COMPRESSIBILITY AND NORMALIZED UNDRAINED SHEAR BEHAVIOR OF SOFT COASTAL FINE-GRAINED SOILS

Arash Pirouzi
University of Massachusetts Amherst

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COMPRESSIBILITY AND NORMALIZED UNDRAINED SHEAR BEHAVIOR
OF SOFT COASTAL FINE-GRAINED SOILS

A Dissertation Presented
by
ARASH PIROUZI

Submitted to the Graduate School of the
University of Massachusetts Amherst in partial fulfillment
of the requirements for the degree of

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Civil and Environmental Engineering
COMPRESSIBILITY AND NORMALIZED UNDRAINED SHEAR BEHAVIOR
OF SOFT COASTAL FINE-GRAINED SOILS

A Dissertation Presented
by
ARASH PIROUZI

Approved as to style and content by:

________________________
Don J. DeGroot, Chair

________________________
Guoping Zhang, Member

________________________
Jon Woodruff, Member

________________________
Jason T. DeJong, Member

Richard N. Palmer, Department Head
Department of Civil and Environmental Engineering
DEDICATION

To my selfless parents Lotfollah and Farkhondeh
my supportive siblings Shirin and Kourosh
and
my loving wife Talaye
ACKNOWLEDGMENTS

I would like to express my sincere gratitude to my advisor and mentor Dr. Don DeGroot for his dedication supervision and guidance throughout this journey. The greatest admiration is given to his knowledge of laboratory testing and soil behavior as well as his teaching skills. With no doubt he has taught me more about geotechnical engineering than any other person and I am truly indebted to him for that. I would like to thank Dr. Guoping Zhang, Dr. Jon Woodruff, and Dr. Jason DeJong for serving as my committee members and for their guidance throughout this project.

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Finally, I would like to acknowledge GeoEngineers, Inc. for providing data, some of the tested soils, and partial funding for this research.
ABSTRACT

COMPRESSIBILITY AND NORMALIZED UNDRAINED SHEAR BEHAVIOR OF SOFT COASTAL FINE-GRAINED SOILS

SEPTEMBER 2018

ARASH PIROUZI, B.S., AZAD UNIVERSITY, AHVAZ, IRAN

M.S., AZAD UNIVERSITY, TEHRAN, IRAN

Ph.D., UNIVERSITY OF MASSACHUSETTS AMHERST

Directed by: Professor Don J. DeGroot

This thesis investigates empirical correlations between consolidation design parameters and index properties of soft fine-grained soils from coastal Louisiana region, normalized undrained shear behavior of high liquid limit organic fine-grained coastal soils, and consolidation behavior of fine-grained soils.

The first phase of this research consisted of studying a database of site investigation data from 15 marsh creation projects across the coastal Louisiana region. The database includes a wide variety of fine-grained soils ranging from low-plasticity inorganic clays and silts to high-plasticity organic clays and silts with a large range of water content and liquid limit. Most of the empirical correlations in the literature do not cover the soils in this data set. Correlations between consolidation parameters (compressibility, preconsolidation stress, and coefficient of consolidation) determined from 1-D incremental loading consolidation tests and index properties (water content, void ratio, Atterberg Limits, and dry unit weight) were developed. The degree of correlation between the index parameters and different consolidation design parameters varied significantly. In many cases, considering inorganic and organic soil separately improved the correlations.
The second phase of this research investigated the undrained shear behavior of high liquid limit, organic soils from coastal Louisiana region over the consolidation effective stress range of 50 to 1600 kPa. Undrained direct simple shear (DSS) behavior of 6 resedimented natural organic soils with liquid limit ranging from 81 to 215% and two natural inorganic soils with liquid limit equal to 45% and 46% was studied. CK₀UDSS tests were performed on normally consolidated samples. Normalized undrained shear strength and normalized undrained Young’s modulus decreased with increasing consolidation stress level. The organic soils had significantly higher normalized undrained shear strengths than the inorganic soils especially at lower stresses with the difference became smaller at higher stresses. The rate of decrease in normalized undrained shear strength was found to correlate well with liquid limit or organic matter and new correlations were developed to relate undrained shear strength and consolidation stress level as a function of liquid limit. Such correlations were not observed for normalized undrained modulus and liquid limit or organic matter. Thus, a collection of plots of undrained modulus normalized by undrained shear strength versus applied stress ratio for the organic soils tested are provided.

The third phase of this research involved a suite of CRS consolidation tests to investigate different methods of determining the recompression ratio (RR). Tests were performed on a variety of natural clays and silts from different quality samples (intact, highly disturbed, and resedimented) by conducting unload-reload loops at different stress levels and different unloading ratios. Seven different methods were used to determine recompression ratio from each loop resulting, on average, in over 240% difference in RR estimates from the different methods on a loop. The results showed that RR from all the methods increased with increasing stress level and unloading ratio with higher influence
for higher OCR soils and sensitive clays. Recommendations for practice are provided for conduct of CRS tests and how to interpret the test results to best estimate RR.
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LIST OF SYMBOLS

\( C_\alpha \)  Coefficient of secondary compression

\( C_c \)  Compression index

\( C_r \)  Recompression index

\( c_v \)  Vertical coefficient of consolidation

\( e \)  Void ratio

\( e_0 \)  Initial void ratio

\( E_u \)  Secant Young's Modulus

\( G \)  Shear modulus

\( G_s \)  Specific gravity of solids

\( H_0 \)  Thickness of soil layer

\( I_c \)  Soil behavior type

\( k \)  CPTU correlation coefficient for \( \sigma'_p \)

\( K \)  Stress ratio

\( K_c \)  Consolidation stress ratio = \( \sigma'_h/\sigma'_v \)

\( K_0 \)  Coefficient of lateral earth pressure at rest

\( LL \)  Liquid limit

\( LI \)  Liquidity index = \( (w - PL)/PI \)

\( M \)  Constrained (oedometer) modulus = \( \Delta \sigma'_v/\Delta \varepsilon_v \)

\( m_v \)  Coefficient of volume change

\( N_{kt} \)  Cone factor

\( PI \)  Plasticity index

\( PL \)  Plastic limit
\( q \) Shear stress \( = (\sigma_v - \sigma_h)/2 \)

\( q_t \) Corrected tip resistance

\( Q_{tn} \) Normalized tip resistance

\( R_f \) Friction ratio \( = f_s/q_t \)

\( s_{ci} \) Consolidation settlement

\( S_{FV} \) Field vane undrained strength ratio

\( S_t \) Sensitivity

\( s_u \) Undrained shear strength

\( t \) Time

\( u \) Shear induced pore pressure (psf or tsf)

\( w \) Water content (\%)

\( w_L \) Liquid limit

\( w_n \) Natural water content (\%)

\( w_p \) Plastic limit

\( \Delta \) Change

\( \Delta L \) Horizontal displacement

\( \Delta H \) Vertical displacement (change in specimen height)

\( \Delta u \) Equivalent DSS shear induced pore pressure

\( \varepsilon_v \) Vertical strain

\( \varepsilon_{vc} \) Vertical consolidation strain

\( \varepsilon_{vf} \) Final vertical consolidation strain

\( \varepsilon_{vol} \) Volumetric strain
\( \varepsilon_{vmax} \)  Maximum vertical strain during consolidation phase of DSS test

\( \mu \)  Bjerrum's field vane test correction factor

\( \gamma \)  Shear strain for DSS test

\( \gamma_d \)  Dry unit weight

\( \gamma_t \)  Total unit weight

\( \gamma_w \)  Unit weight of water

\( \% \)  Percentage

\( \sigma'_v0 \)  In situ vertical effective stress

\( \sigma'_hc \)  Horizontal effective stress at end of consolidation

\( \sigma'_h0 \)  In situ horizontal effective stress

\( \sigma'_p \)  Preconsolidation stress

\( \sigma'_v \)  Vertical effective stress

\( \sigma'_v0 \)  In situ total vertical stress

\( \sigma'_vc \)  Vertical effective stress at end of consolidation

\( \sigma'_vf \)  Final vertical effective stress (at mid height soil layer)

\( \sigma'_{vmax} \)  Maximum vertical stress during consolidation phase of DSS test

\( \tau_h \)  Horizontal shear stress
CHAPTER 1

INTRODUCTION

Coastal Louisiana has been experiencing significant land loss in the past several decades resulting in an increase in flooding frequency and impact. Marsh creation projects, conducted by construction of confined areas with dykes and placement of dredged material from seabed or waterways in these areas, are a common practice in the area to restore lost land. Numerous projects are being planned for construction requiring undisturbed sampling and conducting laboratory consolidation testing. Empirical correlations can be a useful tool in such cases to estimate design parameters in early stages of the projects or as a quality control measure. The first objective of this research was to develop a database of index properties and consolidation design parameters from past marsh creation projects and explore creation of empirical correlations between consolidation parameters and index soil properties for this region.

Dyke stability is another key design aspect of marsh creation projects. Thus, undrained shear strength anisotropy needs to be considered for stability analyses. The direct simple shear (DSS) mode of testing coupled with the SHANSEP method has proved to be a reliable approach for such problems. However, there is not much data available on DSS shear behavior of the high liquid limit, organic fine-grained soils that are common in the region of marsh creation projects. In addition, recent research has shown that SHANSEP parameters for fine-grained inorganic soils are not constant over a wide range of stresses which is a key assumption of the framework. Therefore, the second objective of this research was to study the undrained DSS behavior of high liquid limit organic fine-grained
soils from the coastal LA region and investigate the stress dependency of the SHANSEP undrained shear strength parameters.

Compressibility parameters as determined from consolidation tests are key soil properties for estimating the magnitude of the consolidation settlement. There has been extensive research conducted on measurement and evaluation of the preconsolidation stress and compression ratio for normally consolidation loading. The recompression ratio for recompression loading from in situ stresses up to the preconsolidation stress has received much less attention generally due to the lower strains, and hence lower consolidation settlement in this stress range. However, for some design problems, such as for example heavy loading of thick high overconsolidation ratio clays obtaining reliable estimates of the recompression ratio can be important. Yet, there is no consensus on the best practice to conduct consolidation tests for determining the recompression ratio. The third objective of this research was to develop a better understanding of the effects of stress level and unloading ratio as well as the different methods of estimating the recompression ratio.

Chapters 2, 3 and 4 present the products from this research for the three topics listed above. All three chapters have been prepared for submission as a journal article. Chapter 2 presents the results of an investigation into the correlations between index soil properties and consolidation design parameters for soft soil of coastal Louisiana region. The author is the lead author, responsible for writing and organizing the paper, processing the data, and developing the correlations. Coauthors on this paper are expected to be DeGroot, D.J. and Zhang, G.

Chapter 3 presents the results of an investigation into the effects of consolidation effective stress level on DSS undrained shear behavior of organic soils from coastal
Louisiana area. The author is the lead author, responsible for writing and organizing the paper, testing, and evaluating experimental results. Coauthor on this paper is expected to be DeGroot, D.J.

Chapter 4 presents the results of an investigation into the best practice for determination of recompression ratio from consolidation tests. The author is the lead author, responsible for writing and organizing the paper, testing, and evaluating experimental results. Coauthors on this paper are expected to be DeGroot, D.J. and DeJong, J.T.
CHAPTER 2

EMPIRICAL CORRELATIONS FOR ESTIMATING FOUNDATION SOIL CONSOLIDATION PARAMETERS FROM INDEX PROPERTIES FOR LOUISIANA MARSH CREATION

This paper presents a collection of empirical correlations for estimating consolidation parameters of fine-grained soils for settlement calculations using basic index and classification measurements. The database was generated using site investigation data from 15 marsh creation projects in the coastal Louisiana region. These projects typically involved low to near normally consolidated soft, high liquid limit organic silts and clays, for which a majority of published empirical correlations do not exist. Index and classification properties included in the database were water content, void ratio, Atterberg Limits, and dry unit weight. Consolidation design parameters (compressibility, preconsolidation stress and coefficient of consolidation) were determined from 1-D incremental loading consolidation test results. The degree of correlation between the index parameters and design parameters varied significantly with the strongest one being for the compression ratio as a function of water content considering inorganic and organic soil separately. No useful correlation was found for the preconsolidation stress which is the most important parameter for settlement calculations. Recommendations for use of the correlations in practice are provided.

2.1. Introduction

The geotechnical engineering literature contains numerous examples of empirical correlations between basic index properties and soil design parameters (e.g., NAVFAC,
1982, Kleven et al., 1986, Kulhawy and Mayne, 1990, Terzaghi et al., 1996, Mitchell and Soga, 2005). The key index properties for clays are water content \((w)\) and the Atterberg limits including plastic limit \((PL)\), liquid limit \((LL)\), plasticity index \((PI)\), and liquidity index \((LI)\). For correlations that use the Atterberg limits it is important to consider that they are performed on completely remoulded soil. Therefore, any naturally formed structure that existed in situ is destroyed and yet it is the in situ structure of the soil that largely controls its behaviour and hence is of most interest in design. Although, Holtz et al. (2011) note that Atterberg limits can correlate well with some engineering properties because both are affected by many similar factors including clay mineralogy, pore water chemistry and geologic history. Thus, in a very general sense differences in the Atterberg limits of clays imply differences in their engineering behaviour. In terms of water content, it is anticipated, again in a very general sense, that if the natural water content is close to the liquid limit the clay will typically be of lower strength and more compressible, i.e., like a low overconsolidation ratio (OCR) clay as compared to a much stronger and stiffer response, i.e., like a higher OCR clay when the water content is close to the plastic limit.

Some of the better empirical correlations for clays involve consolidation parameters (e.g., compressibility, coefficient of consolidation) while the weaker ones tend to be for stress state and shear strength parameters such as preconsolidation stress \((\sigma'_{p})\) and undrained shear strength \((s_u)\). For these latter parameters there is often a strong interrelationship between two design parameters (e.g., the strong link between \(s_u\) and \(\sigma'_{p}\)) thus making it difficult to develop a simple correlation using a single index parameter. Skempton (1969) showed that for normally consolidated clays there is a unique relationship between the in situ vertical effective stress \((\sigma'_v)\) and \(LI\). However, most clays do not exist
in a truly normally consolidated state and the potential existence of such a relationship for overconsolidated clays is often observed empirically. That is, lightly overconsolidated clays often have a water content around the liquid limit ($LI = 1$) while heavily overconsolidated clays often have a water content around the plastic limit ($LI = 0$). In terms of $\sigma_p'$, it is only for the case of removal of overburden would it be expected that overconsolidated clays might also exhibit a unique relationship between $\sigma_p'$ and $LI$. Other factors such as weathering, aging, diagenetic bonding and other physico-chemical processes also change the in situ stress state of a clay (Stas and Kulhawy 1984, Ladd and DeGroot 2003) and the relationship between $\sigma_p'$ and $LI$ becomes weaker and more scattered. Yet in spite of these complicating factors, most empirical correlations for $\sigma_p'$ use $LI$ (e.g., NAVFAC, 1982, Wroth, 1979, Stas and Kulhawy, 1984). NAVFAC (1982) presents a relationship between $\sigma_p'$ and $LI$ that makes use of the sensitivity ($S_t$) to refine the correlation. Stas and Kulhawy (1984) reviewed data for clays with sensitivities ranging from 1 to 10 and suggest a direct correlation between $\sigma_p'$ and $LI$ and contrary to NAVFAC, found no influence of $S_t$ on the correlation.

Many correlations have been presented between the 1-D compressibility of clays, as for example expressed by compression index $C_c$, and either the natural water content or the plasticity index. Terzaghi et al. (1996) present a correlation between $C_c$ and $w$ for a large variety of clays and suggest that such a direct relationship should exist because both properties are controlled by composition and structure unlike $PI$ and $LI$. Leroueil et al. (1983) found that $S_t$ strongly influences the value of $C_c$ and present a correlation that links $C_c$ with the in situ void ratio ($e_0$) and $S_t$ for sensitive clays. The coefficient of consolidation ($c_v$) for the normally consolidated state has been found to correlate well with $w_n$ (e.g., Janbu
1985) while NAVFAC (1982) presents a correlation between $c_v$ and $LL$ for normally consolidated, overconsolidated and remoulded states.

The most reliable way to determine soil parameters is by conducting an integrated site characterization program that combines in situ testing and laboratory consolidation testing on good quality undisturbed samples as described for example in Hight and Leroueil (2003) and Ladd and DeGroot (2003), and DeJong et al. (1998). However, there are practical circumstances for which it may be necessary to rely on empirical correlations such as for example (NGI, 2002): 1) to derive soil design parameters at an early stage (e.g., feasibility study) before advanced laboratory testing is planned or conducted; 2) in projects where budgets for performing advanced laboratory tests are limited or not available, and 3) as a quality control to check whether new testing results for a new site are consistent with previous experience. However, the usefulness and applicability of any correlation are strongly dependent on the reliability of datasets that are used to develop the correlation. Mixing data from a variety of sources can result in increasing scatter and a decrease in the reliability of the correlations because of the differences in measurement techniques and the quality of the data used. Furthermore, there can be significant differences in soil composition and behavior among soils worldwide due to differences in geologic origin, depositional environmental and geologic stress history.

