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Relationship Between Small Strain Shear Modulus and Undrained Shear Strength in Direct Simple Shear

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RELATIONSHIP BETWEEN SMALL STRAIN SHEAR
MODULUS AND UNDRAINED SHEAR STRENGTH IN
DIRECT SIMPLE SHEAR

BY

BRIAN A. BAFFER

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE OF
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MASTER OF YOUR DEGREE THESIS
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ABSTRACT

Over the past several years, measuring the shear wave velocity (V_s) of soils has become more common for earthquake site response analyses and for estimating soil type and strength to better assist geotechnical engineers in forming more accurate soil profiles for engineering design. Most of the measurements in the laboratory have been conducted in conjunction with triaxial tests due to the larger sample sizes and availability of equipment in the U.S. It is less common to measure the shear wave velocity in the direct simple shear (DSS) apparatus. This is partly because, up until the past decade, there have been few manufacturers of DSS equipment. In addition, soil samples in the DSS test are typically less than 3 cm high and measuring shear wave velocity over these distances is affected by electrical near-field effects. This is unfortunate considering that the undrained shear strength obtained from the DSS test is preferred because it provides an average value of shear strength compared to other modes of shearing (e.g. compression, extension).

There are two objectives of this thesis. The first objective is to develop a shear wave velocity measurement system for a commercially available DSS apparatus operated at the University of Rhode Island. The second objective is to use this new system to evaluate a possible link between small strain properties (e.g. shear wave velocity and the small strain shear modulus) and large strain properties (e.g. undrained shear strength) of cohesive soils.

A laboratory testing program was performed in which shear waves were generated and measured using piezoceramic bender elements mounted in DSS end caps constructed of brass. Careful grounding and waterproofing of the bender

elements significantly reduced electrical noise and near-field effects, enabling clear interpretation of shear wave velocities over a wide range of densities and effective stresses. Three soils were tested in this study: a marine clay from the Gulf of Mexico, a sensitive clay from Maine called Presumpscot clay, and an organic silt from Narragansett Bay, Rhode Island. Shear wave velocity measurements were made at the end of consolidation and prior to undrained shearing of the samples. The ratio of small strain shear modulus (from the shear wave velocity) to undrained shear strength for the three soils was compared to published data of 11 different soils of varying stress histories and plasticity. The agreement with published data was very good, illustrating that the new DSS- V_s system works well and there is a clear link between small and large strain properties of cohesive soils.

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1. Introduction

1.1 General overview

The objectives of this thesis are to develop a shear wave velocity measurement system in a direct simple shear apparatus and to evaluate a possible link between small and large strain properties of cohesive soils. Development of such a relationship would help link deformation and stability analyses in practice.

The undrained shear strength (S_u) describes the shear resistance of a soil (usually cohesive) under loading conditions in which drainage is prevented. The undrained shear strength of soil depends on a variety of factors, including soil type, consolidation stress, consolidation time, shear rate or rate of strain, stress history, and loading direction. Typical loading directions encountered in many practical problems such as bearing capacity of foundations and slope stability analyses include compression, extension, and simple shear. This is illustrated in Figure 1.1. The undrained shear strength can be measured in a variety of ways, however the most common tests are the consolidated undrained triaxial compression test (TC) and the unconsolidated undrained triaxial compression test. The direct simple shear test (DSS) gives an average value of undrained strength compared to compression and extension tests, however it has not been widely used in engineering practice in the U.S. because of an historic lack of DSS equipment manufacturers.

Figure 1.1 Different ways the undrained shear strength can relate to the strength of soil in common engineering problems (adapted from Budhu, 1999).

The small strain shear modulus (G_o) is a very important dynamic soil property. It is used primarily in site response analyses to model the propagation of seismic waves through the soil during an earthquake. More recently there have been efforts to utilize the small strain shear modulus in estimating the settlement of foundation systems (Kramer, 1996). Small strain shear modulus can be calculated from shear wave velocity (V_s) directly using the following equation (Zhang et al 2005):

$$G_o = \rho V_s^2 \quad (1)$$

where ρ is the bulk density.

The shear wave velocity can be determined by using piezoelectric bending actuators, also referred to as bender elements. Figure 1.2 shows a diagram of a basic setup with a soil sample and bender elements in a testing apparatus.

Figure 1.2 DSS Specimen with bender elements to measure shear wave velocity (Landon et al., 2007).

These elements generate and receive an elastic shear wave through a sample, and the shear wave velocity is an attractive soil property because it is strongly dependent on the soil type, void ratio, effective stress conditions and behavior at the particle contacts. Shirley and Hampton (1978) first experimented using acoustic measurements to determine the shear speed of certain elements, and these measurements have become more common in research laboratories. These measurements have been made routinely since the early 1990's in some labs in conjunction with triaxial testing. However, bender elements are rarely installed in direct simple shear equipment, partly because the size of the samples is quite small, and also the difficulty in setting up undisturbed samples.

The shear wave velocity measurement system developed in this study will be similar to the set up shown in Figure 1.3. A function generator will be used to transmit a signal to a bender element in a specially designed DSS end cap. A second bender element will receive the shear wave and an amplifier and oscilloscope will be used to interpret the signals.

Figure 1.3 Setup for measuring shear wave velocity in a soil specimen using bender elements (adapted from Chan 2012).

Three soils will be used in this study to validate the new shear wave velocity system and evaluate the link between small strain (G_o) and large strain (S_u) soil properties. These are a marine clay from the Gulf of Mexico, a sensitive clay from Maine called Presumpscot clay, and an organic silt from Narragansett Bay, Rhode Island.

1.2 Organization of This Thesis

The organization of the thesis will be in three parts.

Chapter 2 will present a literature review including background information about the direct simple shear test and shear wave velocity measurements in soils. There will also be a discussion about an approach of quantifying undrained shear strength called Stress History and Normalized Soil Engineering Properties (SHANSEP), and published data relating the small strain shear modulus to the undrained shear strength.

Chapter 3 details the development of the shear wave velocity measurement system and the laboratory testing program.

Chapter 4 presents the results of the laboratory testing program including a comparison with data from the literature.

Chapter 5 summarizes the results of this study and presents recommendations for future work.

2. Review of Literature

This chapter presents a review of the literature related to a possible link between the small strain shear modulus and undrained shear strength of clay. A review of undrained shear strength is presented first with particular emphasis on its measurement using the direct simple shear apparatus. The measurement of the small strain shear modulus from shear wave velocity is then described, followed by details affecting shear wave velocity measurements in the laboratory. The chapter ends with published data relating the small strain shear modulus to the undrained shear strength of different soils.

2.1 Undrained Shear Strength

The undrained shear strength is a measurement of a soils' strength under loading conditions in which drainage is not allowed. It is most often used for short term loading conditions of cohesive soil. Cohesive soils have a low hydraulic conductivity, which prevents rapid drainage during most loading situations encountered in engineering practice.

The shear strength of soils is generally described using the Mohr-Coulomb failure criterion as expressed by:

$$\tau = \sigma \tan (\varphi) + c \quad (2)$$

where τ = shear strength (max shear stress at failure)
 c = cohesive strength of a material
 σ = applied normal stress on a material
 φ = internal friction angle of a material

For a saturated soil under undrained loading conditions, the strength is independent of the total stress and the failure envelope. The failure envelope can be described as a horizontal line or at an angle of 0 ($\phi=0$) as shown in Figure 2.1.

$$\tau = c = S_u$$

Figure 2.1 Mohr-Coulomb failure envelope for a cohesive material with the undrained shear strength equal to the max shear stress (Holtz et al., 2011).

The shear strength can be measured in-situ and in a laboratory setting. *In-situ* methods include the field vane (direct measurement), and from correlations with the cone penetration test and the standard penetration test. Laboratory tests can include simple tests such as mini-vane shear, fall cone and torvane, or more advanced tests such as the unconsolidated undrained triaxial compression and extension tests, and the direct simple shear test (DSS). The direct simple shear test is the preferred method since it can replicate the loading and unloading conditions similar to realistic field conditions and yields an average shear strength between strengths in compression and extension. Triaxial compression is the most commonly used due to the widespread availability of triaxial equipment in geotechnical laboratories. However, each test loads the soil in a different manner as illustrated below in Figure 2.2.

Figure 2.2 Application of different soil's tests to determine the undrained shear strength (adapted from Budhu, 1999).

Of the different methods used to determine the undrained shear strength of soils, the direct simple shear is the best method to obtain the most accurate results due to the similarities between it and *in-situ* loading conditions. When compared to the values of strength obtained from compression and extension tests, Ladd and Degroot (2003) found undrained shear strength from the DSS test to be the average of all three methods as shown in Figure 2.3.

Figure 2.3 Comparison of different ways to obtain the undrained shear strength (Ladd and DeGroot, 2003).

2.1.1 Undrained Shear Strength from a Direct Simple Shear Test

The direct simple shear test involves two loading phases. The first phase involves the application of a vertical stress and consolidation of the sample under K_0 conditions, also known as at-rest conditions. The second phase involves the application of shear stress on a horizontal plane until failure (see Figure 2.4). The undrained shear strength is defined as the maximum measured applied shear stress.

