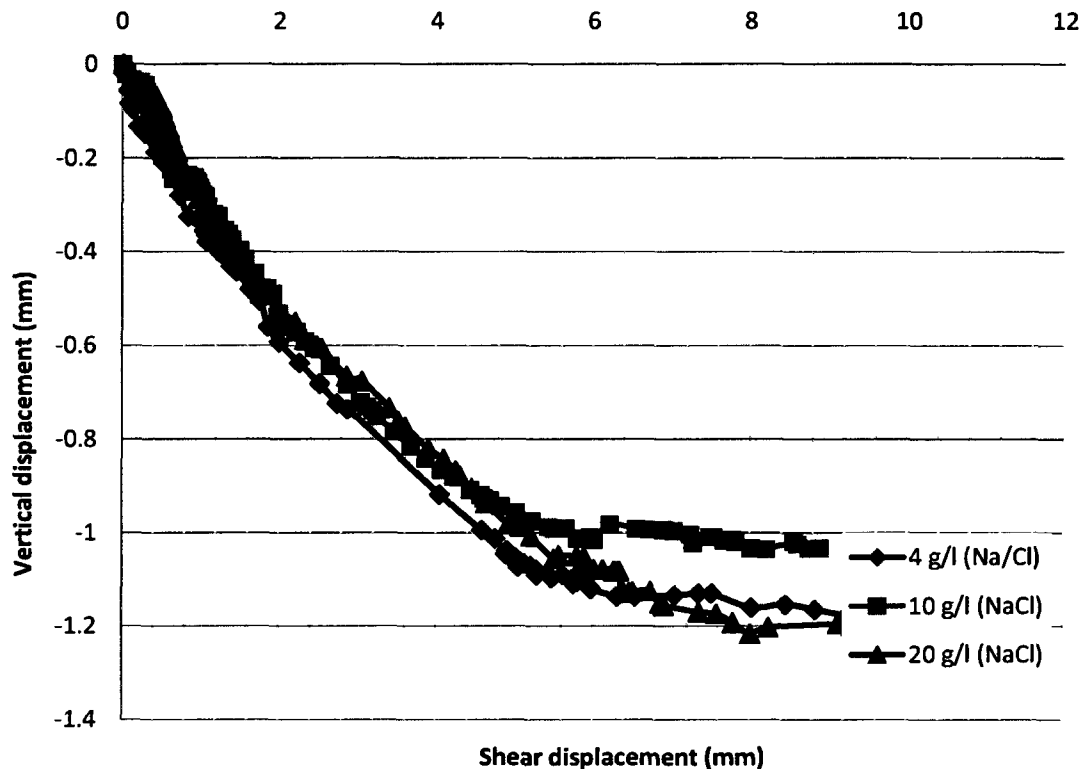


Figure 4-11: Vertical displacement vs. relative shear displacement comparison at 450 kPa and NaCL concentrations of 4g/l, 10g/l and 20 g/l.

Figure 4-10 indicates that the interface shear resistance significantly increases as Leda clay's salt concentration increases from 10 g/l to 20 g/l. It was explained in research carried out by Torrance (1975) that increasing the salinity of sensitive clay decreases its sensitivity and enhances its undrained shear strength (C_u). This behavior can be attributed to the fact that the plate-shaped particles of the clay's layered silicate minerals have a net negative charge on their faces and a small positive charge on their edges. These charges are responsible for the flocculation of Leda clay. At high salinities and high concentrations of ions, the effect of the negative charges is suppressed around the particles and the repulsion is decreased. Also, a high concentration of ions makes it possible for the positive edge charges of one particle to interact with the negative surface charges of another particle, which allows an edge-to-face bond to form. Hence, this interaction leads to the increase of bonds between the Leda clay particles (Torrance, 1975). In this research, it is found that the

sensitivity of the Leda clay samples reduces by around 50% as salinity increases up to 20 g/l from laboratory vane shear testing. The Atterberg limits results revealed that the LL and plasticity index (PI) are only slightly different up to a 10 g/l salt concentration, but decrease when the salt content is beyond 10 g/l (Table 4-2). This behavior is similar to the result of the influence of various salts on the LL of Leda clay from the Heron Road - Bronson Avenue interchange in Ottawa (Torrance 1975). These Atterberg limit changes produce a stronger clay structure that resists the interlocking forces between clay and concrete asperities.

Table 4-2: LL and plasticity index of Leda clay with various salt contents.

NaCl Concentration	4 g/l	10 g/l	20 g/l
LL	66	69	60
PI	41	42	30

Figure 4-11 indicates that up to a relative displacement of approximately 8%, the vertical displacement at the interface is not significantly sensitive to the changes in the salt content. However, beyond the 8 % displacement range, specimens with 10 g/l concentration of NaCl show less vertical displacement than specimens with 20 g/l and 4 g/l of NaCl.

4.4.3. Effect of the initial saturation degree

Underground foundation structures (e.g., piles and retaining wall backfills) which come into contact with unsaturated soils have a contact zone known as an “unsaturated interface”. The unsaturated interface problems have been investigated by several researchers (e.g., Hamid and Miller 2009, and Sharma et al. 2007) for accurate performance, prediction and proper characterization of the interface strength properties (Hamid and Miller 2009). However, no study has addressed the interface shear behavior of unsaturated Leda clay. Therefore, this research has carried out an investigation on the influence of various initial saturation degrees of Leda clay on its interface shear behavior.

Figures 4-12 and 4-13 show representative examples of the effect of initial saturation degrees (or suction) on the shear stress vs. relative shear displacement curves of the interface between Leda clay specimens and concrete. These figures reveal that the placement saturation degree has a considerable effect on the shear resistance of the Leda clay interfaces.