This paper presents the results of a study that analysed an extensive past project database and developed empirical correlations between basic soil index test data and consolidation design parameters for soft fine-grained coastal soils commonly encountered as underlying foundations in Louisiana (LA) marsh creation projects. While the literature presents numerous examples of empirical correlations the majority are not for high liquid
limit and organic soft fine-grained soils that are commonly involved in LA marsh creation projects. The database created in this work includes soil properties from fifteen (15) LA marsh creation projects that cover a wide variety of fine-grained soils ranging from low-plasticity inorganic clays and silts (CL, ML) to high-plasticity organic clays and silts (OH, MH). The paper presents an overview of the created database, presents the developed correlations and provides recommendations for use of the correlations in practice. The work presented herein follows the framework developed by the Norwegian Geotechnical Institute (NGI 2002) for the study of empirical correlations for offshore soils.

2.2. Background

2.2.1 Settlement Calculations

For settlement analyses, the magnitude of the final consolidation settlement for an individual soil layer can be estimated using the generic Eq.

\[ s_{ci} = C_r[H_0/(1 + e_0)]\log \left( \frac{\sigma'_v}{\sigma'_{v0}} \right) + C_c[H_0/(1 + e_0)]\log \left( \frac{\sigma'_{vf}}{\sigma'_p} \right) \]  
Eq. 2.1

where for a given soil layer i

- \( s_{ci} \) = consolidation settlement
- \( C_r \) = recompression index = \( \Delta e/\Delta \log \sigma'_v \) for recompression stresses, \( \sigma'_v \leq \sigma'_p \)
- \( C_c \) = compression index = \( \Delta e/\Delta \log \sigma'_{v} \) for normally consolidated stresses, \( \sigma'_{v} \geq \sigma'_p \)
- \( H_0 \) = thickness of soil layer
- \( e_0 \) = initial void ratio
- \( \sigma'_{v0} \) = vertical effective stress (at mid height soil layer)
- \( \sigma'_p \) = preconsolidation stress (at mid height soil layer)
- \( \sigma'_{vf} \) = final vertical effective stress (at mid height soil layer)
The overall consolidation settlement $s_c$ is the sum of the individual soil layer $s_{ci}$ values. This does not consider any other potential sources of settlement such as initial undrained shear induced, elastic, or drained creep (i.e., secondary compression).

For a soil layer that exists in a normally consolidated (NC) state prior to construction with an overconsolidation ratio (OCR):

$$OCR = \sigma'_p/\sigma'_{v0}$$

Eq. 2.2

equal to 1.0, then Eq. 2.1 reduces to

$$s_{ci} = C_c[H_0/(1 + e_0)] \log (\sigma'_{vf}/\sigma'_{v0})$$

for $\sigma'_p = \sigma'_{v0}$

Eq. 2.3

For a soil layer that exists in an overconsolidated (OC) state and remains OC at the end of construction (i.e., OCR > 1 prior to and after construction), Eq. 2.1 reduces to

$$s_{ci} = C_r[H_0/(1 + e_0)] \log (\sigma'_{vf}/\sigma'_{v0})$$

for $\sigma'_{vf} \leq \sigma'_p$

Eq. 2.4

Calculating the rate of primary consolidation settlement requires an estimate of the coefficient of consolidation ($c_v$) and calculating secondary consolidation settlement requires an estimate of the secondary compression index $C_\alpha (= \Delta e/\Delta \log t)$. Thus, the required soil parameters for conducting settlement predictions are: 1) state parameters: $e_0$, $\sigma'_{v0}$ and $\sigma'_p$ and 2) consolidation parameters: $C_r$, $C_c$, $c_v$, $C_\alpha$.

2.2.2 Influence of Sample Disturbance

Figure 2.1 illustrates the significant changes in one-dimensional compressibility and flow properties when a soft clay is loaded beyond the preconsolidation stress. This transition stress separates small, mostly elastic strains as defined by $C_r$ from large mostly plastic strains as defined by $C_c$. Furthermore, as the loading changes from recompression (OC) to virgin compression (NC), $c_v$ and $C_\alpha$ also undergo marked changes. For undisturbed
clay, \( c_v(OC) \) is typically 5 to 10 times the value of \( c_v(NC) \), which is mostly due to a lower coefficient of volume change \( (m_v = \Delta \varepsilon_v / \Delta \sigma'_v) \) in the OC region. The rate of secondary compression increases as \( \sigma'_v \) approaches \( \sigma'_p \) and often reaches a peak just beyond \( \sigma'_p \). Most of these one-dimensional consolidation properties are adversely influenced by sample disturbance, as also illustrated in Figure 2.1. Sample disturbance results in a more rounded compression curve with greater \( \varepsilon_v \) (or lower \( e \)) at all stress levels. The increased compressibility in the OC range (higher \( C_v \)) and decreased compressibility in the NC range (lower \( C_c \)) tends to obscure and usually lower \( \sigma'_p \). During recompression, \( c_v(OC) \) is usually much lower and \( C_v(OC) \) is higher. The only parameter not significantly affected by sample disturbance is \( c_v(NC) \) well beyond \( \sigma'_p \) and the \( e–\log k_v \) relationship, unless there is severe disturbance.

No definitive method exists for evaluating the quality of samples and it is especially difficult to distinguish disturbance caused by constrained swelling (due to sampling stress relief) versus that caused by shear distortions. The former should have minimal effect on consolidation properties whereas the latter can produce irreversible destructuring that alters basic behavior depending on the degree of damage to the soil structure (Ladd and DeGroot 2003). Nevertheless, some useful methods for assessing sample quality have been developed. X-rays of tube samples can provide useful visual information on variations in soil type, layering, presence of inclusions, and signs of disturbance including bending of soil layers near the tube perimeter, cracks due to stress relief, and voids due to gross disturbance. The most widely recognized quantitative method of evaluating sample quality for clays is the measurement of volume change \( (\varepsilon_{vol} = \varepsilon_v \text{ in 1-D consolidation}) \) during laboratory 1-D reconsolidation to the estimated in situ effective stress state (e.g., from IL
or CRS oedometer testing or from consolidation phase of anisotropically consolidated triaxial tests). Andresen and Kolstad (1979) proposed that increasing sample disturbance should result in increasing values of $\varepsilon_{\text{vol}}$ or $\varepsilon_v$. Terzaghi et al. (1996) adopted this approach, coined the term Specimen Quality Designation (SQD) with sample quality ranging from A (best) to E (worst), and suggested that reliable laboratory data required samples with SQD of B or better for clays with OCR $< 3 – 5$. Lunne et al. (2006) modified the method of Andresen and Kolstad (1979) with the use of the normalized change in void ratio $\Delta e/e_0$ during laboratory reconsolidation to the estimated in situ effective stress state and distinguished between clays of OCR = 1 to 2 versus 2 to 4. The sample quality criterion of Lunne et al. (2006) was developed for clay soils with a plasticity index in the range of 6% to 43%. More recently, Karlsrud and Hernandez-Martinez (2013) and DeJong et al. (2018) proposed methods that use the recompression and virgin compression behavior during 1-D consolidation testing to evaluate sample quality. It is not known if any of the sample quality evaluation methods listed above are applicable to high liquid limit, organic silts and clays.

2.3. Database Development

The database was created using data from laboratory soil classification and 1-D incremental loading (IL) consolidation tests performed on samples collected for selected past LA marsh creation projects. All projects were performed by GeoEngineers, Inc. (Baton Rouge, LA). Table 2.1 lists by project name/location these projects included in the database. All samples were distinguished as being either organic or inorganic based on the reported Unified Soil Classification System (USCS; ASTM D2487) with the designations as CL, CH, ML and MH or inorganic soils and OL, OH and Pt for organic soils. The focus
of this work was on soil properties and design parameters needed to estimate the consolidation settlement of LA marsh creation project foundation soils. These included: total unit weight ($\gamma_t$), specific gravity ($G_s$), preconsolidation stress ($\sigma'_p$), recompression index ($C_r$), compression index ($C_c$), coefficient of consolidation ($c_v$), and coefficient of secondary compression ($C_{\alpha}$). The basic index and classification properties that were studied in developing the recommended correlations included: water content ($w$), initial void ratio ($e_0$), Atterberg Limits (liquid limit, plastic limit, plasticity index), liquidity index, and dry unit weight ($\gamma_d$).

Ideally the database should have been filtered to separate out test results from the better-quality samples and remove those from the poor to very poor-quality samples. However, as noted in the background section, no quantitative method has been developed for evaluating sample quality for high liquid limit organic soils. An effort was made to mitigate, albeit to an unknown degree, the influence of sample disturbance by using: 1) an unload-reload loop to estimate $C_r$, 2) the simplified Schmertmann procedure to estimate $C_c$, and 3) considering $c_v$ not just at $\sigma'_v0$ but also at $\sigma'_p$ and well into the normally consolidated stress state at $5\sigma'_p$. In sum, the following procedures were used to estimate the desired parameters for each IL consolidation test:

- $\sigma'_v0$ was estimated using information from the boring logs and a combination of measured or appropriately estimated total unit weight values for each of the soil units identified in the boring logs
- $\sigma'_p$ was estimated using Casagrande (1936) construction from the 1-D consolidation stress-strain plots
- OCR was computed as the ratio of the above estimated $\sigma'_p$ and $\sigma'_v0$ values
- $C_r$ was estimated using the simplified Schmertmann (1955) construction procedure as shown schematically in Figure 2.2 with the unload-reload loop serving as a guide for selection of $C_r$.

- $C_c$ was estimated using a simplified Schmertmann (1955) construction using the above noted Casagrande estimate of $\sigma_p'$ as shown schematically in Figure 2.2. In all cases this method resulted in a value of $C_c$ that was either equal to or greater than the slope of the measured compression curve for stress levels beyond $\sigma_p'$.

- $c_v$ values were determined using the Taylor (1948) square root of time method for interpreting individual load increment displacement-time curves.

The construction shown in Figure 2.2 is termed the "simplified" Schmertmann (1955) method as it does not involve the full graphical construction procedure suggested by Schmertmann (1955) for adjusting laboratory compression curves to estimate the in situ consolidation behavior. As such, $\sigma_p'$ values were not adjusted for any possible influence of sample disturbance.

2.4. Correlations Investigated

Table 2.1 lists the number of oedometer tests used to create the database by USCS soil type as inorganic (CL, CH, ML and MH) and organic (OL, OH, and Pt). Figure 2.3 plots the Atterberg limits in a Casagrande plasticity chart. As is evident in Table 2.1 and Figure 2.3 the database contains results from tests conducted on a wide variety of soil types that span a large range in water content and liquid limit. In general, the degree of correlation
between the index parameters and design parameters investigated varied significantly; while in many cases there are distinct trends there is also often a large degree of scatter. For the strongest correlations, best fit regression equations are presented whereas for weaker correlations with significant scatter fitting regression equations to the data was not justified.

### 2.4.1 Preconsolidation Stress

The most important soil parameter for estimating consolidation settlement is the preconsolidation stress $\sigma'_p$ as it separates small (i.e., recompression) versus large (i.e., normally consolidated) deformation (Figure 2.1 and Eq. 2.1). However, as noted in the Introduction there are numerous factors that can influence how a soil deposit develops $\sigma'_p$ and generally no reliable universal correlations between $\sigma'_p$ and an index parameter exist. One common correlation presented in the literature is between $\sigma'_p$ and $LI$ as shown for example in Figure 2.4 from the USACE NAVFAC DM-7 (1982). But as shown in Figure 2.5, which plots the LA Marsh Creation database values, no useful correlation can be recommended from this dataset. Sensitivity values were not available for the LA Marsh Creation projects included in the database and it is unknown if the availability of such would reduce the scatter in Figure 2.5. Furthermore, many of the $\sigma'_p$ values are less than $\sigma'_v0$ as shown in Figure 2.6 which corresponds to a computed OCR of less than one (Figure 2.7). Generally, OCR values of less than one is not possible and one of the main reasons such values are obtained from laboratory test results is due to the detrimental effects of sample disturbance. In the absence of a reliable correlation for $\sigma'_p$, recommendations are given at the end of the paper on an hierarchy of procedures to consider for estimating $\sigma'_p$. 

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2.4.2 In Situ Vertical Effective Stress

Computation of the in situ vertical effective stress ($\sigma'_v$) requires estimates of the in situ equilibrium pore water pressure ($u_0$) and the total unit weight to compute the in situ vertical total stress ($\sigma_v$). In the absence of direct measurement of total unit weight from reasonable quality samples, it can be estimated using the specific gravity and assuming 100% saturation as

$$\gamma_i = (1 + w/100)\gamma_d = (1 + w/100)(G_s\gamma_w)/(1 + wG_s/100)$$

Eq. 2.5

where

$\gamma_i$ = total unit weight [pcf]

$\gamma_d$ = dry unit weight [pcf]

$G_s$ = specific gravity [-]

$w$ = natural water content [percent]

$\gamma_w$ = unit weight of water [pcf]

Measurement of the specific gravity $G_s$ is recommended although in the absence of such measurement normally a reasonable estimate of $G_s$ can be made knowing the basic soil type. However, for high liquid limit and organic clays $G_s$ values can be much lower than the range of conventional estimates. Figure 2.8 presents $G_s$ as a function of liquid limit and for which the trend lines suggest;

$G_s = 2.6$ to $2.7$ for $LL \leq 100\%$  

Eq. 2.6

$G_s = 2.86 - 0.002LL$ for $LL > 100\%$  

Eq. 2.7

Figure 2.9 presents the correlation between $G_s$ and water content for use if Atterberg Limits data are not available. The correlation is similar to that for LL but with somewhat more scatter.
Alternatively, the total unit weight can be directly estimated using water content as presented in Figure 2.10 for which the best fit Eq. plotted in Figure 2.10 gives

\[ \gamma_t = \frac{160.8 + 1.367w}{1 + 0.0230w} \]  

Eq. 2.8

where

\[ \gamma_t = \text{total unit weight [pcf]} \]

\[ w = \text{water content [percent]} \]

Eq. 2.8 is a form of Eq. 2.5 but embedded within the best fit regression Eq. of \( \gamma_t \) as a function of \( w \) is a variable \( G_s \) as a function of water content.

For most, but not all sites, it is reasonable to assume that the preconstruction in situ equilibrium pore water pressure (\( u_0 \)) is hydrostatic. However, regions that have undergone recent deposition, submarine slides, or other mechanisms may have pore pressures different than hydrostatic. In cases where excess pore pressures are suspected the correlations recommended herein should be used with caution.

2.4.3 Compression Index

Figure 2.11 presents the correlation for the compression index \( C_c = \Delta e/\Delta \log \sigma_v \) for the normally consolidated stress range. The plot also includes several of the more common published correlations for reference and also the data points (open symbols) presented in Terzaghi et al. (1996) which is the most comprehensive data set available in the literature. Terzaghi et al. (1996) separate clays and silts versus peats. The data from this work's database generally follow the trends in the Terzaghi et al. (1996) dataset with an inflection in the trends between clays and silts versus peats – at about 100% water content. Although for less than 100% water content, the data consistently plot below that of Terzaghi et al.
(1996), which could be in some cases lower sample quality resulting in lower $C_c$ values
and also that a number of the data points in the 50 to 100% water content range in the
Terzaghi et al. (1996) database are for highly structured sensitive clays which have very
large $C_c$ values.

Overall a best fit Eq. for the full data set presented in Figure 2.11 does not reflect
the data well at lower water contents. Therefore, separate correlations for $C_c$ as a function
of water content are recommended and presented in Figure 2.12 and Figure 2.13 for
inorganic and organic soils such that

\[ C_c = 0.015w - 0.16 \quad \text{for inorganic CL, CH, ML and MH soils} \text{ Eq. 2.9} \]

and

\[ C_c = 0.010w \quad \text{for organic OL, OH, and Pt soils} \text{ Eq. 2.10} \]

For reference Mesri and Ajlouni (2007) recommend $C_c = 0.010w$ for fibrous peats
which is the same as Eq. 2.10.

Figure 2.14 presents the correlation for $C_c$ as a function of the initial void ratio $e_0$
along with a number of correlations presented in the literature. The best fit regression to
the full dataset gives a relationship nearly identical to that of Park and Lee (2011).
However, this Eq. overpredicts $C_c$ for low void ratio soils and therefore separate
correlations are recommended for inorganic (Figure 2.15) and inorganic soils (Figure 2.16)
resulting in

\[ C_c = 0.57e_0 - 0.20 \quad \text{for inorganic CL, CH, ML and MH soils} \text{ Eq. 2.11} \]

and

\[ C_c = 0.50e_0 \quad \text{for organic OL, OH, and Pt soils} \text{ Eq. 2.12} \]
Use of Eqs. 2.11 and 2.12 requires measurements to compute $e_0$ or in the absence of such measurements it can be estimated $w$ and $G_s$ for an assumed condition of 100% saturation as

$$e_0 = G_s w$$

Eq. 2.13

and if $G_s$ is not measured it can be estimated using Eqs. 2.6 and 2.7.

Plots of $C_c$ versus liquid limit and $C_c$ versus plasticity index show positive trends but the scatter is somewhat greater than that of the water content correlations (Figure 2.12 and Figure 2.13) and are not recommended.