Figure 2.4 a.) Consolidation and b.) shear phases in a DSS test (Holtz et al., 2011).

ASTM D 6528, the Standard Test Method for Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils, gives the basic guidelines on conducting a direct simple shear (DSS) test. The following is taken directly from ASTM D 6528 and highlights the most significant aspects of the DSS test:

1. "In this test method a specimen of cohesive soil is constrained axially between two parallel, rigid platens and laterally, such that the cross sectional area remains constant."
2. "The specimen is loaded axially and allowed to consolidate one-dimensionally. Each normal load increment is maintained until excess pore water pressures are essentially dissipated as interpreted from the axial displacement rate. The maximum normal load is maintained until completion of one cycle of secondary compression or one day longer than the end of excess pore water pressure dissipation."
3. "The specimen is sheared by displacing one platen tangentially relative to the other at a constant rate of displacement and measuring the resulting shear force. The platens are constrained against rotation and axial movement throughout shear."
4. "The specimen volume is held constant during shear to simulate undrained conditions. Constant volume is achieved by changing the normal load applied to the specimen to maintain constant specimen height. Since the pore pressure is zero through shear, the change in normal stress is equal to the change in effective stress and assumed to be equal to the change in pore water pressure that would occur in a sealed specimen confined by a constant total stress."

Item 4 refers to how the undrained loading condition is maintained during shear. This is accomplished by maintaining a constant height of the specimen, either by actively adjusting the vertical stress during shear or locking the vertical load frame. Active load control is recommended, however the vertical load frame can be locked provided the sample does not change in height more than an axial strain of 0.05% during the shear phase. This is referred to as Passive Height Control, and is used in cases where the active control systems are not efficient enough to produce smooth loading results.

ASTM D 6528 gives a basic schematic of the apparatus in Figure 2.5. The apparatus has been developed over the years and is commercially available from approximately five companies in a variety of designs. The equipment used in this study was manufactured by the Geocomp Corp. and will be described in more detail in Chapter 3.

Figure 2.5 Basic schematic of DSS equipment with essential components as described in ASTM D6528.

Key aspects of all DSS apparatus include the following:

- a vertical loading system that is rigid enough to prevent rocking of the top cap/platen during the shear phase;
- a horizontal shear measurement system that does not transfer moments to a horizontal load cell;
- a lateral confinement system that allows for samples to be consolidated under K_o (or close to) conditions.

There are two types of lateral confinement methods: metal stacked rings or wire reinforced rubber membranes. The metal rings are becoming more commonly used in practice due to their durability and low production cost. Wire reinforced membranes are produced by the Norwegian Geotechnical Institute, and are rubber membranes with a wire wrapped along the outside of the membrane. Application of both confinement methods are shown in Figure 2.6.

Figure 2.6 Confinement methods used in the direct simple shear test: a) wire reinforced membrane and b) stacked rings (Baxter et al., 2010).

Research by Baxter et al. (2010) and McGuire (2011) has shown that both confinement methods yield comparable values of undrained shear strength provided appropriate shear stress corrections are applied. McGuire (2011) performed simple shear tests on water filled membranes at pressures of 5, 7 and 10 kPa to determine the influence of confinement system on measured shear strength. These results are shown in Figure 2.7, and show that for the pressures tested the stacked rings add only about 1 kPa to the measured shear strength (see Figure 2.8).

Figure 2.7 The equation of the best fit line to the 7kPa test was $t=1.23g + 0.53$ (McGuire 2011).

Figure 2.8 Corrected vs. uncorrected results of Narragansett Bay Silt conducted by McGuire (2011).

2.2 Small Strain Shear Modulus

The shear modulus is the slope of the shear stress-shear strain relationship for a material, as illustrated in Figure 2.9. The stress-strain behavior of soils is highly non-linear and the modulus degrades with increasing strain. The initial tangent shear modulus is referred to as G_o or G_{max} , and this is defined as the small strain shear modulus.

Figure 2.9 Shear strain vs. shear stress response curve (adapted from Budhu, 1999).

At very low strains, typically below 0.001%, the behavior is considered to be elastic, and the soil's behavior is linear. At larger strains, there is significant degradation of the modulus (see Figure 2.10).

Figure 2.10 Typical variations of different moduli with strain for soils. G=shear modulus, K'=bulk modulus and E'=elastic modulus (Budhu, 1999).

One of the objectives of this thesis is to determine the small strain shear modulus, G_o , of different clays and relate these values to the undrained shear strength. There are numerous ways to estimate or measure this modulus, including measuring it directly in a resonant column device or with internal load and deformation measurements in a triaxial apparatus. The small strain shear modulus (G_o) can also be calculated from the soils' shear wave velocity (v_s) directly from elastic theory as

$$G_o = \rho V_s^2$$

where ρ is the bulk density.

This approach is increasingly being used in geotechnical research and practice as shear wave velocity measurements can be incorporated into many typically soil testing devices (Kramer, 1996) as will be discussed in the next section.

2.3 Shear Wave Velocity

There are two different types of body waves that can travel through an elastic medium: longitudinal and transverse waves.

Longitudinal waves, also known as compression or p-waves, are waves whose direction of propagation and direction of particle movement are the same. This is illustrated in Figure 2.11 (b). In saturated soils these waves travel primarily through the pore fluid, and the compression wave velocity is relatively insensitive to the properties of the soil.

Transverse waves, also known as shear or s-waves, are waves whose particle motion is perpendicular to the direction of propagation. This is illustrated in Figure 2.11 (c).

Figure 2.11 Wave propagation through an elastic medium (Santamarina, 2001).

These waves travel primarily through solid matter, and not through voids or water. As such the shear wave velocity in soils is influenced greatly by the behavior at the particle contacts, and is a function of effective stress, void ratio, and fabric. This strong dependency on soil properties and its relationship to the small strain shear modulus, and the fact that it is a non-destructive measurement makes the shear wave velocity an attractive property to measure in soils.

2.3.1 Piezoelectric Ceramics (*Bender Elements*)

Piezoelectric ceramic is a material that will mechanically or physically deform when electricity is passed through it or will generate an electrical signal when deformed. These ceramics are typically made of lead zirconate titanate, barium titanate or lead titanate, which are artificially manufactured. Naturally occurring piezoelectric materials include quartz and tourmaline crystals (Brignoli et al., 1996).

Compression waves can be generated by producing a voltage drop parallel to the polarization of the metal component, as shown in Figure 2.12, which creates a change in thickness of the ceramic element.

Figure 2.12 Application of a voltage across a piezoceramic to cause thickening of the element (Piezo Systems).

In order to generate and measure shear waves, two piezoelectric ceramics that have opposite polarization are sandwiched between a metal shim. When a voltage is passed

across the element, one side lengthens and the other shortens, causing a bending motion. Elements in this configuration are called “bender elements” and are well suited for generating and measuring shear waves (see Figure 2.13).

Wiring the metal component in parallel will cause the element to generate more bending for a given voltage, as illustrated in Figure 2-14. This configuration is typically used as a transmitter of shear waves.

Figure 2.13 Bender elements wired in series and parallel (adapted from Piezo Systems).

Wiring the metal component in series will cause the element to generate a larger electrical signal for a given mechanical movement. Bender elements wired in series are typically used as receivers.

Figure 2.14 shows a diagram of a basic setup with a soil sample with bender elements embedded in the end caps.

Figure 2.14 Soil testing apparatus with bender elements installed in the end caps for the generation and measurement of shear waves (Landon et al., 2007).

Typical transmitted and received signals from bender elements are shown in Figure 2.15. The difference in time is measured to determine the velocity of the wave over a known distance.

Figure 2.15 Example of input and output readings of shear waves generated by and received from bender elements (Brignoli et al., 1996).

The transmitting and receiving bender elements must be in direct contact with the soil. This explains why at low consolidation stresses, the shear wave is hard to determine due to the lack of coupling between benders and the soil. Figure 2-16(a) illustrates another error due to the fact that the transducer will send both P and S waves, although only one wave is desired. By having the second wave, the desired wave could be misinterpreted due to the effects of the other wave's presence, which will bounce off the sides of the containment device as shown in Figure 2-16(b) and 2-16(c) where the desired P waves could be misconstrued as S wave.

Figure 2.16 Propagation of S and P waves from the same source (Lee & Santamarina, 2005).

2.3.3 Understanding Shear Wave Results

Interpretation of bender element data to determine the shear wave velocity as shown in Figure 2.15 can be quite difficult. This is because interpretation of the shear wave can be affected by the wave form generated by the transmitter, frequency of the

signal, separation distance and the applied voltage. Each of these factors will be described below.

Generated Wave

Researchers have studied different wave forms to generate a shear wave in soils. These include single square waves and sinusoidal waves to multiple bursts. One particular study conducted by Leong et al. (2005) compared the shear wave of three different soil specimens: sand, mudstone residual soil and kaolin. In Figure 2.17, the results illustrate the differences between the square and sinusoidal waves. From the square waves, there is no representative received signal of the originally transmitted signal unlike the sinusoidal waves. The shape of the received square wave diminishes from the flat portions to the sharp drops, which characterizes the wave. The square waves also show more distortion compared to the sinusoidal waves. These tests were conducted using the same bender elements, so any error caused by other sources or materials would be seen in both plots. This distortion is the direct result of using the square waves.