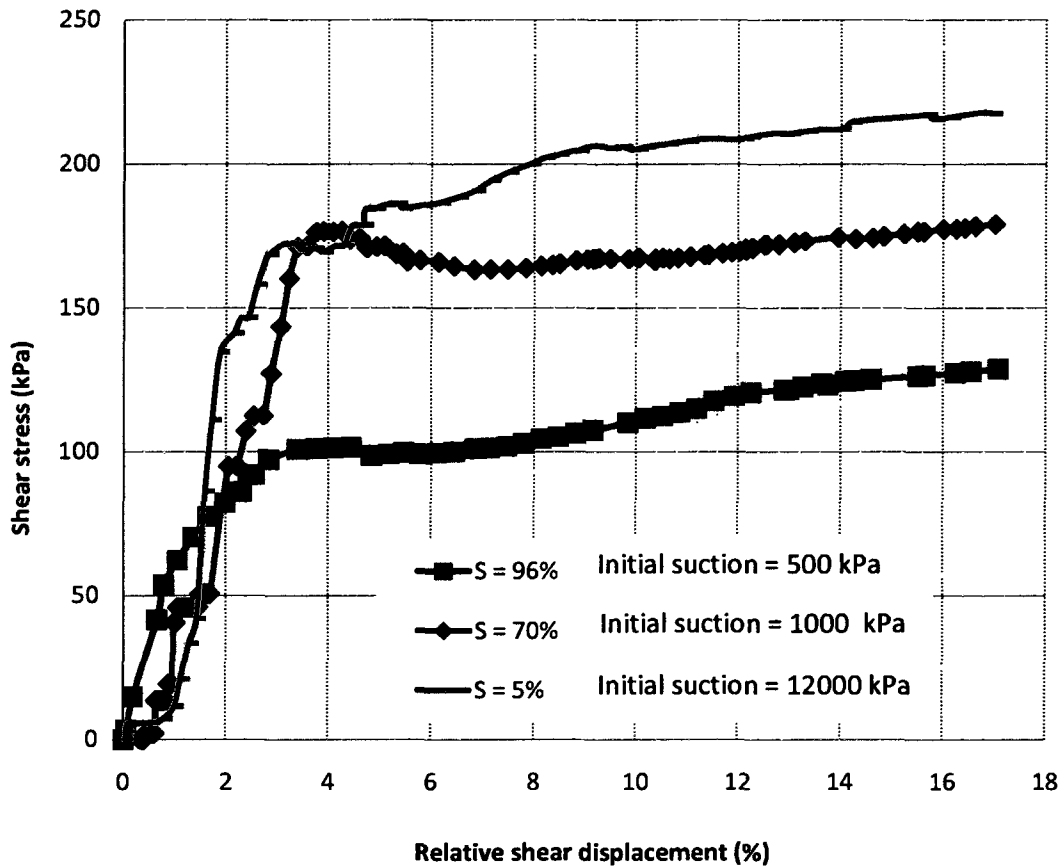


Figure 4-12: Shear stress vs. relative shear displacement at 250 kPa and 96%, 90 % and 5 % initial saturation degree.

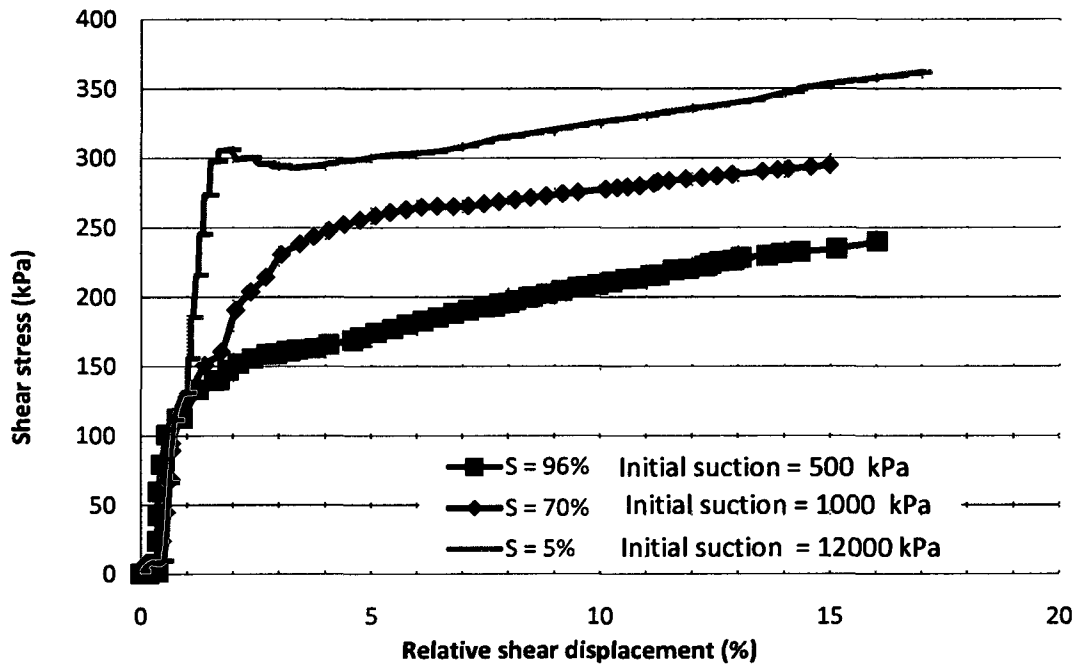


Figure 4-13: Shear stress vs. relative shear displacement at 450 kPa for 96%, 90 % and 5 % initial saturation degrees.

It can be noticed from Figures 4-12 and 4-13 that the peak shear stress increases with an increase in the initial suction, i.e. reduction in saturation degree. This can be attributed to the following mechanisms: (i) the shear resistance which is required to overcome the interlocking force between Leda clay and concrete asperities which increases as the initial saturation degree of Leda clay decreases or as the initial matrix suction increases; and (ii) the increase of bonds between clay particles due to the matrix suction which leads to stronger clay asperities.

The vertical displacement behavior of the unsaturated interfaces at 450 kPa normal stress is illustrated by the vertical displacement vs shear displacement curves shown in Figure 4-14.