If a reliable measure of the dry unit weight is available there is a strong correlation between $C_c$ and $\gamma_d$ that includes both inorganic and organic soils as shown in Figure 2.17. Best-fit regression fit to the data gives

$$C_c = 7.15e^{-0.037 \gamma_d}$$

for inorganic and organic soils

Eq. 2.14

2.4.4 Recompression Index

The recompression index $C_r = \Delta e/\Delta \log \sigma'_v$ is for recompression loading from the in situ vertical effective stress ($\sigma'_{vo}$) to a stress equal to or less than the preconsolidation stress ($\sigma'_p$). Correlations between $C_r$ and $w$, $e_0$, $LL$ or $PL$ show positive trends, but all have significant scatter and are not recommended. Rather it is common in practice to correlate $C_c$ with $C_r$ as plotted in Figure 2.18 for which $C_r = 0.10 C_c$ is within the range of 0.02 to 0.20 reported for most soils (Terzaghi et al., 1996). Figure 2.19 and Figure 2.20 present separate plots for inorganic and organic soils and provide some refinement resulting in

$$C_r = 0.13 C_c$$

for inorganic CL, CH, ML and MH soils

Eq. 2.15

and
2.4.5 Coefficient of Consolidation

The coefficient of consolidation is highly dependent on the stress state relative to the preconsolidation stress as shown schematically in Figure 2.1, Figure 2.21, Figure 2.22, and Figure 2.23 plots versus $c_v$ at $\sigma'_v \approx \sigma'_{v0}$, $\sigma'_v \approx \sigma'_p$, and $\sigma'_v \approx 5\sigma'_p$. The $c_v$ at $\sigma'_v \approx \sigma'_{v0}$ values in Figure 2.21 are from oedometer test for which the test specimen OCR was greater than approximately 1.5. Terzaghi et al. (1996) note the recompression values of $c_v$ (i.e., loading from $\sigma'_{v0}$ towards $\sigma'_p$) can be from one to as much as one hundred times the normally consolidated value although for most soft clays the ratio typically ranges from around 5 to 10. Furthermore, for soft clays and silts $c_v$ is more or less constant in the compression range from $\sigma'_p$ to $5\sigma'_p$ but can decrease by a factor of 10 to 20 for fibrous peats. The Figure 2.21, Figure 2.22, and Figure 2.23 data have significant scatter but depict several trends: 1) $c_v$ values for $\sigma'_v \approx \sigma'_{v0}$ are somewhat more scattered than that for $c_v$ at $\sigma'_v \approx 5\sigma'_p$ which is expected as there is generally more uncertainly in interpretation of $c_v$ data for recompression loading due to sample disturbance; 2) $c_v$ values for $\sigma'_v \approx 5\sigma'_p$ at a given liquid limit are lower than for $c_v$ at $\sigma'_v \approx \sigma'_{v0}$ which is expected for NC vs OC behavior; and 3) $c_v$ initially decreases with an increase in liquid limit and then transitions to increasing with an increase in liquid limit – this is especially evident in Figure 2.23 for $c_v$ at $\sigma'_v \approx 5\sigma'_p$. Figure 2.24 and Figure 2.25 present $c_v$ at $\sigma'_v \approx 5\sigma'_p$ for inorganic and organic soils. Also included in Figure 2.24 is the NAVFAC DM7 (1982) correlation for normally consolidated $c_v$ and data points from Terzaghi et al. (1996) for normally consolidated $c_v$.  

$C_r = 0.10C_c$ for organic OL, OH, and Pt soils Eq. 2.16
Overall, the scatter in the various $c_v$ versus liquid limit plots is large and a pragmatic approach is proposed here on recommendation for use of the correlations in practice. Given the generally greater reliability of estimates of $c_v$ at $\sigma_v' \approx 5 \sigma_p'$ it is recommended that values of $c_v$ be first estimated based on liquid limit values for this stress state using the recommended ranges plotted in Figure 2.24 and Figure 2.25 as a guide

$$c_v(\sigma_v' \approx 5 \sigma_p') = (50 \text{ to } 250)(L L)^{-1.9}$$  
for inorganic soils  
 Eq. 2.17

$$c_v(\sigma_v' \approx 5 \sigma_p') = (0.002 \text{ to } 0.009)e^{0.007 LL}$$  
for organic soils  
 Eq. 2.18

where

$e = \text{exponential function}$

Values of $c_v$ for states of stress around $\sigma_v' \approx \sigma_v$ and $\sigma_p'$ can be estimated using approximate ratios based upon the data plotted in Figure 2.21, Figure 2.22, and Figure 2.23

Inorganic:

$$c_v(\sigma_v' \approx 5 \sigma_p') / c_v(\sigma_v' \approx 5 \sigma_p') \approx 5$$  
for inorganic soils  
 Eq. 2.19

$$c_v(\sigma_v' \approx 5 \sigma_p') / c_v(\sigma_v' \approx 5 \sigma_p') \approx 3$$  
for inorganic soils  
 Eq. 2.20

Organic:

$$c_v(\sigma_v' \approx 5 \sigma_p') / c_v(\sigma_v' \approx 5 \sigma_p') \approx 6$$  
for organic soils  
 Eq. 2.21

$$c_v(\sigma_v' \approx 5 \sigma_p') / c_v(\sigma_v' \approx 5 \sigma_p') \approx 5$$  
for organic soils  
 Eq. 2.22

The Eqs. 2.17 to 2.22 are an approximate guideline as there is overall a significant degree of scatter in the dataset that they are based upon.
2.4.6 Coefficient of Secondary Compression

The coefficient of secondary compression $C_\alpha = \Delta e/\Delta \log t$ is best evaluated from incremental loading (IL) consolidation tests. In the absence of such measurements the Terzaghi et al. (1996) correlations between $C_\alpha$ and $C_c$ are recommended

- $C_\alpha = (0.04 \pm 0.01)C_c$ for inorganic clays and silts \hspace{1cm} \text{Eq. 2.23} \\
- $C_\alpha = (0.05 \pm 0.01)C_c$ for organic clays and silts \hspace{1cm} \text{Eq. 2.24} \\
- $C_\alpha = (0.06 \pm 0.01)C_c$ for peat and muskeg \hspace{1cm} \text{Eq. 2.25}

These correlations are independent of stress level and hold for both recompression and normally consolidated stress states.

2.5. Recommendation for use of Correlations in Practice

Table 2.2 presents a summary of the correlations investigated and includes an assessment of the efficacy of each correlation. Use of the correlations should only be used for feasibility studies, preliminary analyses and for comparative purposes as data are being collected on new projects. Final design should always involve an integrated site characterization program that includes in situ testing and advanced laboratory testing (i.e., consolidation) of undisturbed samples. It is also important to note that while most of the correlations have clear trends there is also often significant scatter adding a further cautionary note to their use. Users of the correlations are encouraged to always inspect the scatter associated with any estimate as opposed to just using the best-fit equations. Given project considerations, appropriate decisions can be made as to whether to select values along the best-fit equations for the correlations or to use a range of values.
The most important parameter for estimating consolidation settlement is $\sigma'_p$ but unfortunately no useful empirical correlations exist between $\sigma'_p$ and any index parameter. As such the options, in the absence of collection of any additional site investigation data, are limited. One approach is to assume an OCR of one (or other appropriate values) based on knowledge of the site geology and past experience. For feasibility studies and preliminary design assuming normally consolidated (OCR = 1) conditions in practice is generally considered a conservative approach. However, this may not necessarily be the case for marsh creation projects where settlement predictions typically need to be within a particular range (i.e., not too large or too small) to make the marsh creation successful.

Beyond using only empirical correlations, conducting in situ tests such as the piezocone (CPTU) and field vane test (FVT) can provide much valuable additional information. The CPTU is more versatile than the FVT and well conducted CPTU tests with reliable pore pressure measurements (typically in the $u_2$ position, which is located on the shoulder behind the cone tip) can provide detailed information for determination of soil units and for estimating (e.g., Lunne et al. 1997, Mayne 2007, Robertson 2009), soil behavior type, $\gamma_t$, $\sigma'_p$ and $c_v$ (if dissipation tests are conducted). The CPTU is generally not reliable for estimating the consolidation compressibility parameters $C_c$ and $C_r$. CPTU data can be used to estimate $\sigma'_p$ for which the commonly used universal Eq. is (Lunne et al. 1997, Mayne 2007)

$$\sigma'_p = k(q_t - \sigma_{v0}) = 0.33(q_t - \sigma_{v0})$$

Eq. 2.26

where

$q_t =$ corrected tip resistance

$\sigma_{v0} =$ in situ total vertical stress
\[ k = \text{CPTU correlation coefficient for } \sigma'_p \]

Alternatively, Mesri (2001) proposed different \( k \) values for inorganic versus organic clays and silts to estimate the end of primary consolidation value of \( \sigma'_p \)

\[
\sigma'_p = 0.28(q_t - \sigma_{v0}) \quad \text{for inorganic clays and silts} \tag{Eq. 2.27}
\]

\[
\sigma'_p = 0.24(q_t - \sigma_{v0}) \quad \text{for organic clays and silts} \tag{Eq. 2.28}
\]

The primary purpose of conducting FV tests is to measure the in situ undrained shear strength. However, FVT data can also be used to estimate \( \sigma'_p \) for inorganic clays as (Chandler 1988, Ladd and DeGroot 2003)

\[
\sigma'_p = \sigma'_{v0}[\frac{s_u(FV)}{\sigma'_{v0}}/S_{FV}]^{1.05} \tag{Eq. 2.29}
\]

where

\[ s_u(FV) = \text{measured field vane undrained shear strength (not corrected)} \]

\[ \sigma'_{v0} = \text{in situ vertical effective stress at depth of } s_u(FV) \]

\[ S_{FV} = \text{coefficient estimated using Figure 2.26} \]

The Eqs. 2.26 to 2.29 correlation coefficients are considered universal values and ideally specific regional or specific soil type correlations should be developed from past projects that involved CPTU testing and parallel laboratory 1-D consolidation tests performed on good quality undisturbed samples. Certainly, any estimates of \( \sigma'_p \) from that result in computed OCRs of less than one should be considered unreliable (presuming a reliable estimate of the in situ equilibrium pore pressure \( u_0 \) is made).

**2.6. Summary and Conclusions**

This paper presents a collection of empirical correlations for estimating consolidation parameters of fine-grained soils for settlement calculations using basic index
and classification measurements. The database was generated using site investigation data from 15 marsh creation projects in the coastal Louisiana region. These projects often involve low to near normally consolidated soft, high liquid limit organic silts and clays, for which a majority of published empirical correlations do not cover. The collective data set included a wide variety of soil types that span a large range in water content and liquid limit. Index and classification properties included in the database were water content, void ratio, Atterberg Limits, and dry unit weight. Consolidation design parameters for the 15 projects were determined from 1-D incremental loading (IL) consolidation test results and included recompression ratio ($C_r$), compression ratio ($C_c$), preconsolidation stress ($\sigma'_p$), and coefficient of consolidation ($c_v$). Sample quality was a concern and the IL test results were interpreted using an unload-reload loop for $C_r$ and simplified Schmertmann's construction for $C_c$. $C_v$ values were evaluated relative to the normally consolidated stress state. For all the correlations investigated, the degree of correlation between the index parameters and design parameters varied significantly (Table 2.2); while in many cases there are distinct trends there is also often a large degree of scatter. The strongest correlation was for $C_c$ as function of $w$ considering inorganic and organic soil separately. No useful correlation was found for $\sigma'_p$, which is the most important parameter for settlement calculations. This requires estimating the in situ stress history based on geology and local experience or conducting additional site investigation testing such as in situ testing. Use of the correlations presented herein should only be used for feasibility studies, preliminary analyses and for comparative purposes as data are being collected on new projects. Final design should always involve an integrated site characterization program
that includes in situ testing and advanced laboratory testing (i.e., consolidation) of good quality undisturbed samples.

2.7. Acknowledgments

The author thanks GeoEngineers, Baton Rouge, LA for access to data for the marsh creation projects and in particular David S. Eley, P.E., GeoEngineers, for coordinating ideas and feedback from GeoEngineers on development of the database and the correlations studied.
Table 2.1: List of projects/locations included in creation of database and number of oedometer tests for each by soil type

<table>
<thead>
<tr>
<th>Project Name/Location</th>
<th>Inorganic CL, CH, ML, MH</th>
<th>Organic OL, OH, Pt</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alligator Bend</td>
<td>15</td>
<td>9</td>
<td>24</td>
</tr>
<tr>
<td>Lake Lery</td>
<td>24</td>
<td>10</td>
<td>34</td>
</tr>
<tr>
<td>Cameron Cereole</td>
<td>28</td>
<td>1</td>
<td>29</td>
</tr>
<tr>
<td>Terrebonne Bay</td>
<td>8</td>
<td>6</td>
<td>14</td>
</tr>
<tr>
<td>Whiskey Island</td>
<td>10</td>
<td>0</td>
<td>10</td>
</tr>
<tr>
<td>Shark Island</td>
<td>2</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>West Bayou Perot</td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>LaBranche</td>
<td>7</td>
<td>17</td>
<td>24</td>
</tr>
<tr>
<td>Plaquemines Parish</td>
<td>38</td>
<td>11</td>
<td>49</td>
</tr>
<tr>
<td>Bayou Sale</td>
<td>29</td>
<td>1</td>
<td>30</td>
</tr>
<tr>
<td>Lost Lake</td>
<td>5</td>
<td>4</td>
<td>9</td>
</tr>
<tr>
<td>Bayou Bonfuca</td>
<td>3</td>
<td>6</td>
<td>9</td>
</tr>
<tr>
<td>Turtle Bay</td>
<td>4</td>
<td>3</td>
<td>7</td>
</tr>
<tr>
<td>Caminada</td>
<td>8</td>
<td>0</td>
<td>8</td>
</tr>
<tr>
<td>Grand Liard</td>
<td>4</td>
<td>6</td>
<td>10</td>
</tr>
<tr>
<td>Totals</td>
<td>186</td>
<td>78</td>
<td>264</td>
</tr>
<tr>
<td>Soil Property</td>
<td>Index parameter</td>
<td>Comments</td>
<td></td>
</tr>
<tr>
<td>---------------</td>
<td>-----------------</td>
<td>----------</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$G_s$</td>
<td>VG</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\gamma$</td>
<td>E</td>
<td>correlation includes $G_s$</td>
<td></td>
</tr>
<tr>
<td>$\sigma'_p$</td>
<td></td>
<td>VP</td>
<td>no reliable correlation exists for $\sigma'_p$</td>
</tr>
<tr>
<td>$C_e$</td>
<td>VG VG G G</td>
<td>separate correlations for inorganic vs organic soils; universal correlation with $\gamma_d$</td>
<td></td>
</tr>
<tr>
<td>$C_r$</td>
<td>P P P P</td>
<td>best option is to correlate with $C_r$ and separate inorganic vs organic soils</td>
<td></td>
</tr>
<tr>
<td>$c_v$</td>
<td>P P P P</td>
<td>significant scatter; separate correlations for inorganic vs organic and by stress level; $\sigma'_v \approx \sigma'_p$ for recompression; $\sigma'_v \approx 5\sigma'_p$ for normally consolidated compression</td>
<td></td>
</tr>
<tr>
<td>$C_\alpha$</td>
<td></td>
<td>best option is to directly correlate with $C_c$</td>
<td></td>
</tr>
</tbody>
</table>

Notes: E = excellent, VG = very good, G = good, P = poor, VP = very poor
Figure 2.1: Fundamentals of 1-D consolidation behavior and influence of sample disturbance: compression curve, coefficient of consolidation and secondary compression versus vertical effective stress (modified from Ladd and DeGroot, 2003)
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Figure 2.7: Preconsolidation ratio versus depth for all consolidation test results included in the LA Marsh Creation database
Figure 2.8: Specific gravity versus liquid limit and recommended correlation (Eqs. 2.6 and 2.7)

\[ G = 2.6 \text{ to } 2.7 \text{ for } LL < 100\% \]
\[ G = 2.86 - 0.002LL \text{ for } LL > 100\% \]
Figure 2.9: Specific gravity versus water content
Figure 2.10: Total unit weight versus water content and recommended correlation (Eq. 2.8)

\[ \gamma_t = \frac{160.8 + 1.367w}{1 + 0.0230w} \]
Figure 2.11: Compression index versus water content with some common published correlations
Figure 2.12: Compression index versus water content with recommended correlation for inorganic CL, CH, ML and MH soils (Eq. 2.9).
Figure 2.13: Compression Index versus water content with recommended correlation for organic OL, OH, and Pt soils (Eq. 2.10)

\[ C_c = 0.010w \]
Figure 2.14: Compression Index versus initial void ratio with some common published correlations
Figure 2.15: Compression Index versus initial void ratio with recommended correlation for inorganic CL, CH, ML and MH soils (Eq. 2.11)

$C_c = 0.57e_0 - 0.20$
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Figure 2.17: Compression Index versus initial void ratio with recommended correlation for organic OL, OH, and Pt soils (Eq. 2.14)

$C_c = 7.15e^{-0.037\gamma_d}$
Figure 2.18: Recompression Index versus Compression Index with recommended correlation for all soils

\[ C_r = 0.10C_c \]
Figure 2.19: Recompression Index versus Compression Index with recommended correlation for inorganic CL, CH, ML and MH soils (Eq. 2.15)

$C_r = 0.13C_c$
Recalculate the Compression Index and Recompression Index for organic OL, OH, and Pt soils (Eq. 2.16)
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Figure 2.24: Coefficient of Consolidation at $\sigma'_{v} \approx 5\sigma'_{p}$ versus Liquid Limit for inorganic CL, CH, ML and MH soils.