Note: t_1 = first deflection, t_2 = first reversal

Figure 2.17 Wave comparison between square and sinusoidal conducted by Leong et al. (2005).

Frequency

Lee and Santamarina (2005) conducted a similar study to Leong et al. (2005) evaluating both the type of generated wave and the frequency content of the transmitted signal. Lee and Santamarina determined that the best received signal came prior to the resonant frequency. At larger frequencies, the received signal is distorted, but as the frequency is lowered, the signal appears better until it hits the resonant frequency of the bender elements (see Figure 2.18).

Figure 2.18 Comparison of different wave forms and frequencies with the resonant frequency being determined to be 3.6 kHz by Lee and Santamarina (2005).

Separation Distance/Traveled Distance (Near Field Effects)

The distance traveled also hampers the received signal of the bender elements. The distance is measured from the tip to tip of each bender element. The greater the distance between transmitter and receiver, the more attenuation of the received signal occurs. The basic example of this is a ripple of water that gets smaller as it gets further away from the wave source, which is analogous to the received signal diminishing as the tip to tip distance increases (Wang et al., 2007).

Conversely, if the bender elements are too close, near field effects will also hamper the received signal. Most experiments in testing for the small strain shear modulus have been utilizing triaxial setups, which are 66mm or higher in height compared to direct simple shear samples, which are only at 25 mm in height (ASTM D4767 and ASTM D6528). Brignoli et al. (1996) explored the near field effect with a

soft clay soil specimen, which concluded with Figure 2.19. The test was conducted on a 10 cm high sample with two transducers sandwiching the sample using sinusoidal waves. As the wavelength was decreased by decreasing the frequency the near field effect became more apparent.

Figure 2.19 Example of near field effects by Brignoli et al. (1996).

Interpreting the Arrival of the Shear Wave

Interpreting the first arrival of the shear wave velocity signal from the bender elements can be very subjective. Researchers have discussed different ways to obtain the most accurate shear wave velocity measurements from detection of the first arrival.

Some of the methods include peak to peak, zero crossing and first take off. Lee and Santamarina (2005) illustrate in Figure 2.20 the differences between the approaches.

Figure 2.20 An illustration from Lee and Santamarina (2005) concerning the different approaches to determining the first arrival of the signal.

- A) The first deflection
- B) The first bump at the max of the curvature
- C) The first zero crossing
- D) The first major peak or maximum

Nakagawa et al. (1996) determined that the peak to peak option was the best, because there are multiple reflections following the first arrival of the signal in Figure 2.21. If these reflections are equally spaced, then they will not be influenced by the near field effects like the other options. By accurately measuring the time difference between the two peaks the signal is already taking into account for the intrusion of the bender element.

Figure 2.21 Nakagawa et al. (1996) comparing the equal time intervals between reflected P and S waves.

Placing bender elements in saturated clay is going to introduce another phenomenon called “crosstalk,” which will distort the received signal. Due to the induced electromagnetic field caused by the bender elements, the received bender element will register a “crosstalk” signal that can be misunderstood for the real signal as shown below in Figure 2.22. This electromagnetic field essentially is a ground, which energizes the “crosstalk” signal. To combat this phenomenon, Lee (2005) and Deniz (2008) suggest using a polyurethane coat for the all bender elements and additionally using a conductive paint for the series bender elements. However, the University of Rhode Island Marine Geomechanics Laboratory (MGL) has already experimented with using polyurethane and conductive coating systems with limited success. The has determined through trial and error that the most effective method is to use 3M Scotchcast 5 Resin followed by a few thin layers of polyurethane (Hanchar, 2006). This resin is a two part electrical epoxy that is considered to be rigid once cured,

which makes it ideal for encapsulating the bender elements (3M Company). With the combination of the resin and the polyurethane any induced crosstalk should appear smaller in size compared to the desired signal.

Figure 2.22 A received signal with crosstalk compared to a signal without cross talk (Lee and Santamarina 2005).

2.4 Link Between the G_0 And S_u

The undrained shear strength is a difficult property to determine accurately in the field and even in a laboratory with controlled settings. Approaches have been developed to try to assess the in situ undrained shear strength from knowledge of the soils' stress history and values of strength from reconstituted samples. This approach is termed SHANSEP and is described in the next section.

2.4.1 Stress History and Normalized Soil Engineering Properties (SHANSEP)

SHANSEP is a way to determine the undrained shear strength by knowing the stress history of the soil (Ladd and Foott, 1974). Using this approach the undrained shear strength ratio is defined as:

$$S_u/\sigma'_{vc} = (S_u/\sigma'_{vc})_{OCR=1} * OCR^m$$

S_u = Undrained Shear Strength

σ'_{vc} = Vertical Effective Consolidation Stress

$(S_u/\sigma'_{vc})_{OCR=1}$ = Undrained Shear Strength Ratio for Normally Consolidated Condition

$OCR = \sigma'_p / \sigma'_v$ = Overconsolidation Ratio

m = Slope of the SHANSEP curve

Ladd and Foott (1974) used Figure 2.23, to illustrate how the undrained shear strength can be determined by knowing the OCR and the vertical effective stress of the soil.

Figure 2.23 Variation of undrained shear strength as a function of OCR for different clays (Ladd and Foott 1974).

2.4.2 Link Between Small Strain Shear Modulus and Undrained Shear Strength

Analogous to the SHANSEP approach, Anderson (1991) normalized the small strain shear modulus by the undrained shear strength for 11 different clays with varying plasticity indices. Figure 2.24 shows that, as the plasticity index increases, the ratio of G_o/S_u decreases. This method could potentially be a useful method in determining the undrained shear strength because the shear wave velocity can be determined *in-situ*.

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3. Laboratory Testing Program

The laboratory testing program for this study included the design and fabrication of a shear wave measurement system in the Direct Simple Shear apparatus, validation of the V_s measurements, and development of a testing matrix. The work was accomplished at the Marine Geomechanics Laboratory (MGL) at the University of Rhode Island. The objective was to develop a shear wave velocity measurement system in a DSS apparatus to be able to link small strain (G_o) and large strain (S_u) soil properties. This chapter discusses the description of the DSS apparatus, the design and fabrication of the bender elements, validation of the bender end caps, sample preparation, properties of the tested soil, and the testing matrix of this study.

3.1 Direct Simple Shear Apparatus

The undrained shear strength was determined using a direct simple shear (DSS) apparatus manufactured by the Geocomp Corporation, as shown in Figure 3.1.

Figure 3.1 Shear Trac II system for performing DSS tests to determine the undrained shear strength (Geocomp, 2009).

The Shear Trac-II system is a fully automated system that uses a proportional integral derivative (PID) closed loop system to control the application of both loads and displacements. Four channels of data are recorded: horizontal and vertical displacement, and horizontal and vertical load. Table 3.1 provides specifics of each of the components, and Figure 3.2 shows their location on the apparatus.

Table 3.1 Description of the load cells and displacement transducers located on the Geocomp Corp. DSS apparatus.

	Horizontal	Vertical
Load Cell Capacity & Model Number	1,000 lb., SML-1000	1,000 lb., SBA-1KLB-I
LVDT Model Number	TR 75	TR 50

Figure 3.2 Location of displacement transducers and load cells (adapted from Geocomp 2009).

The user can keep track of each loading phase, consolidation or shear, and obtain the results concurrently throughout the test. From the data, the compressibility, shear strength and stress-strain behavior can be assessed.

The Geocomp system uses Teflon-coated rings and a rubber membrane to provide lateral confinement of the soil sample. These are shown in Figure 3.3.

Figure 3.3 Rubber membrane and Teflon rings utilized during this research. The inside diameter of the rings are 63.5mm and the thickness of the membrane is 2mm.

3.2 Shear Wave Velocity Measurement System

The shear wave velocity measurement system was constructed specifically for this study for inclusion in the DSS equipment. There is considerable experience in the measurement of the shear wave velocity of soils at URI (Baxter 1999; Baxter and Mitchell 2004; Bradshaw 2006; Hanchar 2006; Baxter et al. 2008; Sharma 2010) and this experience was used to design a simple and efficient system. Figure 3.4 shows the electronics used in this study (function generator, amplifier, and an oscilloscope).

Figure 3.4 Oscilloscope, function generator and amplifier used for this research.

3.2.1 Bender Element End Cap Construction

The bender elements used for this study were manufactured by Piezosystems, Inc. of Cambridge, MA. Table 3.2 summarizes the material properties of the bender

elements and Figure 3.5 shows their dimensions. The 303 bending actuator is a standard item from Piezosystems, Inc. and comes with a wiring bracket attached. These elements are optimal since they are only 41 mm in length when the DSS apparatus only offers 120 mm for two end caps and a 19 mm thick base. The inclusion of a standard quick-mount bracket offers additional protection by increasing the stiffness for the electrical connections, decreasing electrical noise by proper insulation, and by having a bleed resistor for the unpredictable electrical spikes (Piezo Systems Inc., 2008).