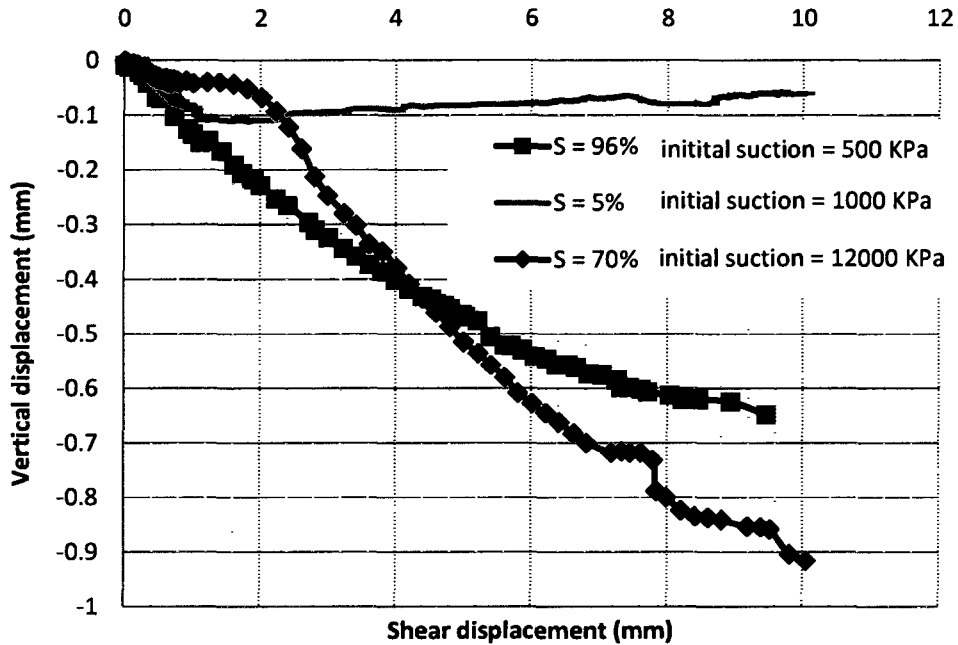


Figure 4-14: Vertical deformation vs. relative shear displacement comparison at 450 KPa and 96%, 90 % and 5% saturation.

From Figure 4-14, it can be seen that compression is the vertical displacement behavior during shearing. This behavior is similar for all tested initial saturation degrees (5 %, 70 %, and 96%) up to 2% relative displacement which corresponds to the peak shear stress of the 5 % sample. Beyond this 2% relative displacement, while the 70% and 90% interfaces continue to show compression behavior (in which vertical displacement increases with shear deformation), the “dry interface” (5%) shows slight dilation behavior. This behavior can be attributed to the fact that a very low saturation degree or moisture develops a rigid clay structure that undergoes the least contraction. Moreover, the reduction in void spaces due to clay shrinkage contributes to this minimal contraction behavior. Similar contraction behavior was observed by Hamid and Miller (2009) during their interface tests on Oklahoma Minco silt at a net normal stress of 105 KPa.

4.4.4. Effect of dry density on the Leda clay interface shear behavior

In the construction practice, many soils have been widely used as foundation material for highways and embankments. Moreover, underneath shallow foundations and beside retaining walls, soils have been compacted in layers to support these structures. The interaction between compacted soils which come into contact with concrete structures gives rise to studying the interface zone. Therefore, this research has carried out interface tests to examine the interface shear behavior of disturbed Leda clay prepared at different dry densities.

Under a normal stress of 450 kPa, Figures 4-15 and 4-16 illustrate the effect of increasing the dry density on the relative shear displacement vs shear stress and vertical displacement curves of the Leda clay-concrete interface.

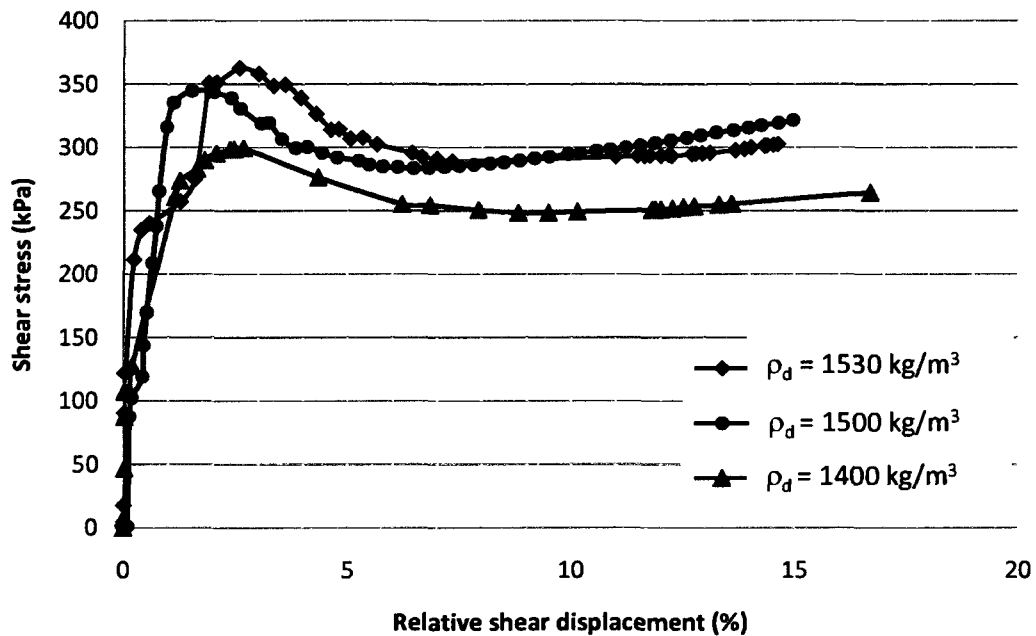


Figure 4-15: Shear stress vs. relative shear displacement at 450 KPa for dry densities of 1530, 1500 , 1400 kg/m^3 . ($R_{\text{max}} = 10 \text{ }\mu\text{m}$)