- DM7.01 for NC soils
- Recommended Range
- Terzaghi et al. (1996)

Legend:
- Alligator Bend
- Lake Lery
- Cameron Cereole
- Terrebonne Bay
- Whiskey Island
- Shark Island
- West Bayou Perot
- LaBranche
- Plaquemines Parish
- Bayou Sale
- Lost Lake
- Bayou Bonfuca
- Turtle Bay
- Caminada
- Grand Liard
- Terrebonne Bay
- Plaquemines Parish
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CHAPTER 3

NORMALIZED UNDRAINED DIRECT SIMPLE SHEAR BEHAVIOR OF SOFT COASTAL ORGANIC SOILS

The SHANSEP procedure for estimating the in situ undrained shear behavior of fine-grained soils assumes perfectly normalized undrained shear behavior. However, recent findings indicate that such is not the case for the triaxial compression behavior of resedimented inorganic soils with liquid limits \((LL)\) less than 100%. This paper presents results from a laboratory investigation of the influence of vertical consolidation stress \((\sigma'_{vc})\) on the undrained direct simple shear (DSS) behavior of six resedimented natural organic soils with \(LL\) ranging from 81 to 215% and two inorganic soils with \(LL\) equal to 45 and 46%. DSS tests were conducted on normally consolidated specimens with \(\sigma'_{vc}\) ranging from 50 to 1600 kPa. The results show that the organic soils have much higher \(s_u/\sigma'_{vc}\) and lower normalized undrained Young's modulus \((E_u/\sigma'_{vc})\) than the inorganic soils. Furthermore, the decrease in \(s_u/\sigma'_{vc}\) with an increase in \(\sigma'_{vc}\) is much greater than that of the lower \(LL\) inorganic soils. Correlations are presented for the organic soils that relate normally consolidated \(s_u/\sigma'_{vc}\) and \(\sigma'_{vc}\) for DSS mode of shear as a function of \(LL\) or organic matter. These equations can be used to estimated \(s_u\) at \(\sigma'_{vc}\) values not included in a laboratory test program.

3.1. Introduction

Various factors, including subsidence caused by oil extraction and sea-level rise due to climate change, have resulted in significant land loss in coastal Louisiana (Walker
et al., 1987) and other regions in the Gulf of Mexico. Marsh creation projects, conducted by construction of confined areas with dykes and placement of dredged material from seabed or waterways in these areas, are being used to restore lost land. The soft, organic and often near normally consolidated nature of soils in the area are of concern for stability and undrained shear deformation of the levees with the key design parameters being undrained shear strength ($s_u$) and undrained Young's modulus ($E_u$). For such stability problems undrained shear strength anisotropy needs to be considered as it varies with the orientation of the major principal stress at failure as shown for example in Figure 3.1. More advanced design approaches consider the various modes of shear shown in Figure 3.1, which include triaxial compression (TC), direct simple shear (DSS) and triaxial extension (TE), in anisotropic undrained stability analyses. Although such an approach requires a relatively extensive laboratory test program and a pragmatic alternative is to focus only on DSS testing in stability analyses as the peak shear stress measured in the DSS has proved to provide a reliable estimate of the average or mobilized undrained shear strength ($s_u(\text{ave})$ or $s_u(\text{mob})$) for isotropic stability analyses (e.g., Ladd 1991, Ladd and DeGroot 1991). Likewise, $E_u$ determined from DSS tests has also proved useful for estimating the undrained shear deformation of embankments or dikes constructed on soft soils (Ladd et al. 1977).

Organic fine-grained soils are often very soft and compressible although they typically have a higher normalized undrained shear strength ($s_u/\sigma_{v0}'$), where $\sigma_{v0}' = \text{in situ vertical effective stress}$, than inorganic soils (e.g., Mesri 1993, Terzaghi et al. 1996). However, collecting undisturbed samples of soft, organic coastal soils can be a challenge and laboratory test programs need to consider the potential effects of testing disturbed
samples. The Stress History and Normalized Soil Engineering Properties (SHANSEP) method was developed by Ladd and Fout (1974) as a practical framework for dealing with the influence of sample disturbance for fine-grained soils that have low to moderate structure. The foundation of the method is the assumption of perfectly normalized behavior, i.e., $s_u/\sigma'_vc$ is a constant independent of $\sigma'_vc$, where $\sigma'_vc =$ vertical consolidation stress. However, the recent work by Casey and Germaine (2013) showed that such was not the case for a collection of inorganic fine-grained soils with a liquid limit ranging from around 40 to 75% for which $s_u/\sigma'_vc$ decreased with an increase in $\sigma'_vc$ over the stress range 100 kPa to 10 MPa. These findings have practical implications for use of the SHANSEP method for design applications. It is unknown if such behavior is also the case for higher liquid limit (i.e., $LL > 100\%$), organic fine-grained soils.

This paper presents the results from a suite of DSS tests conducted on resedimented samples of six organic soils collected from coastal areas of Louisiana and two inorganic marine soils collected from Massachusetts and Maine. The tests were conducted over a range of stresses varying from 100 to 1600 kPa. The objective of the research was to determine how $s_u$ and $E_u$ of high liquid limit, organic soils vary as a function of vertical consolidation stress level. The laboratory test program focused on testing the soil at a normally consolidated state of stress as that is a key component of the SHANSEP method and furthermore many soils in coastal regions such as Louisiana are normally or near normally consolidated.
3.2. Background

3.2.1 Undrained Shear Strength

It is common in practice to analyze undrained shear strength data obtained from triaxial or DSS tests on clays normalized by the in situ vertical effective stress. The SHANSEP procedure is based on the experimental observation that clays at different consolidation stresses but at the same overconsolidation ratio (OCR) exhibit comparable normalized undrained shear behavior for both normally consolidated (NC) and overconsolidated (OC) conditions. The variation of the normalized undrained shear strength with OCR is expressed as the SHANSEP equation:

\[ \frac{s_u}{\sigma'_{vc}} = S(OCR)^m \]  

Eq. 3.1

where \( S \) is undrained strength ratio \( s_u/\sigma'_{vc} \) for NC state and \( m \) is a power coefficient that expresses the increase in \( s_u/\sigma'_{vc} \) with an increase in OCR. As described in Ladd (1991) and Ladd and DeGroot (2003), application of the SHANSEP method requires specimens to be \( K_0 \)-consolidated to a \( \sigma'_{vc} \) greater than the preconsolidation stress (\( \sigma'_p \)) into the normally consolidated stress range to measure the value of \( S \). Specimens consolidated in a similar manner but rebounded to various OCRs as needed are used to estimate the \( m \) parameter. Of the two coefficients in Eq. 3.1 the \( S \) parameter is the most important. Alternatively, the Recompression method (Bjerrum 1973, Ladd and DeGroot 2003) advocates dealing with sample disturbance by anisotropically consolidating specimens to the in situ effective stress state \( \sigma'_{v0} \) and \( \sigma'_{h0} \) where \( \sigma'_{h0} \) is the estimated in situ horizontal effective stress. While the Recompression method is philosophically very different than the SHANSEP procedure in dealing with sample disturbance, both methods otherwise advocate the same approach to laboratory testing in terms of evaluating anisotropy and rate.
effects. Results from Recompression tests can also be interpreted using Eq. 3.1 provided an estimate of the in situ OCR profile is available.

Some recent work indicates that the SHANSEP $S$ parameter is not constant with varying $\sigma'_{vc}$ and that perfectly normalized behavior may only be relevant over a relatively narrow range of stresses. Jones (2010) performed a suite of $CK_0UC$ tests on resedimented NC Ugnu Clay and reported that $S$ decreases as stress level increases. Abdulhadi et al. (2012) conducted $CK_0UC$ tests on NC and OC resedimented Boston Blue Clay (RBBC) and reported that $S$ decreased from 0.32 to 0.28 as consolidation stress increased from 0.15 MPa to 10 MPa. However, the authors also reported that $m$ remained essentially constant over the range of stresses investigated. Casey and Germaine (2013) investigated results of $CK_0UC$ tests on eight resedimented soils (liquid limit from 26% to 74%) and confirmed Abdulhadi et al. (2012)’s conclusion that $S$ decreases with increased consolidation stress level while variations in $m$ were insignificant. The one exception to this was the low liquid limit, non-plastic Skibbereen silt for which $s_u/\sigma'_{vc}$ increased with an increase in $\sigma'_{vc}$ (Gennan 2010). Based on a limited set of results from DSS tests on RBBC and Skibbereen silt, they also reported that $S$ did not change with consolidation stress level for RBBC but followed the same trend as TC for Skibbereen silt. To account for the influence of $\sigma'_{vc}$, Casey and Germaine (2013) introduced a modified SHANSEP equation as:

$$\frac{s_u}{\sigma'_{vc}} = S_1(1000\sigma'_{p})^T(OCR)^m$$ \text{ Eq. 3.2}

where $\sigma'_{p}$ is in MPa and coefficients $S_1$ can be estimated from Figure 3.2 or Eq. 3.3, and $T$ from Eq. 3.4.

$$S_1 = 0.0091 \ LL - 0.05$$ \text{ Eq. 3.3}

$$T = -0.48 \log_{10}(LL) + 0.77$$ \text{ Eq. 3.4}
3.2.2 Undrained Young’s Modulus

$E_u$ is commonly estimated from empirical data in the form of $E_u/s_u$ as it is challenging to obtain reliable measurement from routine lab tests, especially at small deformations. Ladd et al. (1977) conducted CK$_0$UDSS tests on seven NC organic and inorganic soils with liquid limit above 35%. They reported that $E_u/s_u$ decreased as plasticity and organic matter increased. Santagata et al. (2005) and Abdulhadi et al. (2012) studied the effects of stress level on $E_u$ by performing a series of CK$_0$UC tests on RBBC samples and concluded that $E_u$ increases with increased consolidation stress at different OCRs. Abdulhadi et al. (2012) also reported that $E_u/\sigma'_{vc}$ is stress dependent and decreases with increasing stress level. Casey et al. (2015) investigated the results from CK$_0$UC tests on eight fine-grained soils (liquid limit ranging from 26% to 79%) for stresses in the range of 0.1 to 100 MPa. They confirmed the abovementioned findings and indicated that OC soils have higher $E_u$, but the value of OCR does not have any tangible effects on $E_u$ measurements. They introduced Eqs. 3.5, 3.6, and 3.7 to estimate $E_u$ from $\sigma'_{vc}$ at applied shear stress ratios of 0.25, 0.50 and 0.75, where the shear stress ratio is defined as $(\tau - \tau_0)/(s_u - \tau_0)$ where $\tau$ is the current maximum shear stress in the specimen and $\tau_0$ is the maximum shear stress in the specimen prior to undrained shear.

\[
\frac{E_{u,0.25}}{\sigma'_{vref}} = 465(\frac{\sigma'_{vc}}{\sigma'_{vref}})^{0.73} \quad \text{Eq. 3.5}
\]

\[
\frac{E_{u,0.50}}{\sigma'_{vref}} = 364(\frac{\sigma'_{vc}}{\sigma'_{vref}})^{0.68} \quad \text{Eq. 3.6}
\]

\[
\frac{E_{u,0.75}}{\sigma'_{vref}} = 260(\frac{\sigma'_{vc}}{\sigma'_{vref}})^{0.61} \quad \text{Eq. 3.7}
\]

where $\sigma'_{vref}$ is taken as 100 kpa.
The variations in $s_u$ and $E_u$ with consolidation stress level described above were primarily based on TC tests conducted on inorganic clays and silts with liquid limits ranging from around 25 to 80%. No such similar results are available for the undrained DSS behavior of organic, high liquid limit (LL > 100%) fine-grained soils.

3.3. Test Results and Procedures

3.3.1 Test Soils

Table 3.1 provides a summary of the index properties and classification according to the United Soil Classification System (USCS, ASTM D2487, 2016) of the eight soils tested in this study. The first six are very soft marine organic soil samples collected from Plaquemines Parish, LA and Jefferson Parish, LA. All the soils are natural, except for soil #5 where 10% Pure Gold Gel, an inorganic commercial bentonite that is mostly Na-montmorillonite was added to the natural soil to increase its liquid limit. The other two soils are inorganic Boston Blue Clay (BBC) and Presumpscot Clay collected from Massachusetts and Maine, respectively. These two inorganic soils were included in the test program to evaluate the test procedures and findings of this work relative to that of Casey and Germaine (2013) as they also tested these two soils. Overall, the data set includes soils with a wide range of liquid limit and organic matter.

Liquid limit tests were conducted using both Casagrande cup (ASTM D4318) and Fall cone (ISO/TS 17892-12) methods. As expected based on the literature (e.g., Sridharan and Prakash 1998) the values are similar for the low liquid limit soils (BBC, Presumpscot) whereas the Casagrande cup results in higher to much higher values for the higher liquid
limit, organic soils. Soil classification was conducted based on the liquid limit from Casagrande cup and the classification as an organic soil was performed in accordance with the USCS through measurement of the liquid limit for both natural and oven-dried samples. Figure 3.3 presents the plasticity chart for the soils in the data set. In addition, organic matter tests were conducted on all soils in accordance with ASTM D6528 by burning the oven-dried specimen in a muffle furnace at 440°C.

### 3.3.2 Sample Preparation

It was not feasible to obtain high quality, undisturbed samples for this test program even though that would be the ideal situation. In addition, for this research, it was necessary to eliminate from the test samples potential variations in soil structure due to soil layering, sample disturbance, different storage conditions, etc. Thus, reconstituting batches of the test soils was considered necessary to test soil specimens in as near as possible identical conditions, so only the effects of different consolidation stresses could be studied. Large diameter block samples could possibly be used for testing of several samples at the same depth, however, block sampling is more complicated and expensive, and there is always the possibility of layering within a block (especially in organic soils). As a result, resedimented soil samples of various $LL$, permeability, and compressibility were used for this research.

For all the organic soils, bag samples and extruded Shelby tube samples obtained from several LA marsh creation projects were thoroughly mixed into soil batches of various liquid limits. In some cases, short fibers were visible in the samples and were removed. The samples were resedimented using the general procedure described by Lukas et al. (2018) to form a soil cake from which test specimens were trimmed. This was initially
done by mixing the soils with distilled water at 1.25-2 times the liquid limit and allowing the slurry to hydrate overnight. The slurry was then thoroughly mixed under vacuum after being placed in a 102 mm ID acrylic consolidometer to remove trapped air. Later, the soil was incrementally 1-D consolidated to the desired stress level using a computer controlled GeoJac system. Each load increment was applied until the End of Primary (EOP) was achieved. Given the limitation of maximum vertical consolidation strain obtainable in the DSS device, soil cakes for the DSS tests planned for higher consolidation stresses (i.e., 400 to 1600 kPa) were consolidated to higher stresses in the consolidometer. At the end of consolidation, the soil cake was extruded and either trimmed immediately for testing or coated by a layer of a 50-50 mixture of petroleum jelly and paraffin wax and at least 2 layers of plastic film dipped in the same mixture (La Rochelle et al. 1981) before being stored in a humid room with a controlled temperature of 11°C and >85% relative humidity for future testing.

3.3.3 Direct Simple Shear

The Direct Simple Shear (DSS) tests were conducted using a Geonor DSS device in general accordance to the procedures described by Bjerrum and Landva (1966), DeGroot et al. (1992) and ASTM D6528 Standard Test Method for Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils. The Geonor DSS device consists of a specimen chamber, lever arm for application of consolidation weights and a gear driven thrust shaft for applying the horizontal shear stress to the specimen. Load cells and linear variable differential transformers were connected to a dedicated data acquisition system and used for measurement of load and displacement. Specimens were prepared for testing by trimming the 35 cm² soil into a set of thin stainless-steel stacked rings with a 0.3 mm thick
internal membrane to a nominal target height of 19 mm. The stacked ring-membrane system allowed for one-dimensional consolidation during the consolidation phase of a test and direct simple shear strain mode of deformation during the shear phase of a test. Top and bottom drainage stones with a waffle pattern were used to increase the contact surface between the specimen and the stones to minimize the possibility of slippage at the soil-stone interface. The final vertical effective consolidation stresses ($\sigma'_{vc}$) ranged from 50 kPa to 1600 kPa. To mitigate possible effects of disturbance during extraction of soil cakes from the consolidometer and ensure that the specimens reached the true normally consolidated virgin compression line, the specimens were loaded beyond a minimum of twice the cake sedimentation stress or until $> 10\%$ vertical strain occurred, as suggested by Ladd and DeGroot (2003). Once $\sigma'_{vc}$ was reached, the specimens were held at that stress for up to 24 h of secondary compression. Constant volume shear was conducted at a shear strain rate of 5%/hr up until 20% horizontal strain was reached. The constant volume procedure consisted of maintaining specimen height constant by changing the vertical load using a computer-controlled servo system. It was assumed that the required changes in the vertical stress to keep the height constant during shear were equal to the pore pressure that would have generated during a truly undrained shear (Dyvik et al. 1987). All the measured deformations and forces were corrected for apparatus deflection and stacked ring-membrane resistance as applicable. The undrained shear strength was assumed to be equal to the maximum measured horizontal shear stress (DeGroot et al. 1992). The shear modulus $G$ was computed as $\tau/\gamma$ and the undrained Young's modulus $E_u$ was computed assuming Poisson's ratio $\nu = 0.5$ for undrained shear and $E_u = 2(1 + \nu)G = 3G$. 
3.4. Test Results

3.4.1 Consolidation Behavior

Detailed results from Soil #6, the highest plasticity soil in the data set ($LL = 215\%$ and $OM = 20.1\%$), are presented here as being representative of the organic soils. All the other test soils followed the same general behavior during consolidation and undrained shear.