Table 3.2 Material properties of the bender elements (Piezo Systems Inc).

Part Number	Q220-A4-303YB
Piezo Material	5A4E
Weight (g)	2.3
Stiffness (N/m)	760
Capacitance (nF)	52
Rated Voltage (V)	± 90
Resonant Frequency (Hz)	275
Free Deflection (μm)	± 315
Blocked Force (N)	$\pm .24$

-303 Size Quick-Mount Dimensions

**Figure 3.5 Dimensions of the 303 bender actuator in inches (mm in parentheses)
(Piezo Systems Inc.).**

The receiving bender element was wired in series and the transmitting bender element was wired in parallel as suggested by the manufacturer (Piezo Systems, Inc 2008). Because the parallel connection does not split the voltage, both of the ceramic plates are influenced by the same amount of voltage, which makes it ideal as a transmitter. However, because the series connection is influenced at the same time across both ceramic layers, it is better suited as a receiver. Other researchers have successfully used bender elements wired in parallel for transmitting shear waves and in series for receiving shear waves (Hanchar, 2006; Landon, 2007; Deniz, 2008).

In order to transmit and receive signals, the bender element's factory wiring was removed and a Belden 8240 R6 58/U coaxial cable was attached using solder. This allowed the connection between the signal generator and receiver to the bender elements to be as clean as possible and minimize possible magnetic field interference.

On each end of the Belden coaxial cables are factory installed BNC connectors, which allowed for a clean connection to the equipment and the bender elements. Previous setups without BNC connectors forced the use of alligator clips or similar clip-on attachments to connect the wires to the signal generator or receiver.

The use of bender elements with the quick-mount attachments and the coaxial cable with BNC connectors provided a clean signal with reduced electrical noise.

3.3 Bender Element End Cap Design and Fabrication

The design and the fabrication of the DSS end caps containing the bender elements was governed primarily by the location of the bender elements and the height constraint of the DSS apparatus. In addition, the end caps were designed to carry up to 454 kg., be easily portable for one person, provide easy access to the benders for when they needed to be replaced, and have limited cross talk.

The selection of material for the end caps was based on corrosion resistance, modulus of elasticity and electrical conductivity. However, only PVC, steel and brass was considered. PVC is nonconductive, so it was not used despite the fact that it would have been ideal for corrosion resistance. A36 Steel would have worked from a conductivity standpoint, but it would have corroded since salt is present in the soil samples. Brass is relatively easy to machine, it is conductive and yet it still has the structural integrity for the required loads. For these reasons, brass was utilized for the bender element end caps.

The designs of the top and bottom end caps are shown in Figure 3.6 and 3.7. Since the DSS has a height restriction of 120 mm and the benders needed to be able to be fully enclosed within the end cap, 38 mm of height was left to work with. The top cap has a 9.7 mm hole in the center that can be tightly screwed onto the vertical rod, two drainage holes, and a slot for the bender element.

Figure 3.6 Final design of the top bender end cap.

The bottom end cap is not as tall as the top end cap because it is mounted on a PVC base. It also has drainage holes and a center screw hole for the bottom end cap to be rigidly connected to the PVC base.

Figure 3.7 Final design for the bottom bender element.

The PVC base is shown in Figure 3.8. It is made of two parts to provide a steady platform for the Teflon rings and the bottom end cap. The first part was a square piece of PVC to allow the bottom element to be securely fastened to the DSS equipment. It had drainage channels to allow pore pressure to dissipate and allow wiring to travel to and from the bender element without causing a perpendicular angle in the wire or causing the base not to be level. The second PVC piece was circular and was secured to the bottom PVC piece via adhesive and screws. This piece carries the weight of the Teflon rings and allows them to shift with the soil sample without causing significant frictional resistance.

Figure 3.8 Final design of the bottom base.

The use of stacked rings in the DSS creates some challenges for sample preparation and mounting of undisturbed soil samples. In particular, there is no easy way to stretch a membrane around a trimmed soil sample. The solution for this study was to fabricate a specialized split-mold housing based designed by Dr. Ravi Sharma of URI

that covers the stacked rings. This is shown in Figure 3.9. The split mold is sealed against the bottom end cap and a vacuum is applied through the stacked rings to stretch the membrane. The split mold is removed once the top cap has been carefully placed on the sample.

Figure 3.9 Split PVC mold to guide the top end cap into place onto the soil sample.

3.3.1 Installation of the Bender Elements

The bender elements were recessed into the end caps so that they protruded a few millimeters into the sample. The adhesive used to coat the bender elements and secure them in the brass end caps was 3M Scotchcast 5 Resin followed by a few thin layers of polyurethane (Hanchar 2006). To set the epoxy and create a uniform coating around the bender element, a mold was made to hold the bender in place from the wire end and to ensure the brass was not in contact with the bender element. Another mold was needed to pour the epoxy around the bender element and to keep it contained within the particular area of the end cap (see Figure 3.10). This epoxy set up was constructed within an hour and generates considerable heat (3M Company). Once the

epoxy cured a soldering iron was used to trim the excess epoxy into a nice box shape around the bender element.

Figure 3.10 The mold for the epoxy being cured to set the bender element.

The 3M Scotchcast 5 Resin has several advantages. This resin is an electrical epoxy and is considered to be rigid and fully insulating. Any stray current cannot jump from the bender element to the brass cap and vice versa. This will decrease the chances of the signal being altered or disturbed by electrical noise or grounding problems. Also, if the bender element needs to be replaced, it can be removed easily without damaging the end cap or the ceramic itself. By applying heat to the brass end caps, the bond between the end caps and epoxy will decrease and the elements can be easily removed.

3.3.2 Porous Stone Fabrication

In each of the end caps, a porous stone was fabricated around the bender elements. The porous stones needed to be free draining and provide a rough surface to resist sliding during horizontal shear. Prior to placing the porous stone in the end

caps, nine samples with different ratios of adhesive-to-sand mixtures were evaluated to find a proper mixture.

Since there is no standard for fabrication of porous stones, a small experiment was performed to learn how to place and cure a wet mixture of sand and some form of adhesive. Samples of Ottawa sand were mixed with different amounts of HUNTSMAN's Araldite AY103 and Resin 956. Each sample weighed at least 11 to 17 grams, and was placed in a PVC ring and allowed to cure, as illustrated in Figure 3.11.

Figure 3.11 Porous stone samples.

The final results showed that, out of the nine samples, only two failed in that air could not be blown through the mixture. The ratio of epoxy to sand was not the determining factor for success. The lesson learned from these experiments was to use wax paper and some weights to sandwich the sand and epoxy mixture together. This allowed the mixture to be evenly distributed and forced the epoxy within the voids of the sand, which slowed the rate of descent due to gravity before curing. Without the applied force caused by the weights the epoxy collected on the bottom and created a solid barrier that did not allow air to pass through the sample.

In order to place the porous stone successfully within the end caps, the method of application was the same as the porous stone samples. However, some modifications were needed because of the presence of the bender elements. Once the

3M Electrical Resin 5 had cured to keep the bender elements in place, small pieces of geo grid were placed over the drainage holes to keep them open to ensure that the porous stone mixture would not be allowed to collect and clog the holes. A mixture of sand and epoxy weighing approximately 50g was placed on each of the caps surrounding the bender elements. A knife was then used to ensure that the mixture was flush with each of the sides of the end caps which ensured the porous stone would only be 3 mm thick. Wax paper was then placed on top of the porous stone followed by dry sand and weights. Because of the fragility of the bender elements, sand was placed on the wax paper to give an even applied force distribution as seen in Figure 3.12, which did apply a pressure directly on the bender element.

Figure 3.12 Applying an even force onto the porous stone.

Figure 3.13 shows the completed bottom plate and top and bottom end caps used in this study.

Figure 3.13 Bender element end caps.

3.4 Validation of the Shear Wave Velocity Measurement System

In order to evaluate whether the new DSS end caps with bender elements performed properly, a test was performed on a sample of Ottawa sand and the results were compared to published values from the literature. The time delay was determined to be 6 microseconds, and a dry sample was prepared to a void ratio of 0.63 and loaded under K_0 conditions to vertical stresses of 40, 60, 80, 100 and 120 kPa. At each consolidation stress the shear wave velocity was measured. The results are shown in Figure 3.14 along with published values measured by Hanchar (2006). The agreement with published data is good enough to conclude that the new apparatus is working properly.

Figure 3.14 Comparison between measured shear wave velocities using the new DSS end caps and published data (adapted from Hanchar 2006).

3.5 Sample Preparation

The sample preparation was conducted in a way to limit sample disturbance from handling and trimming. The majority of the tests performed in this study were from 1.5m long, 10cm diameter cores of clay from the Gulf of Mexico (Silva et al., 2003), and this section will detail how these samples were prepared. For each test a 76 mm long section was cut away from the 1.5m long core. This was done using a large band saw. Once cut, the remaining core was placed back in the storage refrigerator. The soil was then debonded from the 76mm long sub-sample using the approach recommended by Ladd and Degroot (2003). A small diameter tube containing a piano wire was pushed into the soil along the inside wall of the PVC liner. The sub-section was then held tight using a chain vice (see Figure 3.15) and the tube was removed,

leaving the piano wire. The wire was gripped with pliers and dragged around the inside of the liner a few times breaking the bond between the soil the PVC.