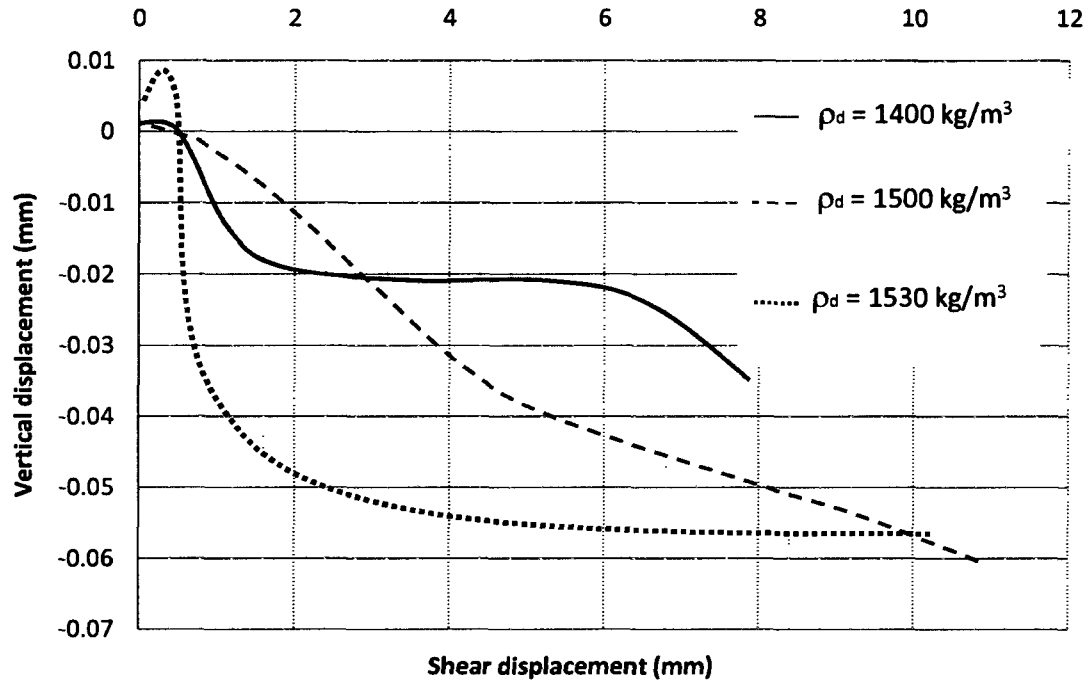


Figure 4-16: Vertical displacement vs. relative displacement at 450 kPa for dry densities of 1500, 1530, 1400 kg/m³.

It can be observed from Figure 4-15 that the peak interface shear stress is achieved within a 1% to 3 % shear displacement range followed by a reduction to a post peak value. Also, the peak interface shear stress increases with an increase in the dry density. This behavior can be attributed to the strengthening Leda clay asperities by densification that lead to higher clay-concrete interlocking forces. Furthermore, at higher dry density values, more clay particles are in contact with the surface of the concrete, which results in an increased contact area, and therefore, increased interface shear resistance. These results agree with the research observations by DeJong and Wetstgate (2005, 2009) on cohesionless soils which indicated higher interface shear resistance with dense specimens and lower shear resistance with loose specimens.

Figure 4.16 shows that in the case of Leda clay with the highest dry density, the concrete-Leda clay interface undergoes dilation within 1% of shear displacement. This behavior can be attributed to the escalating of clay particles from their embedment locations within the concrete aspirates, which leads to expansion at the

interface and higher shear stress concentrations. However, in the post peak range, crushing the clay's aspirates results in more contraction in the interface with clay at the optimum dry density compared with that of clay specimens with less dry density. This behavior can be attributed to the fact that the presence of more moisture content in the optimum dry density specimens allows more particle parallelism and reorientation. It can be observed that before reaching the peak shear stress of the remolded Leda clay, this vertical deformation behavior is similar to that recorded by DeJong and Westgate (2005 and 2009). On the other hand, the disagreement between the Leda clay interface results in the post peak range and the results of these researchers can be attributed to the nature of Leda clay which is sensitive to the presence of water.

4.5. Conclusions

The following points summarize the paper's main findings:

- in the CD test series for all concrete interface surfaces, shearing within clay dominates the interface shear displacement behavior;
- the interface friction angle increases as the Leda clay-concrete interface surface roughness increases. Also, increasing the surface roughness does not cause significant contractions at the interface. Hence, rough concrete piles can be used with Leda clay in order to increase the skin friction capacity of the piles;
- the interface friction resistance increases as the pore water salinity of Leda clay increases. Thus, chemical treatment could be an appropriate soil improvement method for the Leda clay interface;
- Leda clays with lower saturation degrees or matrix suction show higher interface friction angles and less interface contraction; and
- from the dry of optimum side, the interface angle increases as the Leda clay dry density increases.

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Chapter 5

Conclusions and Recommendations

5.1. Summary

This study has carried out a comprehensive experimental program to understand the shear behavior of Leda clay with respect to the two most common structural interfaces, steel and concrete. The study has examined the effects of the most common parameters that are investigated in the field of soil–structural interfaces: interface roughness, soil OCR, soil density and soil saturation degree. Moreover, the study investigates the effect of salt content on the interface shear behavior which is considered an additional parameter for research work in this field. The study has three main parts:

- a review of the geological and geotechnical properties of sensitive marine clays and the interface problem
- experimental characterization and analysis of the shear behavior of the Leda clay-steel interface, and
- experimental characterization and analysis of the shear behavior of the Leda clay-concrete interface.

5.2. Conclusions

5.2.1. Shear behavior of Leda clay-steel interfaces

An experimental program has been carried out to study the effects of interface surface roughness, OCR, saturation degree, dry density and salt content on Leda clay-steel interface shear behavior. It is noticed during the execution of the testing program that the interface sliding failure dominates the shear displacement behavior except when the steel interface is relatively rough. The interface testing also reveals that as the steel surface roughness increases, the interface friction angle also increases. In addition, it is also found that the interface friction angle increases as the Leda clay's OCR increases. A lower saturation degree at the Leda clay-steel interface enhances the interface shear resistance. Moreover, the interface testing which uses Leda clay with different dry densities and water content reveals that on the dry side of optimum, the interface shear resistance increases as the dry density increases. Finally, the salinity of the Leda clay's pore water has a positive effect on its interface shear resistance.

5.2.2. Shear behavior of Leda clay-concrete interfaces

The experimental program that was carried out to study the shear behavior of the interface between Leda clay and concrete includes an investigation of the effects of concrete surface roughness, saturation degree of the clay, dry density of the clay and salt content on the Leda clay-concrete interface shear behavior. For all concrete interface surfaces, the CD test series reveal that shearing within clay dominates the interface shear displacement behavior. On the other hand, the CU test series indicate that sliding failure occurs when Leda clay is relatively dry. Also, the interface test results indicate that the interface friction angle increases as concrete surface roughness increases. Furthermore, the obtained results show that the Leda interface shear resistance is a function of the Leda clay's saturation degree. The shear resistance increases as the initial saturation degrees decrease, or as the initial suction increases. The results also indicate that the interface shear resistance increases as the

Leda clay's dry density increases. Finally, higher pore water salt content of the Leda clay results in improvement of the shear resistance of the Leda clay-concrete interface.