Figure 3.4 and Figure 3.5 present the stress-strain and stress-void ratio behavior of soil #6 during the $K_0$-consolidation phase of the DSS tests. Figure 3.6 shows the virgin compression behavior of this soil created by synthesizing the results from all the test specimens. The higher plasticity soils had significantly higher void ratios at low stresses and experienced much larger compression during consolidation. At larger stresses (i.e. over 1 MPa), the void ratios approach that of the low plasticity BBC and Presumpscot clay. As a result, higher plasticity soils undergo larger strains, specifically in the virgin compression range (i.e. much larger compression ratio), under a similar load. For instance, soil #6 experienced deformations more than two times that of BBC and Presumpscot clay under the final load increment. Expectedly, it was observed that the EOP is significantly larger in organic soils, and also tend to increase among organic soils with increasing liquid limit.

3.4.2 DSS Shear Stress-Strain Behavior

Shear stress and normalized shear stress ($\tau_v/\sigma'_{vc}$) for soil #6 are presented as a function of shear strain at consolidation stresses from 60 kPa to 1600 kPa in Figure 3.7 and Figure 3.8. In all tests, horizontal shear stress increased quickly towards the peak shear stress at which the rate of increase decreased up to the peak followed by a gradual strain.
softening response. As a general trend, with increasing consolidation stress level, the stress-strain behavior became more ductile, the strain to failure increased, and strain softening was less. This is consistent with that obtained by Abdulhadi et al. (2012) and Casey and Germaine (2013). In addition, failure in the inorganic soils occurred at smaller horizontal strains around 4 to 6% compared to greater than 9% and most often around 14%-16% for the organic soils. There is no clear trend between this increase in strain at failure (ductile behavior) and liquid limit. Figure 3.8 illustrates that there is a distinct reduction in $\tau_h/\sigma'_{vc}$ with an increase in consolidation stress level except for the 60 kPa test. It was found that for most of the low stress tests (i.e., <100 kPa) performed on all the soils, slippage was often observed to occur during shear despite use of the waffle stones. This may have in some cases resulted in premature failure of the test at the stone-soil interface instead of developing failure within the soil specimen and a consequent underestimation of $s_u$. Accordingly, interpretation and synthesis to follow of the full set of results for all the test soils is based on the results for tests with consolidation stresses above 100 kPa, and the data related to the tests conducted at stresses below 100 kPa are for illustration purposes.

Figure 3.9 and Figure 3.10 present pore pressure and normalized pore pressure in the specimen during shear for soil #6. As expected for a NC soil, all the specimens have contractive behavior, i.e., positive shear induced pore pressures, throughout shear. Normalized pore pressure increases more slowly with increasing stress level, but it reaches a very similar value for all tests at higher strains towards end of the test.

Figure 3.11 and Figure 3.12 present the effective stress plots and normalized effective stress plots for soil #6. Given that all the specimens were consolidated to a normally consolidated stress state, all specimen showed contractive behavior or the
equivalent of developing positive shear induced pore pressures. For all the tests, in the beginning, the vertical effective stress decreased slightly while shear stress increased quickly followed by a more rapid reduction in vertical stress as shear stress slowly increased towards the peak. For soil #6, peak shear stress occurred when the normalized vertical stress \(\frac{\sigma'_v}{\sigma'_ve}\) reached 0.64 to 0.68.

### 3.4.3 Summary Undrained Shear Behavior

Table 3.2 presents a summary of all the DSS test results and Figure 3.13 presents the undrained strength ratio versus consolidation stress for all tests. The \(s_u/\sigma'_ve\) values are significantly greater than that reported for inorganic soils while the inorganic BBC and Presumpscot clay are within the typical range reported for inorganic clays (e.g., Figure 3.1). Furthermore, \(s_u/\sigma'_ve\) at a given \(\sigma'_ve\) value tends to increase with an increase in liquid limit. It is also clear that all the soils show a general trend of a decrease in \(s_u/\sigma'_ve\) with increasing \(\sigma'_ve\) which is consistent with the findings of Quirós et al. (2000) for a large collection of NC DSS tests performed on inorganic soils with the majority having a liquid limit between 50 and 100% and some limited organic soils. It is also consistent with Casey and Germaine (2013) results for CK0UC tests on inorganic soils. Although the decrease in \(s_u/\sigma'_ve\) with increasing \(\sigma'_ve\) for this work is much greater for the organic soils. Furthermore, the inorganic soils tested by Casey and Germaine (2013) exhibited most of the decrease in \(s_u/\sigma'_ve\) up to \(\sigma'_ve\) equal to around 1000 kPa; whereas the results presented in Figure 3.13 show that there is still a significant decrease in \(s_u/\sigma'_ve\) from 800 to 1600 kPa. Figure 3.13 shows that the \(s_u/\sigma'_ve\) values for tests with \(\sigma'_ve\) below 100 kPa are often inconsistent with the results from the tests at higher stresses which is believed to be due to slippage at the
soil-porous stone interface as noted above. Best fit regression lines for \( \sigma'_{vc} \geq 100 \) kPa and grouping all the organic soil together and the two inorganic soils together results in

\[
\frac{s_u}{\sigma'_{vc}} = 0.697(\sigma'_{vc})^{0.141} \quad \text{six organic soils} \quad \text{Eq. 3.8}
\]

\[
\frac{s_u}{\sigma'_{vc}} = 0.327(\sigma'_{vc})^{0.067} \quad \text{two inorganic soils} \quad \text{Eq. 3.9}
\]

### 3.4.4 Undrained Young’s Modulus

Figure 3.14 and Figure 3.15 illustrate the variation of secant undrained modulus and normalized undrained modulus with shear strain for soil #6. Modulus values at strains lower than 0.02% are not included as the data is not considered reliable at such low strain levels. As expected, \( E_u \) decreases with an increase in \( \gamma \) while it increases with increase in \( \sigma'_{vc} \). Figure 3.15 shows that \( E_u/\sigma'_{vc} \) decreases with increasing \( \sigma'_{vc} \) similar to that found for \( s_u/\sigma'_{vc} \). At smaller strains, the difference between \( E_u/\sigma'_{vc} \) from tests at high and low stresses is larger and this gap decreases at higher stresses and all the measurements tend to merge together. Figure 3.16 presents \( E_u/s_u \) versus shear stress ratio which is defined as the ratio of the measured shear stress and the undrained shear stress \( (\tau_h/s_u) \), which is essentially \( 1/FS \), where FS = factor of safety. The modulus values are plotted for \( \tau_h/s_u \) from 0.2 to 0.8; values less than 0.2 are generally not reliable because of the very small strains at those low stress ratios. Aside from the test with \( \sigma'_{vc} = 60 \) kPa, all the results plot relatively close together throughout shear.

### 3.4.5 Summary Undrained Young’s Modulus

Figure 3.17, Figure 3.18, and Figure 3.19 present secant undrained modulus for all soils tested at shear stress ratios 0.25, 0.50, and 0.75. In all cases \( E_u \) goes up with an increase
in $\sigma'_vc$ and the high plasticity organic soils have lower $E_u$ than the low plasticity inorganic BBC and Presumpscot clay. There is more scatter in the results at lower shear stress ratio of 0.25 which can be due to the sensitivity of the results to accurate measurement of the shear strain and apparatus compressibility at small strains. This scatter reduces as the shear stress ratio increases. There is also greater variation in the results for tests below 100 kPa.

Regression fits for the 6 organic soils for the shear stress ratios of 0.25, 0.50 and 0.75 give:

$$\frac{E_{u.0.25}}{\sigma'_{vref}} = 184\left(\frac{\sigma'_vc}{\sigma'_{vref}}\right)^{0.74} \text{ Eq. 3.10}$$
$$\frac{E_{u.0.50}}{\sigma'_{vref}} = 82\left(\frac{\sigma'_vc}{\sigma'_{vref}}\right)^{0.81} \text{ Eq. 3.11}$$
$$\frac{E_{u.0.75}}{\sigma'_{vref}} = 33\left(\frac{\sigma'_vc}{\sigma'_{vref}}\right)^{0.89} \text{ Eq. 3.12}$$

Figure 3.17, Figure 3.18, and Figure 3.19 also present, for reference, the Casey et al. (2015) equations (Eqs. 3.5, 3.6, and 3.7) which were derived from CK$_0$UC tests. The BBC and Presumpscot results from this work plot near the Casey et al. (2015) equations for $\tau/s_u = 0.25$, below for $\tau/s_u = 0.50$ and well below for $\tau/s_u = 0.75$. This is expected as generally the strain to failure in DSS mode of shear is greater than that for CK$_0$UC shear and as such there is greater degradation in $E_u$ for the same $\tau/s_u$ ratio as the soil strains towards $s_u$ in the DSS. The value of the exponents in Eqs. 3.10 to 3.12 increase with increasing shear stress ratio which is opposite of that found by Casey et al. (2015) but consistent with that reported by others (e.g., Wroth et al. 1979; Viggiani and Atkinson 1995).

Figure 3.20, Figure 3.21, and Figure 3.22 plot $E_u/\sigma'_vc$ versus consolidation stress level for all soils at shear stress ratios of 0.25, 0.50, and 0.75. For the organic soils, $E_u/\sigma'_vc$
decreases with increasing $\sigma'_{vc}$ and the rate of decrease also decreases with an increase in $\tau_b/s_u$. Regression fits for the organic soils with $\sigma'_{vc} \geq 100$ kPa are

$$E_{u,0.25}/\sigma'_{vc} = 622(\sigma'_{vc})^{0.264} \quad \text{Eq. 3.13}$$

$$E_{u,0.50}/\sigma'_{vc} = 199(\sigma'_{vc})^{0.193} \quad \text{Eq. 3.14}$$

$$E_{u,0.75}/\sigma'_{vc} = 55.8(\sigma'_{vc})^{0.113} \quad \text{Eq. 3.15}$$

3.5. Interpretation and Discussion of Results

3.5.1 Undrained Shear Strength

The $s_u/\sigma'_{vc}$ data plotted in Figure 3.13 were fitted using Eq. 3.2 for NC conditions (i.e., $\sigma'_p = \sigma'_{vc}$ and $(OCR)^m = 1$) for each soil to determine the $S_1$ and $T$ parameters. Figure 3.23 and Figure 3.24 present $S_1$ and $T$ versus Casagrande cup LL and show strong correlations, for which best fit regressions give:

$$S_1 = 0.0016 \, LL + 0.47 \quad \text{Eq. 3.16}$$

$$T = -0.0002 \, LL - 0.1127 \quad \text{Eq. 3.17}$$

The LL from the Casagrande Cup was used for Eqs. 3.16 and 3.17 as it showed somewhat less scatter than using the Fall cone LL data. It is unclear why this is the case although the Casagrande cup does have a greater range of LL values compared to the Fall cone. Also plotted for reference in Figure 3.23 and Figure 3.24 are the equations recommend by Casey and Germaine (2013) for CK$_0$UC behavior of inorganic soils between with maximum liquid limit = 80% and $\sigma'_{vc}$ values ranging up to 10 MPa (Eqs. 3.3 and 3.4). Figure 3.23 shows that $S_1$ linearly increases with an increase in LL for the organic clays which is in agreement with findings of Casey and Germaine (2013) although with a significantly lower slope. As for the inorganic soils tested in this work, the BBC result plots
directly on the Casey and Germaine (2013) correlation while the Presumpscot clay result also plots on the line if the 1600 kPa test is not considered or below the line if it is. The Presumpscot clay tested by Casey and Germaine had a lower liquid limit of 33% compared to the 47% for this work and the 1600 kPa test in this study had a high $s_u$ value relative to the overall trend based on the other Presumpscot clay tests and influenced the correlations for that soil as shown in Figure 3.23.

Figure 3.24 illustrates that parameter $T$ decreases with an increase in $LL$. Once again this is similar to that found by Casey and Germaine (2013), but the slope of the organic soils tested in this work is significantly lower; in fact, a simple linear regression fits the data as well as a log regression used by Casey and Germaine (2013). It is also significant to note that the $T$ values for the BBC and Presumpscot clay tested in this work plot well below the Casey and Germaine (2013) equation, unlike that found for the $S_1$ parameter. Given that this work involved DSS mode of shear it is expected that $s_u/\sigma’_v$ values should be lower than that of the CK$_{0}\text{UC}$ tests performed by Casey and Germaine (2013). As such, the results presented in Figure 3.23 and Figure 3.25 suggest that the $T$ parameter reflects undrained shear strength anisotropy and the $S_1$ parameter is apparently independent of mode of shear. For example, BBC with $LL = 47\%$ (for which whether determined from Casagrande Cup or Fall cone makes no difference; Table 3.1) the Casey and Germaine (2013) $S_1$ and $T$ values are 0.38 and -0.032 (Eqs. 3.2 and 3.3) whereas the DSS $S_1$ and $T$ values for BBC from this work are 0.36 and -0.079 (Table 3.2).

It appears that one line fits all the data from this study and Casey and Germaine (2013) in Figs. 23 and 24 (the gray short dotted line). However, these universal correlations are not recommended in the absence of more data from DSS tests on low-plasticity organic
soils and high-plasticity inorganic soils as well as triaxial compression tests on high-
plasticity organic soils. These best fit regressions are as follows

\[ S_1 = -0.75 + \frac{1.71 LL}{22.88 + L} \]  
Eq. 3.18

\[ T = -0.15 + 0.76 e^{-0.045 LL} \]  
Eq. 3.19

Figure 3.25 presents a summary of \( s_u/\sigma'_{vc} \) predictions using the Casey and Germaine (2013) CK\(_0\)UC \( S_l \) and \( T \) values (Eqs. 3.3 and 3.4) for inorganic soils with \( LLs \) of 25, 50 and 75\% (Figure 3.25a) and from this work using Eqs. 3.16 and 3.17 for DSS mode of shear for organic soils with \( LLs \) ranging from 75 to 250\% (Figure 3.25b). The relatively low \( LL \) inorganic soils of Casey and Germaine (2013) go through a transition at liquid limit equal to 40\%; below \( LL = 40\% \) \( s_u/\sigma'_{vc} \) increases with an increase in \( \sigma'_{vc} \), is constant at \( LL = 40\% \), and decreases for \( LL > 40\% \). Conversely, the \( S_l \) and \( T \) parameters derived from the organic soils tested in this work with \( LL \) ranging from 81 to 215\% predict a progressive increase in \( s_u/\sigma'_{vc} \) with increasing \( LL \) at a given \( \sigma'_{vc} \) and a decrease in \( s_u/\sigma'_{vc} \) with increasing \( \sigma'_{vc} \) over the full range of \( LL \) values considered.

### 3.5.2 Correlation with Organic Matter

The \( LL \) of the organic soils correlates well with organic matter (OM) as presented in Figure 3.26 and may be estimated as

\[ LL = 9.1 (OM) + 43.8 \]  
Eq. 3.20

The strong correlation between \( LL \) and organic matter suggests that organic matter can be used as a quick and easy measurement for estimating \( LL \) and consequently \( S_l \) and \( T \). Figure 3.27 illustrates the increase in \( S_l \) with increasing organic matter with

\[ S_1 = 0.0142 (OM) + 0.541 \]  
Eq. 3.21
Figure 3.28 presents the change in $T$ with organic matter with

$$T = -0.0017 \, LL - 0.1225 \quad \text{Eq. 3.22}$$

While there is somewhat more scatter in both these plots compared to the $S_i$ and $T$ correlations with liquid limit the trends are still strong.

### 3.5.3 Undrained Young’s Modulus

Figure 3.29 presents $E_u/\sigma'_{vc}$ versus $LL$ for the six organic soils at $\tau/s_u$ values of 0.25, 0.50 and 0.75 and $\sigma'_{vc} = 100, 200, 400$ and 1600 kPa. The normalized undrained modulus generally decreases as the $LL$ increases for shear stress ratio equal to 0.25 and either slightly decreases with $LL$ or is essentially independent of $LL$ for shear stress ratios of 0.50 and 0.75. Likewise, plots of the Eqs. 3.13 to 3.15 coefficients versus $LL$ for the organic soils show little to no trend for all cases other than the '$a'$ coefficient for Eq. 3.13 for $E_{u,0.25}/\sigma'_{vc}$ which somewhat decreases with an increase in $LL$. Overall consideration of $LL$ does not result in improved correlations unlike that found for the $s_u/\sigma'_{vc}$ relationships (i.e., Eqs. 3.16 and 3.17).