Figure 3.15 Piano wire trims the inner diameter of the PVC liner to break bond with soil specimen.

The soil was then extruded from the tube by placing the soil on a lubricated base and pushing the tube down by hand. This is shown in Figure 3.16.

Figure 3.16 76mm soil specimen removed from the PVC liner.

The sample was then rough-cut from 102mm diameter to approximately 76mm using a cutting wire and placed on a trimming stand as shown in Figure 3.17. The sample was then trimmed to 71mm diameter in preparation for placement of a cutting ring.

Figure 3.17 Soil sample being trimmed to 71mm.

The cutting ring is 63 mm in diameter and was fixed at the end of the top plate of the trimming device and pushed evenly into the sample. The cutting ring was lubricated with mineral oil to minimize side friction during trimming. As the rod is pushed down in small increments (5mm), excess soil was removed to prevent the formation of cracks inside the sample. Figure 3.18 shows excess soil being removed as the rod and cutting ring is pushed down.

Figure 3.18 Excess soil being removed as the cutting ring is pushed through the sample.

Once the cutting ring cut 51 mm into the sample, the sample with the cutting ring was placed on another stand to finish the trimming process.

The final stage of the trimming process reduces the sample's height from 76 mm to 25.4 mm. After being placed on the height trimming stand, the top of the cutting ring would be trimmed to make it flush with the cutting ring as illustrated in Figure 3.19 and Figure 3.20. This trimming was used for a water content measurement.

Figure 3.19 Top of soil sample being trimmed flush to the cutting ring.

Figure 3.20 Top of soil sample is flush with the cutting ring.

Next, a locking ring was lowered and attached to the cutting ring to prevent any horizontal and vertical movement. The sample was then flipped and the bottom of the sample was cut flush with the cutting ring as shown in Figure 3.21 and Figure 3.22. A second water content measurement was made from these trimmings.

Figure 3.21 The cutting ring is flipped to trim the bottom of the sample flush with the ring.

Figure 3.22 Bottom of soil sample is trimmed flush with the cutting ring.

The soil sample was flipped and a 6.4 mm thick porous stone covered with wax paper was placed on top of the cutting ring and pushed down to set the height of the soil sample at 25.4 mm as shown in Figure 3.23 and Figure 2.24. This enables the soil sample to be the same height for every test and wax paper prevents the soil sticking to the porous stone.

Figure 3.23 Porous stone being placed from the top of the sample.

Figure 3.24 Porous stone is flush with the top of the cutting ring.

The sample was flipped again and placed bottom up to enable the final trim from the bottom of the cutting ring as illustrated in Figure 3.25 and Figure 3.26. The cutting ring is placed directly on the stand to ensure that the height of the soil sample is 25.4 mm.

Figure 3.25 The soil sample is ready for the final trim.

Figure 3.26 The bottom of the soil sample has been trimmed flush with the cutting ring.

3.5.1 Extruding The Sample Onto the Bottom End Cap

The end caps and bender elements are cleaned prior to each test. Filter paper is cut to the exact dimensions of the end caps with holes for the bender elements to stick through. The filter paper was saturated with water to ensure the moisture content of the sample remains relatively constant as it is being sandwiched between the end caps.

With both of the end caps ready to be set in place, the cutting ring is removed from the locking ring and a membrane is placed in a membrane stretcher. The soil sample is placed on the bottom end cap and the same is extruded from the cutting ring by pushing lightly on the porous stone. The final sample is shown in Figure 3.27.

Figure 3.27 The soil sample is pushed out from the top onto the bottom element end cap.

The membrane is placed around the soil sample as shown in Figure 3.28 and Figure 3.29. This allowed the membrane to be placed around the trimmed sample without being handled or disturbed by fingers, or cause the sample to be lifted off the bottom end cap.

Figure 3.28 The membrane stretcher is about to be placed around the soil sample.

Figure 3.29 The membrane stretcher is removed leaving the membrane around the soil sample.

3.5.2 Assembling the Sample in the DSS Apparatus

Once the membrane is placed around the sample, the bottom end cap, base and soil sample is then moved and placed into the shear box of the DSS equipment. The base is securely fastened to the shear box and the stacked rings are carefully placed around the sample as shown in Figure 3.30.

Figure 3.30 Placing the Teflon rings around the soil sample.

The PVC split mold is then placed around the rings to allow the membrane to be stretched again for the top end cap to slide onto the soil sample as shown in Figure 3.31. The split PVC mold is then removed, and any loose hardware would be tightened prior to testing (Figure 3.32).

Figure 3.31 The top end cap is lowered onto the sample after the PVC split mold is attached and sealed with electrical tape.

Figure 3.32 The split mold is removed and the sample ready to be sheared.

3.6 Properties of Soil Tested

Three soils were tested as part of this study: a high plasticity clay from the Gulf of Mexico, a sensitive clay from Maine called Presumpscot clay, and an organic silt from Narragansett Bay, Rhode Island. The Gulf of Mexico clay was from Jumbo Piston Cores collected by researchers at URI in 1998 (Silva et al., 2003). Samples of the Presumpscot clay came from a high quality block samples collected in Falmouth, Maine by Dr. Landon Maynard's lab from the University of Maine (Langlais, 2011). The organic silt was obtained as part of a dredging study using gravity cores and samples were reconstituted in the laboratory. A summary of relevant soil properties for the three soils are show in Table 3.3.

Table 3.3 Summary of soil properties.

Soil	USCS ¹ Classification	Plastic ² Limit	Liquid ² Limit	Plasticity ² Index	Natural Water Content	Coeff. of Consolidation (cm ² /sec)	Specific Gravity	Specific ³ Gravity	References
Gulf of Mexico	CH	34	75	41	68%	0.005	2.68	2.72	Bradshaw, 1999 Brausse, 2001 Silva et al., 2003
Presumpscot Clay	CL	25	57	32	50%	0.004	2.70 to 2.79	2.72	Langlais, 2011
Organic Silt	ML	23	30	7	NA	NA	2.70	2.70	Baxter et al., 2004

1. ASTM D2487

2. ASTM D4318

3. ASTM D854

The coefficient of consolidation for the Gulf of Mexico clay was estimated to be 0.005 cm²/s from the work of Brausse (2001). This estimate was used to set a minimum time of 45 minutes for consolidation for each load increment. To ensure the samples were at equilibrium before shear, all samples were consolidated for a minimum of 2 hours before shear.

Plasticity Indices determined in this study were compared to the literature for accuracy. Bradshaw (1999) reported the Plasticity Index of Gulf of Mexico clay at the same depth to be 46, which is comparable to the results in Table 3.3. For the Presumpscot clay, the plasticity index was 32. For the organic silt, a plasticity index

was 7 with a liquid limit of 30 and a plastic limit of 23. All of these values are within the range of values measured in this study.

3.7 Details of the Laboratory Testing Program

This section provides additional details regarding the testing program. These include load control settings in the software of the Geocomp system, the strain rate for the shear phase, the load increment ratio for the consolidation phase and the consolidation stress.

3.7.1 PID Settings

The load control system used by the Geocomp equipment relies on Proportional Integral Derivative (PID) control theory. Understanding how the PID settings in the software affect the loading is critical for getting good test results.

A PID setting is a closed loop feedback system within the software that controls the equipment by going through the loop every 250 msec. PID is essentially a methodology for how a target load is reached automatically. Based on what a sensor reads relative to a target value, the program will instruct the equipment to do a certain action to reduce the difference.

Recommended settings for the P-Gain for different soil types are shown in Table 3.4. Screenshots of the PID setting's window from the software showing the values used in this study for the vertical and horizontal loads is shown in Figure 3.33.

Table 3.4 Geocomp recommended settings depending on the soil (Geocomp, 2007)

Sample stiffness	Recommended P-Gain value for actuator
LOOSE /VERY SOFT	0.5
MEDIUM SOFT/MEDIUM	2.0
DENSE/STIFF	5.0

Figure 3.33 PID settings for each of the DSS tests.

An important aspect of DSS testing is that the height of the specimens had to remain constant during shear to maintain constant volume/undrained conditions. To achieve this, the vertical load on the sample has to decrease or increase during shear, depending on the volume change characteristics of the sample. This is accomplished in two ways with the Geocomp system: active and passive height control. Active control means that the vertical motor is continually adjusted to maintain a constant height and the load is recorded. For this to work properly, the PID settings must be tuned for the particular soil. Using the vertical settings shown in Figure 3.33 resulted in a “staircase” vertical load/pore pressure response. An example of this is shown in Figure 3.34.

Figure 3.34 Example of using active height control.

This was deemed unsuitable and passive height control was used instead. In this approach, the vertical motor is locked in place during shear by decreasing the velocity limit to zero (Figure 3.34), and recording the change in load. ASTM D6528 allows for passive height control provided the change in height does not exceed 0.05% during shear. Passive height control was used successfully for all the tests in this study.