5.3. Practical implication

The following practical implications can be related to the Leda clay-structure interface problem:

- Correcting the methodology of calculating piles skin friction and the active and passive earth pressure coefficient related to earth retaining walls designs. In both cases, designers under estimate the interface friction and assume that it is equal to or less than the internal friction of the soil while it is proven by this research that the interface friction can be higher than that of Leda clay (Bowels, 1997). Higher interface friction would lead to cost effective design.
- The use of rough steel or concrete surfaces enhances the interface shear strength of the geotechnical structure that is in contact with Leda clay such as piles earth retaining walls sheet piles and anchor blocks.
- Compacting Leda clay on the dry side of optimum behind earth retaining structure increases the interface shear strength and reduces the interface contraction.
- Preloading Leda clay to achieve the overconsolidation conditions would enhance the interface shear strength of the geotechnical structure in contact with Leda clay.
- Reducing the saturation of Leda clay enhances the interface shear strength. In relatively dry stratum of Leda clay, considering this result in calculating the skin friction and earth coefficients would lead to cost effective designs.
- Chemical treatment could be a suitable means to improve the interface shear strength of Leda clay and hence, enhance its interaction with the geotechnical structure. This could be achieved through injecting saline water to increase the ion concentration in Leda clay's pore water.

5.4. Future Research Recommendations

The following future research recommendations are made:

- To test a larger variety of Leda clay (e.g., higher sensitivity, various pore water salinity compositions, different mineral composition, from different geographical areas).
- To study the interface shear behavior between the Leda clay and wood as construction material. Wood is a common construction material in Canada and Scandinavia.
- To study the residual shear strength parameters of the interface between the Leda clay and construction materials (steel, concrete, wood). The residual interface shear strength cannot be obtained with the conventional direct shear machine.
- To conduct suction-controlled interface shear tests on Leda clay to investigate the unsaturated interface shear behavior. This will need the modification of the direct shear machine used in this study
- To develop a model that can predict the shear behavior of the Leda clay interface.

5.5. References:

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6-Appendix

6.1. Lab photos

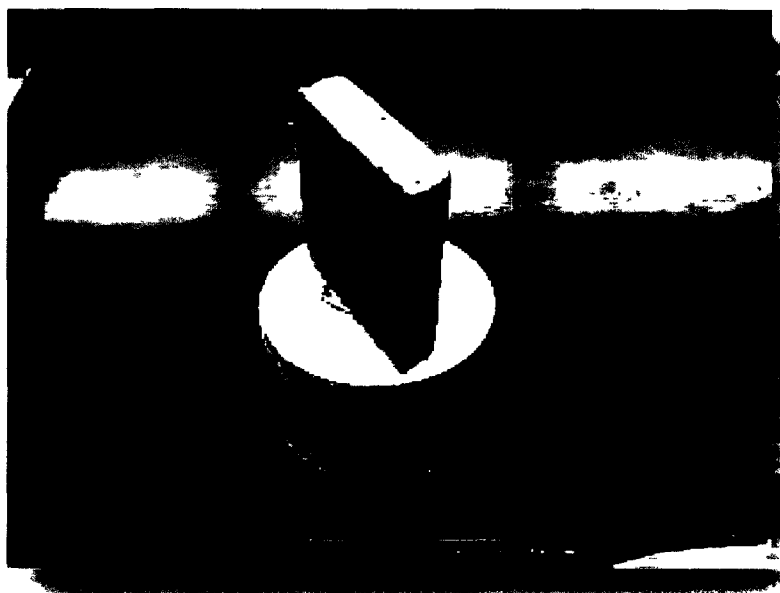


Figure 6-1: Undisturbed Leda clay sample after preparation for the interface test