It has historically been common to present $E_u$ data as $E_u/s_u$ versus $\tau/s_u$ (e.g., Ladd et al. 1977, Duncan and Buchignani 1976) but without consolidation stress level. Figure 3.30 presents the data from this work for $\sigma'_{vc}$ values equal to 100, 200, 400 and 1600 kPa. The results from all the tests fall within the lower range of $E_u/s_u$ proposed by Ladd et al. (1977). In most cases, the slope of the decrease in $E_u/s_u$ is very similar to Ladd et al. (1977)’s results for Atchafalaya clay ($LL = 95\%$ and $PI = 75\%$), i.e. decreasing from around 400 at $\tau_h/s_u = 0.2$ to around 55 at $\tau_h/s_u = 0.8$. 

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3.5.4 Implications for Practice

The results from this work clearly show that the normally consolidated DSS normalized undrained shear strength for the six high LL organic soils tested decreases with an increase in consolidation stress level. This is the same as that found by Casey and Germaine (2013) for the CK₀UC behavior of the lower LL inorganic clays that they tested. However, the rate of decrease in \( s_u/\sigma'_{vc} \) with an increase in \( \sigma'_{vc} \) is much greater for the organic clays. The \( S_1 \) and \( T \) parameters from Eqs. 3.16 and 3.17 correlate well with \( LL \) and provide a means for predicting DSS \( s_u/\sigma'_{vc} \) for \( \sigma'_{vc} \) values not included in a laboratory test program. The direct implication of these findings is that the \( S \) parameter in the SHANSEP equation is not a constant (i.e., the soils do not exhibit perfectly normalized behavior) as is typically assumed in practice. On average for the six organic soils tested in this work \( s_u/\sigma'_{vc} \) decreases from 0.36 to 0.24 for an order of magnitude increase in \( \sigma'_{vc} \) from 100 to 1000 kPa. The practical implications of this depends on project specifics such as the depth of interest for design (and hence values of \( \sigma'_{v0} \)) and stress levels at which laboratory tests are conducted. Tests performed at low consolidation stress levels for designs involving higher in situ stresses would result in unsafe \( s_u \) values. Although the SHANSEP procedure requires that specimens be loaded well past the \( \sigma'_p \) into the normally consolidated stress range and hence well beyond \( \sigma'_{v0} \). Furthermore, rarely are truly normally consolidated soils encountered in nature (with exceptions being relatively recent deposits still undergoing self-weight consolidation or the foundation soil of a stage constructed embankment if the first stage or stages load the soil into the NC range). Thus, in many cases the SHANSEP tests will be conducted at \( \sigma'_{vc} \) values too much greater than \( \sigma'_{v0} \) and the resulting \( s_u \) values will often be on the conservative side. Conversely it does mean that in some cases the
laboratory determined \( s_u \) values could be too conservative and uneconomical. The important point being that the laboratory testing scope should carefully consider the specified consolidation stresses for test specimens relative to the design in situ vertical effective stress states. This should avoid unsafe designs or provide an opportunity for more cost-effective designs through adjustment of laboratory measured strengths using the correlations presented herein.

3.6. Summary and Conclusion

This study investigated the results of \( K_0 \) consolidated undrained direct simple shear (DSS) tests performed on six resedimented natural organic soils and two natural inorganic soils at preshear vertical consolidation stress (\( \sigma'_{vc} \)) levels ranging from 50 kPa to 1600 kPa. All tests were conducted on at a normally consolidated state of stress. The six organic soils consisted of three organic clays and three organic silts with liquid limits ranging from 80 to 215%. The two inorganic clays had a liquid limit of 45 and 46%. The organic soils had much lower unit weights, higher void ratios and underwent significantly greater consolidation strains compared to the inorganic soils. They also exhibited, during DSS shear, more ductile behavior, especially with an increase in \( \sigma'_{vc} \), a larger strain at failure and less strain softening behavior. However, the normalized undrained shear strengths were much higher at low \( \sigma'_{vc} \) values than the inorganic soils with this difference becoming smaller at higher stresses (i.e., above 1 MPa).

The organic soils \( s_u/\sigma'_{vc} \) values were found to be a function of \( \sigma'_{vc} \) extending the findings of Abdulhadi et al. (2012) and Casey and Germaine (2013) for low LL inorganic soils to high LL organic soils. Furthermore, the rate of decrease in \( s_u/\sigma'_{vc} \) with an increase
in \( \sigma'_{vc} \) is much greater for the organic soils. These findings have implications for use of the SHANSEP procedure which inherently assumes perfectly normalized behavior and hence constant \( s_u/\sigma'_{vc} \) regardless of \( \sigma'_{vc} \). The decrease in \( s_u/\sigma'_{vc} \) with \( \sigma'_{vc} \) was found to correlate well with liquid limit or organic matter and new equations were developed from this work for estimating \( s_u/\sigma'_{vc} \) at \( \sigma'_{vc} \) values not included in a laboratory test program.

The undrained Young's modulus (\( E_u \)) of the organic soils was also found to be dependent on \( \sigma'_{vc} \) but unlike for \( s_u/\sigma'_{vc} \) the rate of change in \( E_u/\sigma'_{vc} \) with \( \sigma'_{vc} \) did not correlate well with liquid limit. Alternatively, correlations are presented for estimating \( E_u \) and \( E_u/\sigma'_{vc} \) as a function of \( \sigma'_{vc} \) for shear stress ratios (\( \tau/s_u \)) of 0.25, 0.50 and 0.75. Furthermore, plots of \( E_u/s_u \) versus \( \tau/s_u \) are also presented and supplement those presented by Ladd et al (1977).

### 3.7. Acknowledgment

The authors thank David S. Eley, P.E. and his colleagues at GeoEngineers, Inc. (Baton Rouge, LA) for providing the organic soil samples used in this work.
Table 3.1: Summary of the index properties for the eight test soils

<table>
<thead>
<tr>
<th>Soil</th>
<th>LL* (%)</th>
<th>LL** (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>LL&lt;sub&gt;oven&lt;/sub&gt; dried/LL&lt;sub&gt;not dried&lt;/sub&gt;</th>
<th>USCS</th>
<th>Organic Matter (%)</th>
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Note: * Fall cone, ** Casagrande cup
Table 3.2: Summary of undrained shear results from the CKₙDSS tests

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<th>( \gamma ) (°)</th>
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<th>( \sigma'<em>v/\sigma'</em>{vc} )</th>
<th>( \psi ) (°)</th>
<th>( S_i )</th>
<th>( T )</th>
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<th>( E_{u,0.50} ) (MPa)</th>
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Notes: $\psi = \tan^{-1}(s_u/\sigma'_v)$ †values are range for analysis with and without the $\sigma'_w = 1600$ kPa test
Figure 3.1: Undrained strength anisotropy from CK0UC tests on normally consolidated clays and silts (from Ladd and DeGroot, 2003)

Figure 3.2: Correlation between S1 (Eq. 2) and liquid limit for inorganic soils with liquid limit between 26 and 74% (Casey and Germaine, 2013)
Figure 3.3: Plasticity chart for the eight test soils

Figure 3.4: Compression curves in terms of vertical strain for consolidation phase of DSS tests performed on soil #6
Figure 3.5: Compression curves in terms of void ratio for consolidation phase of DSS tests performed on soil #6

Figure 3.6: Composite compression curves in terms of void ratio for normally consolidated stress state for soil #6
Figure 3.7: Shear stress-strain behavior for soil #6 during shear

Figure 3.8: Normalized shear stress-strain behavior for soil #6 during shear
Figure 3.9: Pore pressure versus shear strain for soil #6

Figure 3.10: Normalized pore Pressure versus shear strain for soil #6
Figure 3.11: Stress plots for soil #6 during shear

Figure 3.12: Normalized stress plot for soil #6 during shear
Figure 3.13: Variation in the measured undrained strength ratios for all the soils (regression lines are for $\sigma'_{vc} \geq 100$ kPa)

Figure 3.14: Undrained Young’s modulus versus shear strain for soil #6
Figure 3.15: Normalized undrained Young’s modulus versus shear strain for soil #6

Figure 3.16: Variation in secant undrained modulus (normalized by $s_u$) for soil #6 as a function of shear stress ratio
Figure 3.17: Secant undrained modulus against vertical consolidation stress for a shear stress ratio of 0.25

Figure 3.18: Secant undrained modulus against vertical consolidation stress for a shear stress ratio of 0.50
Figure 3.19: Secant undrained modulus against vertical consolidation stress for a shear stress ratio of 0.75

Figure 3.20: Variation of normalized undrained modulus with vertical consolidation stress for a shear stress ratio of 0.25
Figure 3.21: Variation of normalized undrained modulus with vertical consolidation stress for a shear stress ratio of 0.50

Figure 3.22: Variation of normalized undrained modulus with vertical consolidation stress for a shear stress ratio of 0.75
Figure 3.23: Correlation between the Eq. 3.2 parameter $S_1$ and liquid limit for organic soils. * symbols in gray show the data from Casey and Germaine (2013)
Figure 3.24: Correlation between the Eq. 3.2 parameter $T$ and liquid limit for organic soils. * symbols in gray show the data from Casey and Germaine (2013)
Figure 3.25: Predictions of variation in NC $s_u/\sigma'_{vc}$ vs. $\sigma'_{vc}$ for a) CK0UC behavior of inorganic soils with $LL = 25$, 40, 50 and 75% (Eqs. 3.2, 3.3 and 3.4) and b) DSS behavior of organic soils with $LL = 75$, 100, 150, 200 and 250% (Eqs. 3.2, 3.16, and 3.17)
Figure 3.26: Correlation between liquid limit and organic matter

Figure 3.27: Correlation between parameter $S_\ell$ and organic matter
Figure 3.28: Correlation between parameter $T$ and organic matter
Figure 3.29: $E_u/\sigma'_{vc}$ versus LL at shear stress ratios ($\tau/s_u$) of 0.25, 0.50 and 0.75 for the organic soils at $\sigma'_{vc}$ = a) 100 kPa, b) 200 kPa, c) 400 kPa and d) 1600 kPa
Figure 3.30: $E_u/s_u$ versus $\tau/s_u$ for the organic soils at $\sigma'_{vc} = a) 100$ kPa, b) 200 kPa, c) 400 kPa and d) 1600 kPa
CHAPTER 4
RECOMPRESSION RATIO OF FINE-GRAINED SOILS

There are well-established procedures available for performing consolidation tests and obtaining preconsolidation stress ($\sigma'_p$) and compression ratio (CR) from these tests as well as adjusting measured values for the effects of sample disturbance. However, determination of the recompression ratio (RR) has not gained nearly similar attention and no clear consensus has been provided in the literature on to the best approach to perform consolidation tests and how to determine RR from the measured data. A suite of CRS tests on a variety of high-quality, highly disturbed, and resedimented samples of natural fine-grained soils with unload-reload (U-R) loops at different stress and strain levels were conducted to investigate the effects of stress level and unloading ratio on estimates of RR. Seven different methods were used to estimate RR from the results of each U-R loop. The results show a consistent increase in RR from almost all the methods with increasing stress level and unloading ratio. Different methods resulted in significantly different RR values specifically for higher OCR soils as well as sensitive clays. Based on the findings from this study recommendations for practice are provided for conduct of CRS tests and how to interpret the test results to best estimate RR.

4.1. Introduction

Recompression ratio (RR), preconsolidation stress ($\sigma'_p$), and compression ratio (CR), as determined from 1-D incremental loading (IL) or constant rate of strain (CRS) test results are the key soil properties for estimating primary consolidation settlement of fine-grained soils. $RR$ is the slope of the 1-D strain versus log effective stress curve ($\varepsilon$-$\log \sigma'_v$)
for recompression loading up to $\sigma'_p$ while CR is the slope for virgin compression (i.e., normally consolidated) loading beyond $\sigma'_p$. The 1-D $\varepsilon$-$\log\sigma'_v$ compression curve can be adversely influenced by sample disturbance and hence estimating of primary consolidation parameters can be unreliable. Methods to determine CR and $\sigma'_p$, and effects of sample disturbance on these parameters have been extensively studied (e.g. Schmertmann, 1955; Crawford, 1986, Sandbaekken et al., 1986; Gregory et al., 2006; Lunne et al. 2006, Park and Lee, 2011). On the other hand, limited research has been conducted on the determination of RR and effects of sample disturbance on this parameter. This is due to the much greater impact CR and $\sigma'_p$ have estimating the magnitude of settlement for soils loaded beyond $\sigma'_p$. However, RR can be a more critical element in some cases, including loading of thick layers of highly overconsolidated clay deposits and especially if the in situ soil remains in the recompression zone after loading. In addition, the Schmertmann (1955) method of reconstructing the equivalent in situ compression curve from a laboratory curve measured on disturbed samples relies on an accurate measure of RR.

RR is strongly dependent on sample quality and increasing disturbance results in overestimating its value. Empirically RR can be estimated as a function of CR, but the typical range of reported RR/CR values spans an order of magnitude from 0.02 to 0.20 (Terzaghi et al., 1996). Determining RR from IL or CRS consolidation tests can be unreliable due to small movements, seating and apparatus errors, swelling, recompression of gas bubbles, and sample disturbance (Leonards, 1976; Crawford, 1996; Holtz et al., 2011). Furthermore, a decision needs to be made at which stage of the test to evaluate RR. Leonards (1976) recommended measuring RR from an unload-reload (U-R) loop with unloading effective stress ($\sigma'_u$) at $\sigma'_p$ (or $\sigma'_v + \Delta\sigma'_v$ if $\sigma'_v < \sigma'_p$ where $\Delta\sigma'_v$ is the design
increase in vertical stress) and reloading effective stress ($\sigma'_r$) at $\sigma'_{v0}$ (Figure 4.1). However, it is not clear how loading the specimen to $\sigma'_p$ and unloading to $\sigma'_{v0}$ would affect recompression slope as the void ratio would be lower (significantly lower in case of highly disturbed samples) than the in-situ condition. In addition, conducting an U-R loop around $\sigma'_p$ could make estimating $\sigma'_p$ more difficult using common graphic construction procedures (e.g., Casagrande procedure). Sandbaekken et al. (1986) recommended determining RR from an U-R loop starting from $\sigma'_p$ or $2\sigma'_p$ and going back to $\sigma'_{v0}$. Lunne et al. (1998) proposed the U-R loop to be performed at $2\sigma'_p$ or higher (until reaching the virgin compression) and unloading with unloading ratio ($\sigma'_u/\sigma'_r$) equal to the in-situ overconsolidation ratio (OCR = $\sigma'_p/\sigma'_{v0}$). DeJong et al. (2018) performed CRS tests on resedimented intermediate soil samples with varying degrees of disturbance and recommended conducting the U-R loop at $2.5\sigma'_p$ and unloading to the estimated $K_0=1$ condition.

If an U-R loop is performed several different methods have been presented in the literature to determine RR from the loop which can results in different RR estimates. In some publications, it is not explicitly described how RR values were calculated, although the most common approach appears to take the average slope of the hysteresis loop by connecting $\sigma'_r$ to the intersection of the unloading and rebound curves (Figure 4.1) as suggested by Leonards (1976). Another practice is to measure RR as the slope of the line connecting $\sigma'_u$ to $\sigma'_r$ (Sandbaekken et al., 1986). DeJong et al. (2018) suggested that RR is overestimated for intermediate soils such as silts if evaluated from the $\sigma'_u$ to $\sigma'_r$ slope and recommended the slope of $\sigma'_r$ to the larger of $\sigma'_u$/OCR or $\sigma'_u$/2 as a more accurate
measurement. Das (2004) suggested RR should be determined as the slope of the unloading section of the U-R loop.

Different values of $\sigma_u'$ at which to perform the U-R loop have been suggested by various publications. However, the slope of the U-R loop tends to vary with $\sigma_u'$ and the unloading $\sigma_u'/\sigma_r'$. Gunduz and Arman (2007) investigated the effect of OCR and $e_0$ on RR and CR for resedimented low-plasticity overconsolidated clayey samples. It was concluded that as $e_0$ increases RR and CR decrease and as OCR increases RR and CR also increase.

Oedometer tests were conducted by performing U-R loops at 400, 800, and 3200 kPa stresses and RR/Cr ratios ranged from 0.05 to 0.14 with the higher values being from the U-R loops started at higher $\sigma_u'$. Vipulanandan et al. (2008) conducted IL tests on nine different soft clay samples from Houston, TX and investigated three different methods to determine RR: Leonards (1976), Das (2004), and connecting $\sigma_{v0}'$ to $\sigma_p'$ on an U-R loop. Three unload U-R loops were performed at different $\sigma_u'$ for each test and as up to 760% difference in RR values were reported for the different methods within a single U-R loop. It was also reported that RR increases (significantly in case of CH soils) with increasing $\sigma_u'$.