3.7.2 Stress Levels and Strain Rates

Due to the high plasticity of the soil samples, a load incremental ratio (LIR) of 0.5 was chosen during the consolidation phase starting at a vertical stress of 20 kPa and ending at 200 kPa. According to AASHTO T216, an LIR of less than 1 might be desirable for highly compressible samples. In addition, decreasing the LIR allows for clearer interpretation of the preconsolidation stress.

Each sample of Gulf of Mexico clay was first consolidated to 200 kPa to ensure that the samples were normally consolidated and that any effects of fabric and structure were removed, and then unloaded to different values of overconsolidation ratios. Samples were sheared at different vertical stresses depending on the desired

OCR (1, 2, 4 and 8). The samples of Presumpscot were consolidated to their in situ vertical effective stress (30 kPa) and the measured preconsolidation stress (156 kPa). These samples are believed to be normally consolidated (geologically) and their overconsolidation stems from development of fabric. The samples of organic silt were also consolidated to vertical effective stresses of 156 kPa and 30 kPa, even though the samples were reconstituted, to be able to compare the results with the tests on the Presumpscot clay.

All samples were sheared at a horizontal displacement rate of 0.015 mm/min. This rate was based on ASTM D6528 suggestion of using a shear strain rate of 5% per hour, which is a common assumption in existing data. The undrained shear strength is dependent on the strain rate, so it is paramount that the same strain rate was used for all of the tests to accurately compare the data (Jung, 2005).

4. Results and Discussion

This chapter presents and discusses all of the data collected during the laboratory testing program. Direct Simple Shear tests were performed on intact samples of marine clay from the Gulf of Mexico, sensitive clay from Maine called Presumpscot clay, and reconstituted samples of organic silt from Narragansett Bay, RI.

All samples were consolidated to a given vertical effective stress and then sheared under undrained conditions. The shear wave velocity was measured at the final vertical effective stress, so that the relationship between the small strain shear modulus and the undrained shear strength (i.e. ratio of G_o/S_u) could be determined.

The samples of Gulf of Mexico clay were consolidated to different overconsolidation ratios and sheared to determine SHANSEP parameters. The samples of Presumpscot Clay were consolidated to the estimated vertical effective stress from which the samples were taken to estimate the *in-situ* undrained shear strength (i.e. recompression tests) and the measured preconsolidation stress. The samples of organic silt were sheared at vertical effective stresses of 30 kPa and 156 kPa.

The three clays were chosen specifically because they represent plastic soils with a range of plasticity indices for studying the link between small strain (G_o) and large strain (S_u) properties.

This chapter is divided into the following sections:

- Consolidation behavior
- Stress-strain behavior and undrained shear strength
- Shear wave velocity during consolidation
- Ratio of small strain shear modulus to undrained shear strength

4.1 Consolidation Behavior

The consolidation data for all of the tests was analyzed to ensure the soil samples were properly consolidated to the desired stresses and to evaluate the sample quality. This is particularly important since the confining stresses acting on a sample strongly influence the shear wave velocity measurements. In addition, a comparison was made between the vertical strains to the vertical consolidation stresses for each of the DSS tests and a standard incremental load consolidation test to evaluate the level of disturbance in the DSS sample preparation methodology and with the stacked rings.

4.1.1 Gulf of Mexico Clay

The Gulf of Mexico consolidation results for both the DSS tests and the incremental load consolidation test are shown in Figure 4.1. All of the results are very similar, which validates the use of the DSS stacked rings. Small differences can be attributed to trimming and preparation of the soil specimens.

The estimated preconsolidation stress for these samples ranged from 30 to 40 kPa, and the vertical strain to the preconsolidation stress ranged from 1% to 4%. According to the sample quality designation proposed by Andresen and Kolstad (1979) and Terzaghi et al. (1996), and Lunne et al. (1997), these samples are considered to be poor to very poor quality. This is understandable considering the samples were collected in 1998 from water depths in excess of 1000m and have been stored horizontally since then at approximately 10° C.

Figure 4.1 Comparison of Gulf of Mexico consolidation data from an IL consolidation test and DSS tests.

4.1.2 Presumpscot Clay

The consolidation results for two DSS tests and one incremental load consolidation test on samples of Presumpscot clay are shown in Figure 4.2. Both the DSS test and the incremental load test came from a block sample from a depth of 4m with an estimated vertical effective stress of 30 kPa. The measured preconsolidation stress from the incremental load test was 156 kPa, which highlights the sensitivity of this clay. The strain to the vertical stress of 30 kPa in the incremental load test was < 2%, which indicates a sample quality designation of “good” (Terzaghi et al., 1996).

One DSS test was consolidated to the in-situ vertical effective stress (30 kPa) and the other was consolidated to the preconsolidation stress (156 kPa). The strain at the in-situ effective stress for both tests was greater than 2%, indicating that some sample disturbance occurred during sample preparation.

Figure 4.2 Comparison of Presumpscot clay consolidation data from an IL test and two DSS tests.

4.1.3 Organic Silt from Narragansett Bay

The primary reason for testing the organic silt was its low plasticity index, which would provide a range of G_o/S_u values for different soils. Since the samples were remolded and each test had different moisture contents, the consolidation phase was not imperative in determining the similarity of behavior during this phase.

However, the compression indices (i.e. slope of the compression behavior) are similar (see Figure 4.3).

Figure 4.3 Consolidation data from Narragansett Bay organic silt from DSS tests.

4.1.4 Summary of Consolidation Test Results

Table 4.1 summarizes the results of the consolidation tests on samples of Gulf of Mexico Clay, Presumpscot Clay and organic silt from Narragansett Bay.

Table 4.1 Summary of the consolidation test results.

Soil ¹	Test No.	Type ²	σ'_p (kPa)	σ'_{vc} (kPa)	σ'_{vf} (kPa)	OCR	C_{er}	C_{ec}
GoM	1	IL	40	1028	1028	-	0.013	0.206
GoM	2	DSS	40	200	200	1	-	252
GoM	3	DSS	40	200	100	2	-	0.246
GoM	4	DSS	40	200	50	4	-	0.255
GoM	5	DSS	40	200	25	8	-	0.233
Presumpscot	6	IL	156	865	865	-	0.011	0.249
Presumpscot	7	DSS	156	156	156	-	-	0.156
Presumpscot	8	DSS	156	156	30	-	-	-
Organic Silt	9	DSS	-	156	156	-	-	-
Organic Silt	10	DSS	-	30	30	-	-	-

1. GoM = Gulf of Mexico Clay, Presumpscot = Presumpscot Clay

2. IL = Incremental Load Test, DSS = Direct Simple Shear

4.2 Stress-Strain Behavior and Undrained Shear Strength

Figures 4.4 through 4.9 show plots of shear stress vs. shear strain and excess pore pressure vs. shear strain for the three soils tested. The excess pore pressure was not measured, but was assumed to be equal to the change in vertical stress required during shear to maintain constant volume (Geocomp Corporation, 2011). In general, the samples of Gulf of Mexico and Presumpscot clay exhibited strain hardening behavior.

The stress-strain data for the Gulf of Mexico clays shows the combined effect of vertical effective consolidation stress and overconsolidation on the peak stress (undrained shear strength). As the samples go from contractive ($\sigma'_{vf} = 200$ kPa, OCR = 1) to dilative ($\sigma'_{vf} = 25$ kPa, OCR = 8), the strain softening behavior decreases. The pore pressure data is not entirely consistent with this trend as Test 3 at $\sigma'_{vf} = 100$ kPa shows the sample becoming contractive as the other samples became dilative as shown in Figure 4.5. It is not clear why this behavior was observed.

Figure 4.4 Shear strain vs. shear stress for the Gulf of Mexico clay with varying confining stresses.

Figure 4.5 Excess pore pressure for the Gulf of Mexico Clay.

The stress-strain data for the Presumpscot clay also shows the effect of consolidation stress on strength. This is a normally consolidated (based on the geology of the site) sensitive clay with an in-situ vertical effective stress estimated to be 30 kPa. Test 7 was consolidated to a vertical effective stress close to the measured preconsolidation stress (156 kPa) and the stress-strain data is consistent with a sensitive, contractive soil. Test 8 was consolidated to the in-situ vertical effective stress, and the stress-strain behavior exhibits strain-hardening behavior. This is consistent with a sensitive clay below its yield stress.

Figure 4.6 Shear strain vs. shear stress of Presumpscot clay.

Figure 4.7 Excess pore pressure for the Presumpscot Clay.

The two samples of organic silt were compacted using a Harvard miniature device and were clearly mechanically overconsolidated. Both samples exhibited strain-hardening behavior typical of dilative samples during undrained shear.

Figure 4.8 Shear strain vs. shear stress of the organic silt.

Figure 4.9 Excess pore pressure for the organic silt.

A summary of the undrained shear strengths for all these tests is shown in Table 4.2.

Table 4.2 Summary of the undrained shear strengths.