Figure 6-2: The automatic direct shear machine

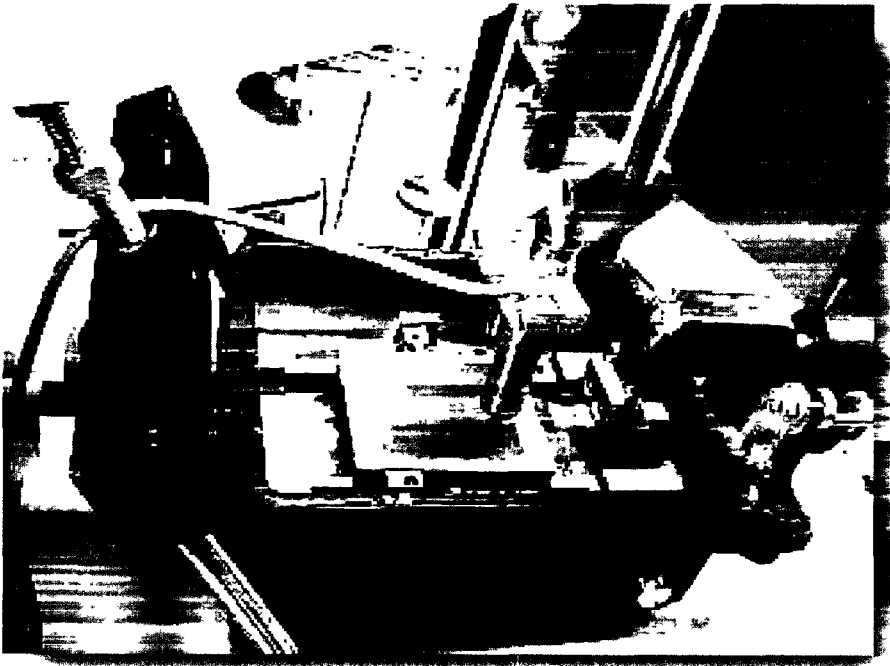


Figure 6-3: Measurement of the surface roughness

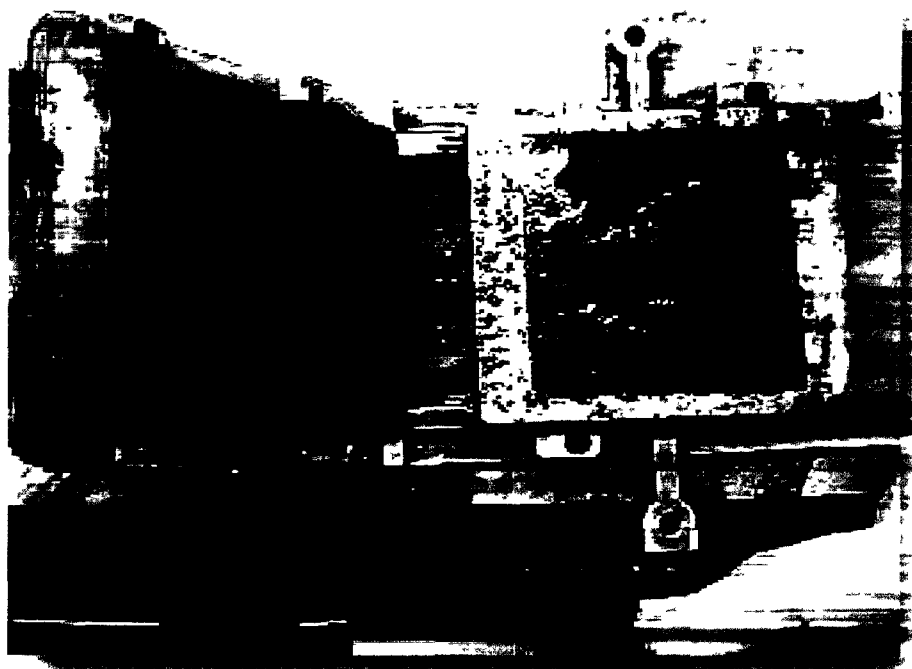


Figure 6-4: The two halves of the shear box

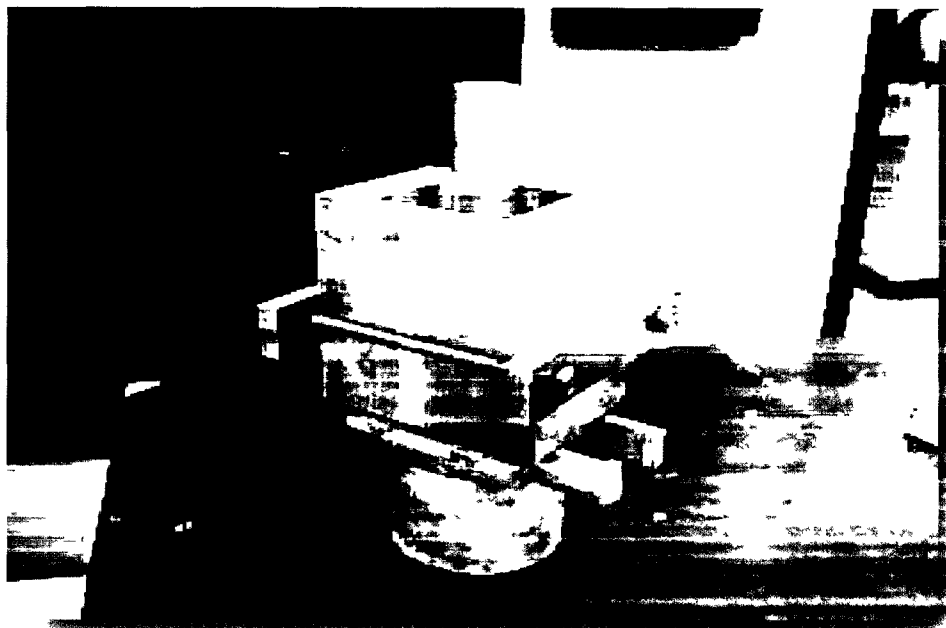


Figure 6-5: Static compaction mold

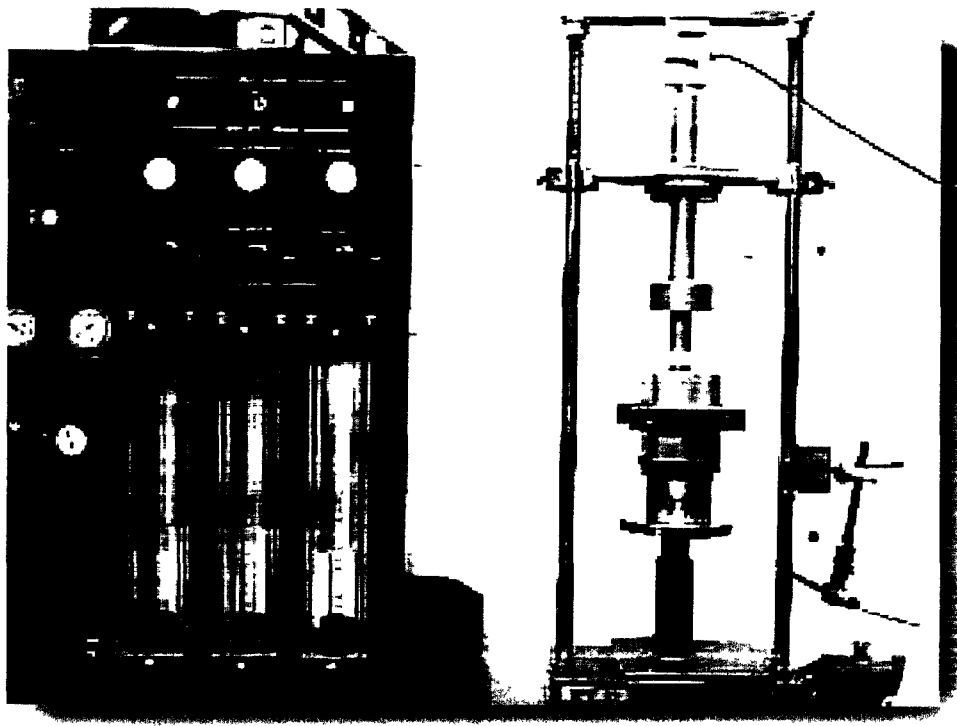