This paper presents the results of a series of CRS tests conducted on high-quality, disturbed, and resedimented samples of natural clays and silts. Tests were performed using U-R loops with constant $\sigma_u'/\sigma_r'$ or constant $\sigma_r'$ and with several such U-R loops performed during each test at different $\sigma_u'$ values to systematically investigate the effect of such test procedure variations on RR. Various methods of estimating RR were examined and compared to develop a better understanding of potential best practice methods for conducting CRS tests and data interpretation for estimating RR.
4.2. Materials and Methods

4.2.1 Test Soils

Table 4.1 lists the index properties and classification according to the United Soil Classification System (USCS, ASTM D2487, 2016) of the soils tested in this study. The samples were collected using Shelby tube, NGI 76 mm fixed piston thin walled tube, and Sherbrooke block sampler. The database comprises 15 samples of low to high-plasticity soils, although the majority are low plasticity, with liquid limit values determined using the Casagrande cup (ASTM D4318, 2016). The Boston Blue Clay (BBC) Shelby tube samples were collected from Boston, MA and the BBC Sherbrooke Block samples were collected from Newbury, MA (Landon et al. 2007); the Halden Silt samples were collected in Halden, Norway (2016); the Presumpscot clay from, Falmouth, ME, the Onsøy sample from Onsøy, Norway (Lunne et al. 2003); the Connecticut Valley Varved Clay (CVVC) from Amherst, MA (DeGroot and Lutenegger 2003), and the Leda clay from Gloucester, Canada. All the soils are marine clays except for CVVC, which is a lacustrine clay.

4.2.2 Sample Preparation

Samples of three broadly defined quality levels were tested to study the effects of sample disturbance on RR. They included intact or “undisturbed” (block samples and Shelby tube samples), laboratory disturbed, and resedimented. Laboratory disturbed samples were prepared using an undisturbed sample and following the general procedures described in DeJong et al. (2018). This was done by extruding an intact specimen, covering it in plastic wrap, freezing it for a minimum of 24 hrs, and thereafter allowing it to thaw for a minimum of 24 hrs. in a humid room with a controlled temperature of 11°C and
relative humidity of >85%. Cracks typically developed in the samples when following this process. Resedimented samples were prepared according to the general described by Lukas et al. (2018) to form a soil cake from which test specimens were trimmed. This was initially done by mixing the soils with distilled water at 1.25-2 times the liquid limit and allowing the slurry to hydrate overnight. The slurry was then thoroughly mixed under vacuum after being placed in a 102 mm ID acrylic consolidometer to remove any trapped air. Later, the soil was incrementally 1-D consolidated to a target $\sigma_{vc}' = 220$ kPa using a computer controlled GeoJac system followed by unloading to an estimated $K_0 = 1$ condition and allowed to fully swell at that final unloading effective stress prior to removal. Each load increment was applied until at least the End of Primary (EOP) was achieved. Once the sample was extruded it was immediately trimmed into the oedometer ring for CRS testing.

4.2.3 Consolidation tests

The constant rate of strain (CRS) consolidation tests were performed in general accordance with ASTM D4186 Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading and Sandbeakken et al. (1986). The tests were conducted using a GeoTac personal computer based test control and data acquisition system, which includes a load frame, flow pump, CRS consolidometer cell and Sigma-1™ CRS control and data acquisition software. Specimens were hand trimmed using a soil lathe together with a sharp trimming ring and sharp trimming tools. The top and bottom surfaces of the specimens were trimmed flat with a wire saw and a long sharp edged knife with the final trimmed right cylinder dimensions equaling a diameter of 63.5 mm and a height of 19.0 mm. Specimens were placed in the CRS cell with moist top and bottom filter stones. Specimens were initially incrementally loaded up to $0.25\sigma'_{vo}$ to
0.50σ′<sub>v0</sub> (or 10 kPa for highly disturbed samples) before back-pressure saturation to 400 kPa at constant height to ensure no swelling. Constant rate of strain loading was performed at a rate that resulted in a normalized base excess pore pressure ratio of less than 10% in the normally consolidated range; for the clay samples this rate was typically 1%/hr (2.8x10<sup>-6</sup> s<sup>-1</sup>). In all the tests, specimens were allowed to creep for 300 min after every load or unload step to allow the base excess pore water pressure to dissipate before reversing the loading direction. All measurements during testing were made using load, displacement and pressure transducers. The measured data were processed using the methods of Wissa et al. (1971) and also described in ASTM D4186 and Sandbækken et al. (1986). All vertical strains were computed taking into account the apparatus compliance that was determined using a steel disk.

Tests were conducted on each soil in the following general sequence, although not all of these tests were performed on each sample (Table 2):

1. Constant σ′<sub>r</sub> = σ′<sub>v0</sub>: The first test on each soil was conducted on an intact block or tube sample by consolidating the specimen to around 10% strain (i.e., into the normally consolidated stress range) before performing the first U-R loop. The specimen was then unloaded to σ′<sub>v0</sub> followed by reloading with two additional U-R loops at strains of about 15% and 20% prior to unloading to σ′<sub>v0</sub> for each loop. The σ′<sub>p</sub> value from this specimen was used to estimate the sample OCR.

2. Constant σ′<sub>u</sub>/σ′<sub>r</sub>: The second test on each soil was also performed on an intact sample but now the first U-R loop was performed at (0.8-1.0)σ′<sub>v0</sub> and unloading back to σ′<sub>u</sub>/σ′<sub>r</sub> equal to the estimated sample OCR from Test 1. Three other U-
R loops at strains similar to the first test were then conducted, each followed by an unloading step with \( \sigma'_{u}/\sigma'_{r} = \text{OCR} \) unless OCR < 2 for which a \( \sigma'_{u}/\sigma'_{r} = 3-4 \) was used.

3. The third test was conducted on the laboratory disturbed samples and with U-R loops similar to the first test.

4. The fourth test was conducted on a resedimented sample of the same soil used for tests 1, 2 and 3 and tested with U-R loops similar to the first test.

4.3. Test Results

4.3.1 Compression Behavior

Table 4.2 and Table 4.3 summarize the consolidation results from the CRS tests performed for this study. Preconsolidation stresses were calculated using Casagrande method and sample quality was determined using NGI method of \( \Delta e/e_0 \) at \( \sigma'_v \) (Lunne et al., 2006) and SQD method of \( \varepsilon_v \) at \( \sigma'_v \) (Terzaghi et al., 1996). Figure 4.2 and Figure 4.3 present an example set of results for Sherbrooke block sample N2SBS3 in \( \varepsilon - \log \sigma'_v \) and \( \varepsilon - \log \sigma'_v \) spaces. The results from the pair of tests on the intact sample and the pair on the resedimented sample were very consistent and follow the same recompression curve up to \( \sigma'_p \). The laboratory disturbed sample categorized as poor quality by NGI method and acted similar to a remolded sample with \( \sigma'_p \) over 85% smaller than that of the intact specimens. Beyond \( \sigma'_p \), the intact specimens showed the behavior of a structured clay with a distinct break in the compression curve just beyond \( \sigma'_p \) and a steep \( CR \) that progressives decreases with increasing stress. The resedimented specimens started with a lower void ratio, had a much more rounded curve beyond \( \sigma'_p \) and much smaller \( CR \) values which is attributed to
the loss of structure from the resedimentation process. In void ratio space (Figure 4.2) all
of the test specimens indicate a convergence at higher stresses and by extrapolation all
appear to merge at around $0.4e_0$ which is in accordance with the findings of Schmertmann
(1955).

The abovementioned general observations were consistently perceived for all sets
of tests. Disturbed samples had initial void ratios 0.8-0.9 times smaller than those of the
undisturbed samples. In addition, compression curves for all soils tested tended to merge
at around $0.4e_0$, excluding for CVVC sample where the curves seemed to merge at around
$0.25e_0$. This difference can be due to the fact that because of the layering in CVVC samples
it was not possible to obtain the same layering of clay and silt in all the specimens.

Table 4.3 presents the maximum CR slope ($CR_{\text{max}}$) measured for the different tests.$CR_{\text{max}}$, as expected, is higher for undisturbed samples and decreases significantly for
disturbed and resedimented samples. $CR_{\text{max}}$ is also higher in case of the more structured
sensitive soils as corroborated by the higher LI values listed in Table 4.1 for those samples.

### 4.3.2 Recompression Ratio

From Figure 4.2 and Figure 4.3 it can be seen that the general slope of the U-R
loops increased with increasing $\sigma'_u$ and $\sigma'_u/\sigma'_r$. To study these changes, nine different
methods of estimating RR were used for each U-R loop, when applicable, as follows
(Figure 4.4):

- $RR_{\text{Casa}}$ is the traditional RR value calculated by Casagrande method from the
  line joining $\sigma'_{v0}$ to $\sigma'_p$.

- $RR_1$ is the slope of the line from $\sigma'_u$ to $\sigma'_r$ (Sandbaekken et al., 1986).
- RR$_2$ is the slope of the line connecting $\sigma'_r$ to the intersection of unload and rebound curves (Leonards, 1976).

- RR$_3$ is determined by calculating a Casagrande preconsolidation stress value for the intended U-R loop ($\sigma'_{p,u-l}$) and measuring the slope of the line joining this point to the point where the recompression loops starts curving (after the first flat portion of the loop). This method is chosen as representative of the average slope of recompression portion of the loop curve.

- RR$_4$ is similar to RR$_3$ with the difference being that the slope of the line connecting $\sigma'_{p,u-l}$ to $\sigma'_{p,u-l}/$OCR is measured. This method is a facsimile of the Casagrande method on every loop.

- RR$_5$ is the slope of the line connecting $\sigma'_{\nu \theta}$ to $\sigma'_p$ on the U-R loop.

- RR$_6$ is the slope of the line joining $\sigma'_u$ to $\sigma'_u/$OCR.

- RR$_7$ is the average slope of the U-L loop just before $\sigma'_{p,u-l}$.

- RR$_8$ is the slope of initial recompression slope right after $\sigma'_{\nu \theta}$.

All the methods were calculated using the reloading portion of the curves and not the rebound part. Table 4.3 presents the RR$_{casa}$ and RR$_2$ results for all the tests as well as $\sigma'_u$ at which each loop was conducted. It can be seen that there are considerable variations in the results for all soils. The measured RR$_2$ values from different loops for a single test in many instances differ between two to nine times.

Figure 4.5, Figure 4.6, and Figure 4.7 present RR values from all the different methods for all the soils with respect to $\sigma'_u/\sigma'_r$, $\sigma'_u/\sigma'_p$, and $\varepsilon_v$ at $\sigma'_u$, respectively. A general trend of increase in RR with increasing $\sigma'_u/\sigma'_r$ and $\sigma'_u/\sigma'_p$ values can be observed in Figure 4.5 and Figure 4.6. This trend is stronger with $\varepsilon_v$ at $\sigma'_u$ as shown in Figure 4.7.
RR_{Casa} and RR_8 values are not shown in this plot as the $\epsilon_v$ at $\sigma_u'$ is only considered the U-R loops, but $\sigma_u'/\sigma_r'$ and $\sigma_u'/\sigma_p'$ are assumed to be equal to OCR and 1, respectively. These trends confirm that RR, in general, increases with increase in $\sigma_u'$ and/or $\sigma_u'/\sigma_r'$. In addition, the more systematic increasing trend between RR and $\epsilon_v$ at $\sigma_u'$ suggests that RR is more influenced by stress or strain level than the unloading ratio.

Figure 4.8 illustrates RR_1 to RR_7 values versus $\epsilon_v$ at $\sigma_u'$ in separate plots which shows that RR values determined from all the methods, excluding RR_5, increased with increasing $\epsilon_v$ at $\sigma_u'$. RR_5 values did not show any meaningful trend with strain level and their values were generally smaller that RR from other methods. Figure 4.9 also presents the range of RR values from each method. The following conclusions are drawn from the trends shown in Figure 4.8 and Figure 4.9:

- RR_1 and RR_2 values were very close as the intersection of rebound and recompression curves and $\sigma_u'$ are close to each other.
- RR_5 as explained above had the lowest mean value followed by RR_1 and RR_2 with values <0.016. These lower values of RR_5 are due to the fact that this method determines RR from the slope of the first portion of the U-R loop and this slope is mostly shallow if the reload starts at $\sigma_{v0}'$ as the recompression loops start with a flat line in the beginning and the slope increases with increasing load.
- RR_8 had the highest average and range of results which suggests it is most influenced by sample disturbance and overestimates RR since it was larger than RR_{Casa}, while RR_{Casa} values tend to be higher due to the disturbance effects.
Mean value of RR\textsubscript{7} was slightly higher than RR\textsubscript{Casa} which suggests this approach also overestimates RR. RR\textsubscript{7}, in most cases, was somewhat larger than RR\textsubscript{4}.

RR\textsubscript{3} and RR\textsubscript{4} values were mostly very close to RR\textsubscript{4} values being slightly higher in the constant $\sigma'$ tests.

Figure 4.10, Figure 4.11, and Figure 4.12 plot the RR values from all the tests on BB-6 sample; for which the undisturbed samples had excellent quality ratings (Table 4.2). RR values range over one order of magnitude from 0.004 (RR\textsubscript{1}) to 0.035 (RR\textsubscript{8}) which could potentially result in that large of a difference in settlement estimates. There is a distinct trend of increase in RR with stress and strain level. RR values from different methods on different loops, excluding RR\textsubscript{5} and RR\textsubscript{7}, increased with a similar slope. However, the rate of gain was higher in the constant $\sigma'_u$ tests since both unloading ratio and stress level increased during these tests. RR\textsubscript{Casa} values were in the higher end of the measured RR values. Similar trends were observed for CVVC and Leda clay.

Figure 4.13, Figure 4.14, and Figure 4.15 show the results of the two tests on undisturbed and the two tests on resedimented samples of N2SBS3, for which the undisturbed samples were also of excellent quality (Table 4.2). The general increase in RR with stress level and $\sigma'_u/\sigma'_r$ can be seen here as well, but the rates at which RR from each method increased is not as distinct as BB-6. This increase for most methods still occurred at a comparable rate. RR values varied from 0.01 to 0.04. RR\textsubscript{Casa}, RR\textsubscript{3}, RR\textsubscript{4}, and RR\textsubscript{6} values from the two constant $\sigma'_u/\sigma'_r$ and the two constant $\sigma'$ tests were similar, respectively. For the constant $\sigma'_u/\sigma'_r$ tests, RR\textsubscript{3}, RR\textsubscript{4}, and RR\textsubscript{7} values were nearly constant with less than 10% variation for the different loops. RR\textsubscript{1}, RR\textsubscript{2}, and RR\textsubscript{6}, on the other hand, increased up
to 61% in these tests which was due to the higher rebound values at higher $\sigma'_u/\sigma'_p$. For the constant $\sigma'_r$ tests, RR values were generally higher than constant $\sigma'_u/\sigma'_r$ tests. In addition, RR values increased for all the methods, excluding RR$_5$, with increasing stress level. Other BBC samples as well as Onsøy sample followed the same patterns.

For $\sigma'_u/\sigma'_r$ values of around 2, U-R loops were very flat (specifically at low strains) and RR$_1$ and RR$_2$ values became very small. For constant $\sigma'_r$ test on ST3 sample, RR$_1$ and RR$_2$ from the first loop ($\varepsilon_v$ at $\sigma'_u = 2.5\%$) were respectively 24 and 19 times smaller than the same values from the final loop ($\varepsilon_v$ at $\sigma'_u = 11.6\%$) while RR$_3$, RR$_4$, and RR$_7$ values remained nearly constant for all the loops.

For the tests on disturbed samples, RR slopes from all the methods, excluding RR$_7$, constantly increased with increasing $\sigma'_u$ and $\sigma'_u/\sigma'_r$ regardless of the type of the tests (i.e., constant $\sigma'_u/\sigma'_r$ versus constant $\sigma'_r$). Similar to the other tests, this increase in RR$_3$, RR$_4$, and RR$_7$ values was smaller than the change in RR$_1$, RR$_2$, and RR$_6$ values.

RR values from the first loop on the highly disturbed N1BS10A sample ($\sigma'_u/\sigma'_r = 3.45$ and $\varepsilon_v$ at $\sigma'_u = 10.7\%$) were considerably smaller than the ones obtained from first loop of the constant $\sigma'_r$ tests on the undisturbed sample ($\sigma'_u/\sigma'_r = 12.2$ and $\varepsilon_v$ at $\sigma'_u = 3.1\%$) which suggests that $\sigma'_u/\sigma'_r$ has more influence on RR values than sample disturbance as the slope RR for the highly disturbed specimen, which is expected to be the highest RR measurement, is lower than the slope from the undisturbed specimen, but with larger $\sigma'_u/\sigma'_r$. A similar pattern was observed for the other soils.
RR3, RR4, and RR7 values from the constant $\sigma'_r$ tests on the low PI Halden silt were constant for all the loops, except from the one loop where $\sigma'_u/\sigma'_r = 92$. RR5 did not change for any of the tests and RR1, RR2, and RR6 increased with stress level.

It is certainly clear that RR4 values tend to be in a narrow range, specifically for the constant $\sigma'_u/\sigma'_r$ tests, where for many cases, RR values were the same for all the loops. RR4 values for $\sigma'_u/\sigma'_r < OCR$ loops were noticeably smaller than those calculated from the loops on the same soils with $\sigma'_u/\sigma'_r > OCR$. RR4 for clays also increased significantly when $\sigma'_u/\sigma'_r > 10$ which might be due to the very high curvature of the recompression part of the loops when $\sigma'_u/\sigma'_r$ is large. This increased curvature also makes obtaining accurate estimates of $\sigma'_{\rho, U-L}$ challenging. The curves also become more rounded as the stress/strain level increases. Maximum curvature points for all soils occurred in the close proximity of the intersection of unloading and reloading curves.