Soil ¹	Test No.	Type ²	σ'_{vf} (kPa)	OCR	S_u ³ (kPa)	S_u/σ'_{vf}
GoM	1	IL	1028	-	-	-
GoM	2	DSS	200	1	60.6	0.30
GoM	3	DSS	100	2	54.0	0.54
GoM	4	DSS	50	4	46.8	0.94
GoM	5	DSS	25	8	37.3	1.49
Presumpscot	6	IL	865	-	-	-
Presumpscot	7	DSS	156	-	34.9	0.22
Presumpscot	8	DSS	30	-	16.0	0.53
Organic Silt	9	DSS	156	-	55.0	0.35
Organic Silt	10	DSS	30	-	17.5	0.58

1. GoM = Gulf of Mexico Clay, Presumpscot = Presumpscot Clay

2. IL = Incremental Load Test, DSS = Direct Simple Shear

3. Peak shear stress was used as the failure criterion for tests 2-7 where a clear peak was visible. 20% shear strain was used for tests 8-10.

4.3 Stress History and Normalized Soil Engineering Properties (SHANSEP)

SHANSEP parameters were evaluated for the samples of Gulf of Mexico clay (see Figure 4.10). These parameters described the undrained shear strength as a function of vertical effective stress and stress history (i.e. overconsolidation ratio) by the following equation (see Chapter 2):

$$S_u = \sigma'_v (S_u/\sigma'_v)_{OCR=1} OCR^m$$

or

$$S_u/\sigma'_{vc} = S^* OCR^m$$

Figure 4.7 shows values of S_u/σ'_{vc} for different values of OCR (see Table 4.2) compared to values published by Ladd and Foott (1974) and McGuire (2011). The resulting values of S and m for the Gulf Mexico clay under DSS loading condition are .3 and .17. These are consistent with published values although the value of S (S_u/σ'_v at $OCR = 1$) is higher than expected and higher than values reported by McGuire

(2011) on similar samples of Gulf of Mexico clay. It is not clear why these values are different.

Figure 4.10 SHANSEP parameters for the Gulf of Mexico Clay (adapted from Ladd and Foott (1974) and McGuire (2011)).

4.4 Shear Wave Velocity During Consolidation

Shear wave velocity was measured using the methods described in Chapter 3 during the consolidation phase for all the DSS tests. The shear wave velocity at the final effective stress before shear was used to obtain G_o for the G_o/S_u relationship described in the next section. At intermediate consolidation stresses, the shear wave velocity was used to evaluate the consistency of samples between tests. Figure 4.11 shows the relationship between shear wave velocity and vertical effective stress for all of the tests.

Overall, the consistency of results for each soil type is excellent. The Gulf of Mexico data shows an increase in shear wave velocity to 130-140 m/s at $\sigma'_{vc} = 200$

kPa. As the samples were unloaded the shear wave velocity did not decrease much, illustrating how stress history (i.e. overconsolidation) “locks in” the strength and stiffness of a soil. Shear wave velocities of the Presumpscot clay are similar to the values of the overconsolidated Gulf of Mexico clay. The shear wave velocities of the organic silts are significantly higher than the clays, which is also understandable given the larger grain sizes.

The dependency of shear wave velocity on soil type can also be seen in the received signals by looking at the first peak from only one soil (Figure 4.12) compared to the received signals from all three soils (Figure 4.13).

Figure 4.11 Effective vertical stress vs. shear wave velocity measurements demonstrates the correlation between the soil type and measured shear wave velocity.

Figure 4.12 Shear wave comparison of different Gulf of Mexico samples under similar confining pressures of 200 kPa and 3 kHz frequency. The solid vertical lines estimate the location of the received signal's peak.

Figure 4.13 Comparisons of different soils under similar confining pressure of 30kPa with a 3kHz frequency signal wave. The solid vertical lines estimate the location of the received signal's peak.

4.5 Relationship Between Small Strain Shear Modulus and Undrained Shear Strength

The primary objectives of this thesis were to develop a shear wave velocity measurement system for the DSS and to evaluate a link between small strain and large strain properties. Andersen (2008) presented laboratory data showing a relationship between the ratio of G_o/S_u and Plasticity Index for 11 different soils at various values of OCR. Table 4.3 shows comparable results for the soils tested in this thesis and Figure 4.14 shows the data from this study compared with Andersen's data. There is a good agreement between the results of this study to support the hypothesis that there is a clear link between the small strain shear modulus and the undrained shear strength of cohesive soils.

Table 4.3 Summary of final results for the small strain shear modulus and the undrained shear strength.

Soil ¹	Test No.	Type ²	Plasticity Index	ρ^3 (Mg/m ³)	S_u^4 (kPa)	$G_{o, \min}^5$ (kg/m s ²)	$G_{o, \max}^6$ (kg/m s ²)	$G_o/S_{u, \min}^5$	$G_o/S_{u, \max}^6$
GoM	1	IL	-	-	-	-	-	-	-
GoM	2	DSS	41	1.60	60.62	41220457.84	51568493.42	679.94	850.64
GoM	3	DSS	41	1.40	53.98	18788904.46	24079763.64	348.04	446.05
GoM	4	DSS	41	1.59	46.85	15441473.54	22627533.25	329.62	483.02
GoM	5	DSS	41	1.43	37.33	10085843.48	13674276.45	270.20	366.34
Presumpscot	6	IL	-	-	-	-	-	-	-
Presumpscot	7	DSS	32	1.68	34.94	19291699.31	28729724.00	552.06	822.15
Presumpscot	8	DSS	32	1.73	16.03	12353689.95	17523805.10	770.54	1093.01
Organic Silt	9	DSS	7	1.83	54.99	84024366.12	205851755.19	1528.11	3743.73
Organic Silt	10	DSS	7	2.02	17.54	36550930.97	52467957.07	2084.38	2992.07

1. GoM = Gulf of Mexico Clay, Presumpscot = Presumpscot Clay

2. IL = Incremental Load Test, DSS = Direct

3. Bulk density of the soil

4. Peak shear stress was used as the failure criterion for tests 2-7 where a clear peak was visible. 20% shear strain was used for tests 8-10.

5. Minimum small strain shear modulus from the minimum measured shear wave velocity.

6. Maximum small strain shear modulus from the maximum measured shear wave velocity.

Figure 4.14 Plasticity Index vs. Ratio of Small Strain Shear Modulus and Shear Strength for Gulf of Mexico clay, Gulf of Maine Clay and organic silt.

5. Conclusions

The objectives of this thesis were to build a shear wave velocity measurement system in a commercially available direct simple shear apparatus and to evaluate a possible link between small and large strain properties of cohesive soils.

Construction of such a system has improved the capabilities to conduct research at URI in such areas as shear modulus degradation during shear and assessment of the liquefaction potential of soils.

Piezoceramic bender elements were installed in new brass end caps specifically designed for this study. The bender elements were coated with polyurethane and potted with electrical epoxy for waterproofing and placement in the brass end caps. Coaxial cables with BNC connectors were used to reduce electrical noise. The transmitted signal was a sinusoidal burst with an amplitude of 10 V peak to peak at frequencies ranging from 3 to 30 kHz. Received signals were filtered with a wide band pass filter and amplified using an analog amplifier, and the time delays were recorded with an oscilloscope. The shear wave velocity of Ottawa sand was measured at different confining stresses and the agreement with published values was very good.

Important benefits of the new DSS- V_s system include the following:

1. If the bender elements fail or are seriously damaged, they can be easily replaced or repaired by applying enough heat to the brass with a propane torch to melt the interface between the epoxy and the brass. This allows the elements to be removed, repaired and reinstalled, without damaging the brass. This is an improvement

over the PVC end caps currently used at URI, in which all bender elements need to be destroyed by being drilled out during the removal process.

2. The 63 mm diameter bender element end caps can be removed from the PVC base and 71 mm diameter bender element end cap can be easily fabricated and installed if research wants to be conducted with wire reinforced membranes.
3. All wiring utilized standard coaxial cable with BNC connectors. This allows for replacement or upgrades to be easily conducted without significantly changing the standard configuration or finding nonstandard wiring.
4. The new DSS- V_s system is transferrable to an existing cyclic-DSS apparatus currently used at URI.

To investigate a possible link between small and large strain properties of cohesive soils, DSS- V_s tests were performed on three soils: a marine clay from the Gulf of Mexico, a sensitive clay from Maine called Presumpscot clay, and organic silt obtained from Narragansett Bay. These soils were chosen because of their plasticity indices ranged from 7 to 42.

Shear wave velocity measurements were made at the end of consolidation and prior to undrained shearing of the samples. The ratio of small strain shear modulus (from the shear wave velocity) to undrained shear strength for the three soils was compared to published data of 11 different soils of varying stress histories and plasticity. The agreement with published data was very good, illustrating that the new

DSS- V_s system works well and there is a clear link between small and large strain properties of cohesive soils.

In addition to these findings, SHANSEP parameters were developed in the DSS for the intact samples of Gulf of Mexico clay. The ratio of S_u/σ'_v for $OCR = 1$ was found to be 0.30 and the slope of the S_u/σ'_v vs. OCR relationship was 0.17. These values were consistent with published values.