Figure 6-6: The process of static compaction

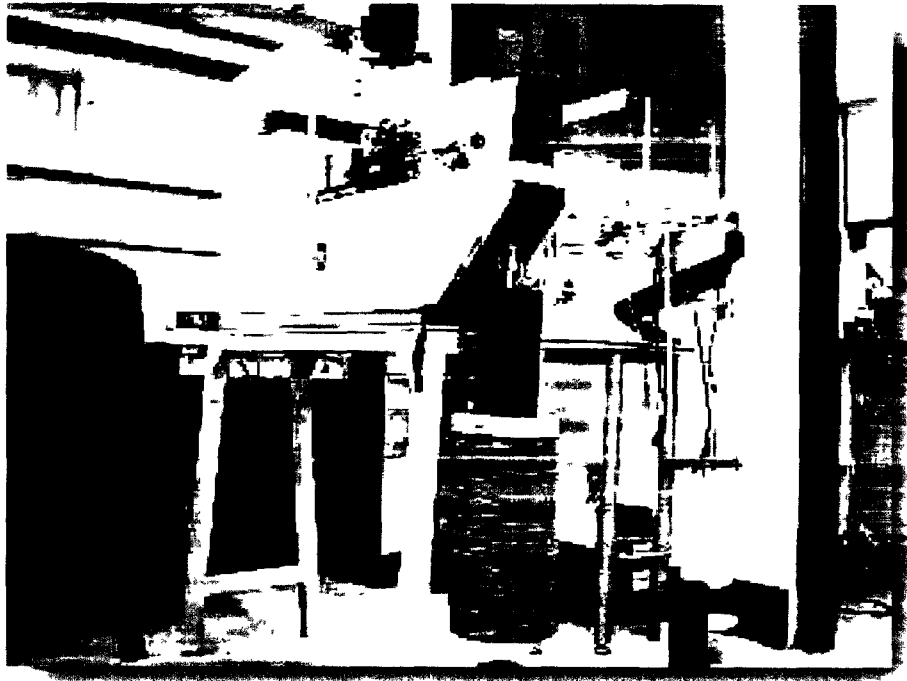


Figure 6-7: Consolidation loads being added to over-consolidated Leda clay

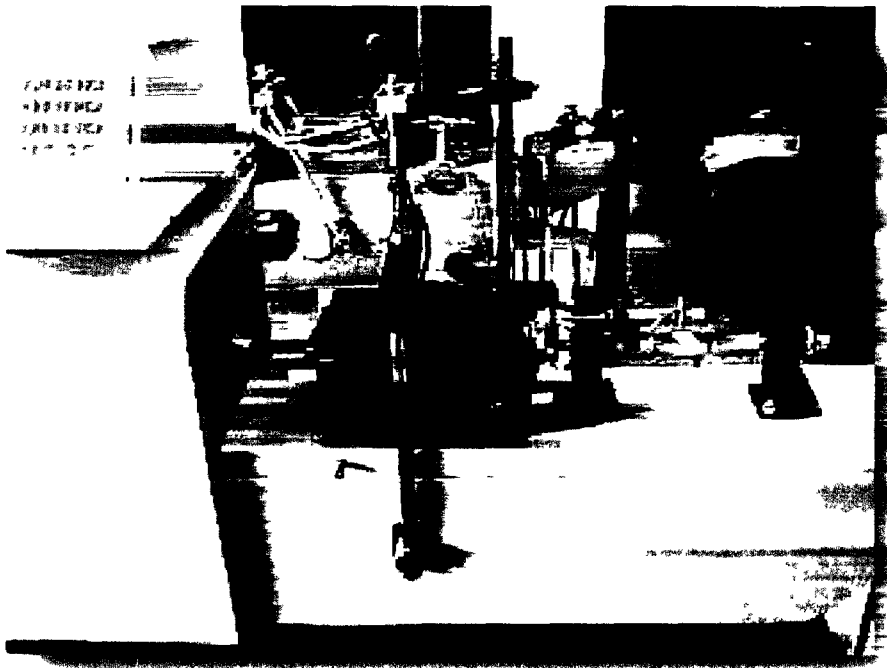


Figure 6-8: Wet cloth covering a Leda clay specimen during a (CU) tests

6.2. Lab plots

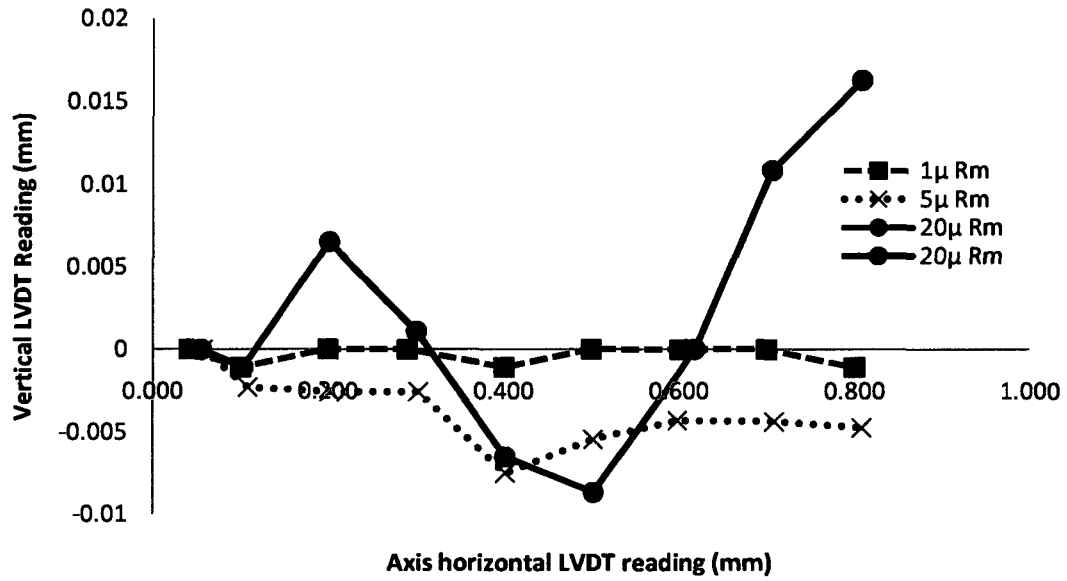