Limited number of IL tests were also performed on some of the soils for comparison purposes. The slope of the U-R loops from those tests were consistent with CRS results. However, the curves were not as well-defined as the continuous CRS curves, specifically for $\sigma'_u/\sigma'_r < 2.0$ loops where rebound and recompression curves mostly plot on top of each other. In general, tested soils do not rebound much up to $\sigma'_u/\sigma'_r < 2$ where the rebound rate starts to increase with increasing $\sigma'_u/\sigma'_r$ values.

4.4. Discussion of Results

The results presented in the previous section show that RR can vary significantly depending on the method used to estimate it. On average, RR may vary more than 240% from a single loop and more than 340% from a single test with higher numbers being for
more sensitive or higher OCR soils. For an ideal sample with no disturbance, \( RR_{Casa} \) should most closely reflect the in-situ value, however, due to the unavoidable, and potentially highly variable, sample disturbance, \( RR_{Casa} \) is expected to overestimate the in situ RR value, with the possible exception of high-quality block samples collected from relatively shallow depth. Theoretically, \( RR_4 \) measured from a loop performed at near or at \( \sigma'_p \) and unloaded back to \( \sigma'_{v0} \) (\( RR_{4,1} \)) on a high-quality sample could mitigate to some degree effects of sample disturbance and provide a practical estimate of RR.

\( RR_{Casa} \) was larger than \( RR_{4,1} \) for all the soils, apart from ST3 sample with OCR = 1.25. Casagrande estimation of the \( \sigma'_p \) for samples with a relatively steep recompression slope up to \( \sigma'_p \) is often considerably above the measured compression curve (i.e., lower vertical strain). In other words, Casagrande method in such cases might compensate for the influence of disturbance to some unknown degree (i.e., the line connecting \( \sigma'_{v0} \) to \( \sigma'_p \) plots noticeably above the recompression curve). For example, it resulted in a very flat slope of \( RR_{Casa} \) for samples with OCR<1.5. Although the in situ recompression slope is not necessarily linear (in \( e - \log \sigma'_v \) space) and could naturally be rounded.

Table 4.4 shows the range of variation of RR with respect to \( RR_{4,1} \) from the constant \( \sigma'_u / \sigma'_r \) tests on the high-quality BBC samples. Obviously, \( RR_5 \) values are not applicable to constant \( \sigma'_u / \sigma'_r \) tests as the soils are not unloaded back to \( \sigma'_{v0} \). \( RR_1 \) and \( RR_2 \) values were significantly smaller than \( RR_{4,1} \) values for the initial loops. \( RR_1 \) and \( RR_2 \) slopes were closer to \( RR_{4,1} \) for the loops performed at large strains (i.e., >15%). \( RR_6 \) followed similar trends with slightly higher values compared to \( RR_1 \) and \( RR_2 \). For CVVC and Presumpscot clay, \( RR_1, RR_2, \) and \( RR_6 \) from the loops with \( \sigma'_u / \sigma'_p \approx 2 \) were in the 20% range of \( RR_{4,1} \). \( RR_3 \) and \( RR_4 \) values for \( \sigma'_u / \sigma'_p < 3 \) loops on BBC samples tended to be in the 20% and 15%
range of RR_{4,1}, respectively. It can be seen from Table 4.4 that these two methods are in the closest range of RR_{4,1}. However, RR_3 and RR_4 for CVVC and Presumpscot clay samples were somehow larger than RR_{4,1}. For both these two soils only one constant \( \sigma'_u/\sigma'_r \) test was performed and \( \sigma'_u/\sigma'_r \) on the first loop was smaller than in-situ OCR which probably resulted in smaller RR_{4,1} values. RR_7/RR_{4,1} variation for all the soils was similar to that of RR_3 and RR_4 but with larger magnitude. Overall, RR_8 values were similar to RR_{Casa} values but there was more scatter in the data which is shown by higher standard deviation in Table 4.4.

The first loop for the constant \( \sigma'_r \) tests was not conducted at \( \sigma'_p \) to obtain accurate estimates of \( \sigma'_p \) and RR_{Casa}. Therefore, the first U-R loop for these tests occurred at \( \sigma'_u/\sigma'_p \approx 2 \) where RR_4 is potentially higher than that of a loop performed at around \( \sigma'_p \). Therefore, RR_{4,1} values from the constant \( \sigma'_u/\sigma'_p \) tests on the same soils are used as the reference point and compared to the values from the loops conducted around 2\( \sigma'_p \) or 10% strain in the constant \( \sigma'_r \) tests. Table 4.5 summarizes the results. RR_{Casa} and RR_8 for all the tests were higher than RR_4 values. RR_1 and RR_2 values for BBC and Presumpscot clay were in good agreement with RR_{4,1} values with RR_1 values being closer to RR_{4,1} mostly for the loops where \( \sigma'_u/\sigma'_p = 1.5-2.0 \) and RR_2 for the \( \sigma'_u/\sigma'_p = 2.0-2.5 \) loops. It is concluded from results presented in Table 5 that RR_1 and RR_2 provided the closest estimate of RR_{4,1} for these soils. This finding has practical significance as it is common practice to use RR_1 or RR_2 values from U-R loops performed at around 2\( \sigma'_p \) with \( \sigma'_r = \sigma'_{v,0} \) as design recompression ratio. For the CVVC sample RR_1 and RR_2 overestimated RR_{4,1} by 27% and 38%, respectively. This difference can be because of the explained underestimation of RR_{4,1} for these soils as well as the fact that the constant \( \sigma'_u/\sigma'_p \) and constant \( \sigma'_r \) tests had
different soil layering (i.e., different $e_0$) and RR$_{4,1}$ values might not be relevant to constant $\sigma'_r$ tests. RR$_3$, RR$_4$, RR$_6$ and RR$_7$ values for all the soils were higher than RR$_{4,1}$. RR$_5$ can be lower or higher than RR$_{4,1}$ with no trend.

The previous conclusions apply to all the resedimented samples as well. However, for constant $\sigma'_r$ tests on highly disturbed samples of BBC and Presumpscot Clay, RR$_1$, RR$_2$, and RR$_6$ values tended to be noticeably larger or smaller than RR$_{4,1}$ from the undisturbed samples. RR$_3$, RR$_4$, and RR$_7$ values from U-R loops conducted at $9\% < \text{strain} < 15\%$ and $\sigma'_u/\sigma'_p > 1$ ($\sigma'_p$ measured from undisturbed tests) seem to vary less than 25% from RR$_{4,1}$. On the other hand, for the CVVC sample, RR$_1$ and RR$_2$ from such loops were very similar to RR$_{4,1}$.

Based on the observations in this study and assuming that RR$_{4,1}$ for good-quality samples is a reliable representative of the recompression ratio, the most appropriate practice to conduct CRS tests with respect to determining RR is as follows:

In the case of access to high-quality samples and a reasonable estimate of $\sigma'_p$, the best approach seems to be loading the specimen up to $\sigma'_p$ before unloading it back to the smaller of $\sigma'_v/1.2$ or $\sigma'_u/3$ (to avoid a flat recompression curve). The specimen should then be loaded to any desired effective stress. RR$_4$ may be calculated as the recommended RR for these tests.

In the case of lower quality samples, it is best to conduct the U-R loop at around 10% strain, or $2\sigma'_p$ and unload the specimen to the smaller of $\sigma'_u/(1.2\text{OCR})$ and $\sigma'_u/3$. Again, recompression ratio can be determined as RR$_4$. 

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Initial estimates of $\sigma'_p$ are not always available in practice. In such conditions, CRS tests can be conducted by loading the specimen to 10% strain or 2$\sigma'_p$ and unloading to $\sigma'_{v0}$. Recompression ratio in this case may be estimated from RR$_2$ method.

4.5. Conclusion

There appears to be no consensus in the literature on best practice methods for conduct of 1-D consolidation test for determining the recompression ratio of soils. CRS test results from a variety of different quality samples (i.e., high-quality, highly disturbed, and resedimented) prepared from block and Shelby tube samples were investigated in this study. Each test was performed with several U-R loops at different stress levels and with different $\sigma'_u/\sigma'_r$ ratios to examine how they effect on estimation of RR. The results show that the slope of the U-R loops becomes steeper with increasing $\sigma'_u/\sigma'_p$, $\sigma'_u/\sigma'_r$, or $\varepsilon_v$ at $\sigma'_u$.

Seven different methods for interpreting an U-R loop (i.e., Fig. 4 RR$_1$ to RR$_7$) and two slopes from the initial recompression part of the compression curve (i.e., RR$_{Casa}$ and RR$_8$) were examined. RR$_{Casa}$ and RR$_8$, as anticipated, produced the highest values, followed by RR$_7$, RR$_4$, and RR$_3$. RR$_1$, RR$_2$, and RR$_5$ gave the lowest estimates of the recompression ratio. RR for all the methods seemed to increase as stress level increased, excluding RR$_5$ and RR$_4$ which were the least stress-dependent methods producing nearly constant results in most constant $\sigma'_u/\sigma'_r$ tests. In addition, RR values from all the methods, apart from RR$_5$, increased with increasing unloading ratio. The results showed that RR estimates can vary more than 240% from a loop and over 340% through one test depending on where and how the U-R loop is performed during the test. This variation can lead to significant uncertainty
in settlement estimations especially in the case of heavy loading of a thick highly overconsolidated clay that does not reach $\sigma'_p$.

RR$_4$ from a U-R loop performed at $\sigma'_u = \sigma'_p$ with $\sigma'_u/\sigma'_r = 1.2\text{OCR}$ (RR$_{4,1}$) is recommended as the most reliable estimate of RR. For constant $\sigma'_u/\sigma'_r$ tests, RR$_4$ provided acceptable estimates of RR$_{4,1}$ where $\sigma'_u/\sigma'_r > 1.2\text{OCR}$. RR$_4$ was significantly smaller in the tests with $\sigma'_u/\sigma'_r < \text{OCR}$. In case of constant $\sigma'_r$ tests on BBC and Presumpscot clay samples, RR$_1$ and RR$_2$ slope on a U-R loop performed at $\sigma'_u/\sigma'_p = 1.5-2.5$ seemed to be a reliable representative of RR$_{4,1}$. RR$_4$ was higher than RR$_{4,1}$ in these tests due to the very large $\sigma'_u/\sigma'_r$ values. For highly disturbed samples, RR$_4$ calculated from a U-R loop at a strain larger than 9% and smaller than 15%, where $\sigma'_u/\sigma'_p > 1$, was consistent with RR$_{4,1}$ values from undisturbed samples.
Table 4.1: Summary of the sampler type and index and classification properties of the soil samples

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<tr>
<th>Sample</th>
<th>Soil</th>
<th>Sampler Type</th>
<th>Depth (m)</th>
<th>Ave. $w$ (%)</th>
<th>LL (%)</th>
<th>PI (%)</th>
<th>LI (-)</th>
<th>USCS</th>
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Notes: Average water content ($w$) from CRS tests, $LL$ = liquid limit, $PI$ = plasticity index, $LI$ = liquidity index, $CL$ = low plasticity clay, $CH$ = high plasticity clay, $ML$ = low plasticity silt
Table 4.2: Summary of the consolidation test results

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<th>w (%)</th>
<th>$e_0$ (%)</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$\sigma'_{v0}$ (kPa)</th>
<th>$\sigma'_{p}$ (kPa)</th>
<th>OCR (-)</th>
<th>$\Delta w/e_0$ (%)</th>
<th>$e_v$ (%)</th>
<th>NGI</th>
<th>SQD</th>
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Note: U = undisturbed, D = Disturbed, R = Resedimented
Table 4.3: Summary of the consolidation test results

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<th>$\sigma'_{\mu}$ (kPa)</th>
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Note: U = undisturbed, D = Disturbed, R = Resedimented
Table 4.4: Variation of recompression ratio values for BBC samples with respect to RR\(_{4,1}\) from constant \(\sigma'_u/\sigma'_r\) tests

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<th>RR(<em>{\text{Cas}})/RR(</em>{4,1})</th>
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<td>0.57</td>
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<td>0.74</td>
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<td>Minimum</td>
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<td>0.03</td>
<td>0.81</td>
<td>0.86</td>
<td>0.41</td>
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<td>Maximum</td>
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<td>0.95</td>
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</table>

Note: S.D. = Standard deviation

Table 4.5: Variation of recompression ratio values for BBC and Presumpscot clay samples with respect to RR\(_{4,1}\) from constant \(\sigma'_r\) tests

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<td>7</td>
<td>7</td>
<td>8</td>
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<tr>
<td>Mean</td>
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<td>0.92</td>
<td>1.00</td>
<td>1.38</td>
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<td>0.79</td>
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Note: S.D. = Standard deviation
Figure 4.1: Common procedures for estimating the recompression ratio RR (after Leonards 1976, Holtz et al. 2011)
Figure 4.2: One-dimensional compression behavior of N2SBS3 block in $\varepsilon - \log \sigma'_v$ space
Figure 4.3: One-dimensional compression behavior of N2SBS3 block in $e - \log \sigma'_v$ space
Figure 4.4: Different methods used to determine RR in this study
Figure 4.5: Variation in recompression ratio with $\sigma'_u/\sigma'_r$.

Figure 4.6: Variation in the recompression ratio with $\sigma'_u/\sigma'_p$. 
Figure 4.7: Variation in the recompression ratio with $\varepsilon_v$ at $\sigma'_u$
Figure 4.8: Variation in the recompression ratio with $\varepsilon_v$ at $\sigma'_u$ for all the tested soils
Figure 4.9: Range of RR values from all the methods. The line in the box presents the median, the top and bottom boarders of the box show 25th and 75th percentile, the two upper and lower whiskers represent 10th and 90th percentile, and the outliers are presented with dots.

Figure 4.10: Variation in recompression ratio with $\sigma'_u/\sigma'_r$ for BB-6 sample
Figure 4.11: Variation in the recompression ratio with $\sigma_u'/\sigma_p'$ for BB-6 sample

Figure 4.12: Variation in the recompression ratio with $\varepsilon_v$ at $\sigma_u'$ for BB-6 sample
Figure 4.13: Variation in recompression ratio with $\sigma'_u/\sigma'_r$ for N2SBS3 sample

Figure 4.14: Variation in the recompression ratio with $\sigma'_u/\sigma'_p$ for N2SBS3 sample
Figure 4.15: Variation in the recompression ratio with $\varepsilon$ at $\sigma_u'$ for N2SBS3 sample
CHAPTER 5
SUMMARY AND CONCLUSIONS

The main objectives of this dissertation were to develop correlations between consolidation parameters and soil index properties for soft fine grained soils from the coastal Louisiana region, gain a better understanding of the normalized undrained shear behavior of high liquid limit organic soils, and investigate different methods for estimating the recompression ratio from consolidation tests. The objectives were met through the research presented in three chapters that detail the results and analysis of the site investigation data and laboratory test program. A brief overview of the most important results of these chapters is presented below.

Chapter 2 presented a collection of empirical correlations for estimating consolidation design parameters of fine-grained soils from the coastal Louisiana region. The correlations were developed through investigation of the relationship between different consolidation parameters (i.e., compressibility, preconsolidation stress and coefficient of consolidation) and basic index measurements (i.e., water content, void ratio, Atterberg Limits, and dry unit weight) using data from 15 marsh creation projects. Different correlations were suggested with the strongest being for estimating compression index from natural water content considering inorganic and organic soils separately.

Chapter 3 presented the results of a suite of DSS tests performed on six resedimented natural high liquid limit organic soils and two lower liquid limit inorganic soils. Tests were performed on normally consolidated specimens at consolidation vertical effectives stresses ranging from 50 to 1600 kPa. The organic soils exhibited more ductile behavior and had higher $s_u/\sigma'_{vc}$ and lower $E_u/\sigma'_{vc}$ than the inorganic soils. $s_u/\sigma'_{vc}$ for all the
soils decreased with increasing $\sigma'_{vc}$. Empirical correlations were developed to estimate $s_u/\sigma'_{vc}$ from $\sigma'_{vc}$ as a function of liquid limit or organic matter for the organic soils. $E_u/\sigma'_{vc}$ was found to decrease as $\sigma'_{vc}$ increased with no trend with liquid limit. Alternatively, correlations are presented for estimating $E_u$ and $E_u/\sigma'_{vc}$ as a function of $\sigma'_{vc}$ for applied shear stress ratios ($\sigma/s_u$) of 0.25, 0.50 and 0.75.

Chapter 4 presented the results of an investigation into the different practices for determining the recompression ratio from consolidation tests. CRS tests were conducted on fourteen different block and Shelby tube samples of clay and silt in intact, disturbed, and resedimented conditions. Unload-reload loops were performed at different stresses and with different unloading ratios (constant $\sigma'_{u}/\sigma'_{r}$ and constant $\sigma'_{r}$). RR was estimated for each U-R loop using seven different methods. The results showed a consistent increase in RR values from almost all interpretation methods with increasing stress level and unloading ratio. In addition, RR values on average varied more than 240% from an U-R loop and over 340% through one test depending on where and how the U-R loop was performed. RR$_4$ determined from a U-R loop performed at $\sigma'_u = \sigma'_p$ with $\sigma'_u/\sigma'_r = 1.2OCR$ (RR$_{4,1}$) was recommended as the best estimate of RR.
REFERENCES