Samples of Presumpscot clay were carved from a high quality block sample donated by Professor Melissa Landon Maynard of the University of Maine. Because of the high quality of the samples, sample preparation methods were refined to minimize sample disturbance. DSS- V_s tests were only performed at the *in-situ* vertical effective stress and the measured preconsolidation stress. It is believed that these samples were the most sensitive and highest quality ever tested at URI.

Appendix A

DSS Test Results

1. Gulf of Mexico Clay

Figure A.1.1 Vertical effective stress vs. vertical strain for each of the tested samples.

Figure A.1.2 Shear stress vs. shear strain plot for each of the tested samples.

Figure A.1.3 Shear strain vs. excess pore pressure plot only during the shear.

Figure A.1.4 Shear stress vs. normal effective stress for each of the tested soils.

Figure A.1.5 Shear strain vs. normalized shear stress by normal effective stress.

2. Presumpscot Clay

Figure A.2.1 Vertical effective stress vs. vertical strain for each of the tested samples and a one dimensional consolidation test.

Figure A.2.2 Shear stress vs. shear strain for each of the tested soils.

Figure A.2.3 Shear strain vs. excess pore pressure only during shear.

Figure A.2.4 Shear stress vs. normal effective stress for the tested soils.

3. Organic Silt

Figure A.3.1 Vertical effective stress vs. vertical strain during consolidation.

Figure A.3.2 Shear stress vs. shear strain of the tested samples.

Figure A.3.3 Shear strain vs. excess pore pressure during shear.

Figure A.3.4 Shear stress vs. normal effective stress of the tested samples.

Appendix B

Shear Wave Velocity Test Results

1. Gulf of Mexico

Figure B.1.1 Shear wave velocity measurements for Gulf of Mexico clay with an OCR of 1 at a final confining stress of 200kPa and a frequency of 3 kHz.

Figure B.1.2 Shear wave velocity measurements for Gulf of Mexico clay with an OCR of 2 at a final confining stress of 100kPa and a frequency of 3 kHz.

Figure B.1.3 Shear wave velocity measurements for Gulf of Mexico clay with an OCR of 4 at a final confining stress of 50kPa and a frequency of 3 kHz.

Figure B.1.4 Shear wave velocity measurements for Gulf of Mexico clay with an OCR of 8 at a final confining stress of 25kPa and a frequency of 3 kHz.

2. Presumpscot Clay

Figure B.2.1 Shear wave velocity measurements for Presumpscot clay with a final confining stress of 156 kPa with a frequency of 3 kHz.

Figure B.2.2 Shear wave velocity measurements for Presumpscot clay with a final confining stress of 30 kPa with a frequency of 3 kHz.

3. Organic Silt

Figure B.3.1 Shear wave velocity measurements for organic silt with a final confining stress of 156 kPa with a frequency of 3 kHz.

Figure B.3.2 Shear wave velocity measurements of organic silt with a final confining stress of 30 kPa with a frequency of 3 kHz.

Appendix C

Laboratory Handling and Preparation Procedures

1. Gulf of Mexico

Figure C.1.1 KNR 159 JPC-24 Sec 13 (1751-1884CM), 1.5m core ready to be cut.

Figure C.1.2 76mm section is cut from the 1.5m core using a band saw.

Figure C.1.3 A tube with piano wire is inserted to break the bond between the PVC and the soil followed by a saw like motion closely following the outer diameter of the soil sample.

Figure C.1.4 The specimen is removed from the PVC liner.

Figure C.1.5 The specimen is trimmed to a diameter of 71mm.

Figure C.1.6 A 63.5mm diameter cutting ring is placed on the top of the specimen, and the top rod is lowered to restrict the cutting ring's motion. The cutting ring has been lubricated with mineral oil to decrease frictional forces, and pushed down in approximate 5 mm increments.

Figure C.1.7 As the cutting ring is lowered the excess soil is removed to prevent cracking.

Figure C.1.8 A second cutting ring is added to allow the first cutting ring to get the approximate middle 25mm portion of the 76mm long sample.

Figure C.1.9 The specimen is moved to a different trimming stand.

Figure C.1.10 The second cutting ring is removed.

Figure C.1.11 The specimen is trimmed by using the cutting ring as a reference while cutting the specimen, as the excess soil is rotated vertically by the spatula.

Figure C.1.12 A reference bar is attached via screws and the whole specimen is flipped to trim the bottom.

Figure C.1.13 The bottom is trimmed using the cutting ring as a guide.

Figure C.1.14 The specimen is flipped and a 6.3mm thick porous stone with attached wax paper is placed on the top of the sample to ensure the sample's height is 25.4mm.

Figure C.1.15 The specimen is flipped and the final trimming to the bottom is made.

Figure C.1.16 The specimen is then push out via the porous stone onto the bottom bender element end cap.

Figure C.1.17 The membrane is brought over the sample via the membrane stretcher.

Figure C.1.18 The membrane stretcher is lowered around the sample.

Figure C.1.19 The membrane is rolled off the stretcher and around the sample. The membrane stretcher is then removed.

Figure C.1.20 The Teflon rings are lowered individually around the specimen.

Figure C.1.21 The bottom base is then placed in the DSS apparatus, and the split mold is placed around the stacked rings. The membrane is stretched around the top of the split mold.

Figure C.1.22 The top bender element end cap is lowered onto the sample. The membrane is then guided off the split mold and around the top cap.

Figure C.1.23 The split mold is removed and the specimen is ready to be tested.

2. Presumpscot Clay: Block Sample

Figure C.2.1 The Presumpscot clay block sample No. 7.

Figure C.2.2 Small cuts are made to the top of the block sample. Deep enough cuts to penetrate only one layer of wax at a time.

Figure C.2.3 A spatula is rolling up the top wax coat towards the center of the sample.

Figure C.2.4 The exposed outside layer of the block sample.

Figure C.2.5 The sample is removed and the rest of the block sample is re-waxed and placed in the storage refrigerator.

Figure C.2.6 The specimen is placed on the trimming platform.

Figure C.2.7 The 71mm cutting ring is fixed and lowered through the sample, the remaining steps follows the Gulf of Mexico Clay trimming process.

3. Presumpscot Clay: Shelby Tubes

Due to the sensitive nature of the Presumpscot Clay, special extraction procedures were implemented to obtain undisturbed samples. The procedure was similar to the Gulf of Mexico clay extraction with the cutting of the tubes being different. This is illustrated in Figures C.3.1 through C.3.3.

Figure C.3.1 The Shelby tube is placed in the specially made brackets, and a pipe cutter with brass cutting blades is rotated to cut through the brass tube. The pipe cutter's speed of rotation needs to be constant, and a second person is needed to provide additional friction to the tube if it starts to rotate.

Figure C.3.2 A wire slowly cuts through the soil to separate the two pieces.

Figure C.3.3 The two pieces are separated and the sample is ready to be extracted from the tube, which followed the same procedure as the Gulf of Mexico Clay.

Appendix D

Assessment of Uncertainty of Shear Wave Velocity Measurements

There are two sources of uncertainty associated with determining the time delay for shear wave velocity measurements: errors in picking the correct peak of the received signal and the effect of transmitted signal frequency on the measured time delay. Each of these will be addressed briefly in this appendix. The time delay for the determination of shear wave velocity in this thesis was estimated by observing the received signal on an oscilloscope and manually picking the peak. In some cases the peak of the received signal was very clear while in others a range of possibilities were possible. This is illustrated in Figure D.1. In each of the examples, the input signal wave was viewed along with the received signal to best estimate the time delay using judgment.

Figure D.2 illustrates the approximate range of uncertainty from visually picking the peak of the received signal for each of the shear wave velocity measurements. In each case, the minimum and maximum plausible peak of the received signal was chosen. The results show that, as the confining stress and the shear wave velocity increased, so did the variability in the data. Figure D.2 also shows that the uncertainty in picking the first arrival time was greatest with the samples of compacted organic silt. These samples were unsaturated and completely reconstituted, and it could be that this state results in more uncertainty than in the natural samples of Gulf of Mexico and Presumpscot clay.

Figure D.1 Common examples of the received signals, and the error accumulated from human judgment required to determine the shear wave velocity measurements. The vertical lines are the estimated locations of the peak for each of the received signals.

Figure D.2 The variability in the data for each of the samples of three soils with a 3 kHz input frequency.

The thesis also looked at the variability of the data by changing the frequencies. Figure D.3 illustrates the range between the 3 kHz to the 30 kHz. 3 KHz frequency was the clearest and selected for the results of this thesis.

Figure D.3 illustrates the effect of the frequency of the transmitted signal on the measured values of shear wave velocity. The error bars represent the shear wave velocities using a 3 kHz and 30 kHz input signal. The 3 kHz values were used for this thesis. The effect of frequency appears to be modest for the samples of Gulf of Mexico and Presumpscot clay, and quite significant for the organic silt. Figure D.4 shows the form of the frequency effect for each soil at a vertical effective consolidation stress of 150 kPa. There appears to be a significant difference between 3 kHz and 15 kHz. Clearly more research is needed into the effects of frequency on shear wave velocity.

Figure D.3 Variability in the shear wave velocity measurement by changing the frequency from 3 kHz to 30 kHz.

Figure D.4 Frequency comparison for the three soils at a 150 kPa confining stress.

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