Figure 6-9: Steel interface profiles for different roughness

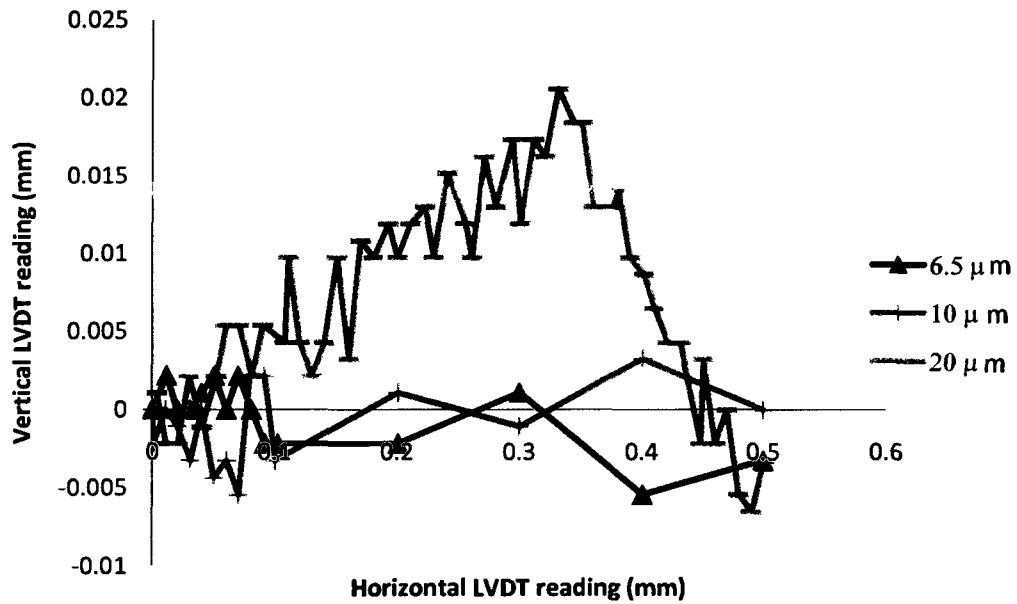


Figure 6-10: Concrete interface profiles for different roughness

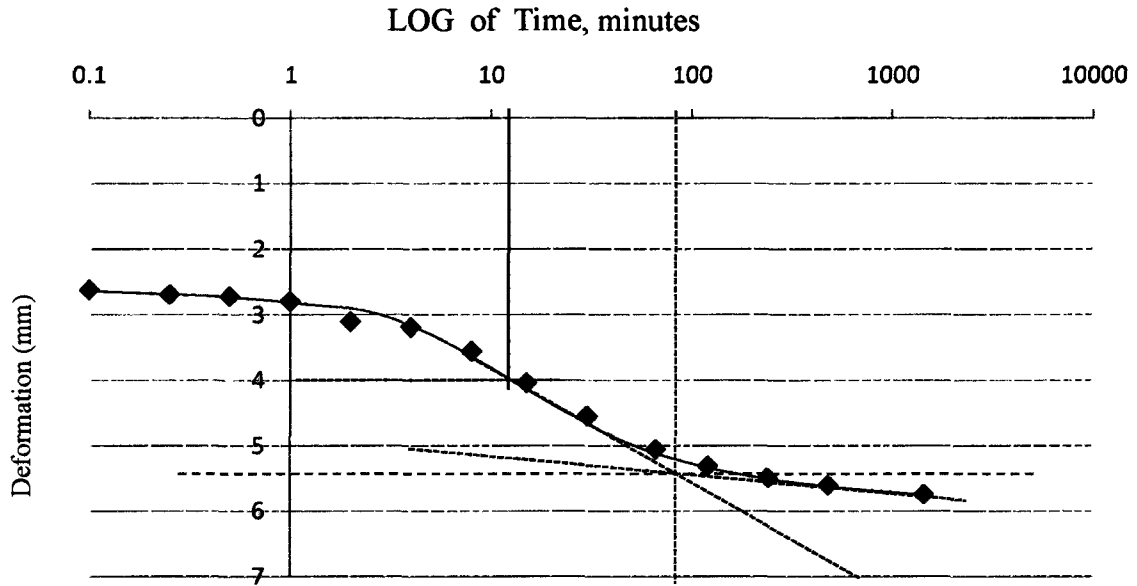


Figure 6-11: time-deformation Curve ; Log of time Method

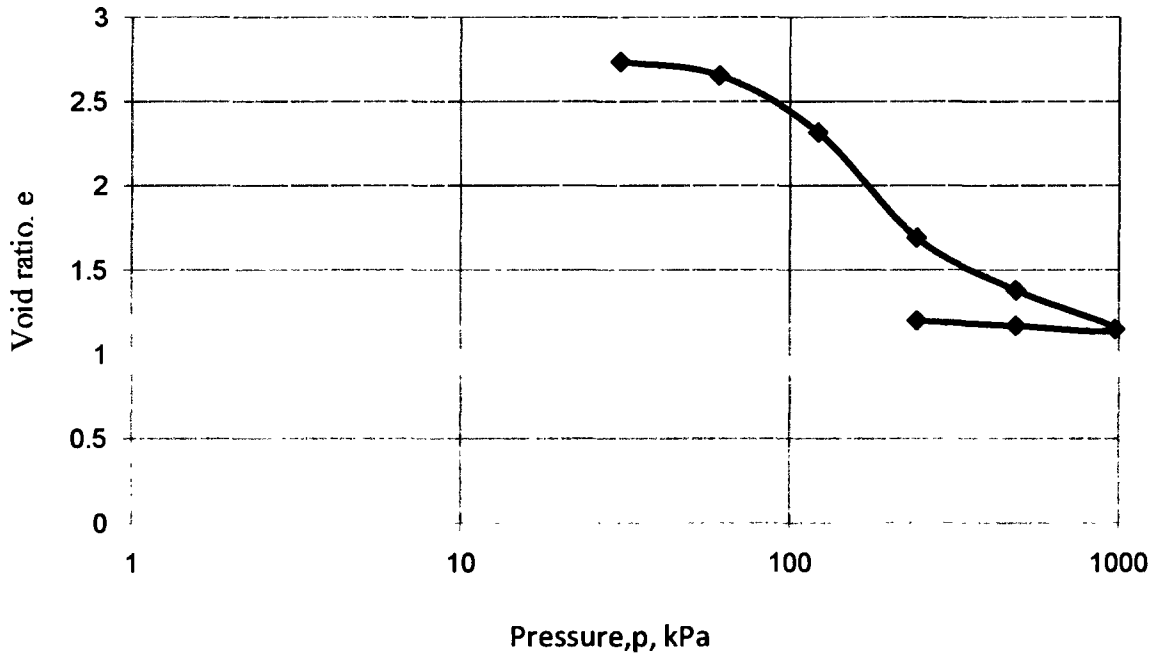


Figure 6-12: Leda clay consolidation test plot