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Undrained Cyclic Shear Resistance of Low Plastic Silts

by

Rawan Almoumen

A thesis submitted in partial fulfillment of the
requirements for the degree of

Master of Science
in
Civil and Environmental Engineering

Thesis Committee:
Arash Khosravifar, Chair
Diane Moug
Thomas Schumacher

Portland State University
2020

Abstract

A magnitude 9 Cascadia Subduction Zone earthquake is expected to trigger widespread liquefaction in loose material in Oregon. The geotechnical engineering studies have determined that the Willamette Silts may be susceptible to liquefaction or cyclic softening, as their plasticity indices ranges between non-plastic to low plasticity. While the majority of past studies have focused on liquefaction of sand and cyclic behavior of clays, there is not enough data on the cyclic response behavior of silty soils and the liquefaction susceptibility of these soils. A research focus in the geotechnical engineering program at Portland State University revolves around is performing cyclic tests on samples obtained from regional silts to fill the gap in data, in an effort to better characterize the liquefaction susceptibility of non-plastic to low plasticity silts. The primary goal of this thesis is to study the behavior of undrained cyclic shear resistance of low plastic fine-grained soils that were extracted from Beaverton, OR. The constant volume direct simple shear test device made by Geocomp is used to test the soil samples. The critical challenge in this project was to prepare close-to-identical remolded specimens to be able to evaluate intricate differences in the cyclic behavior of silts under different loading conditions. The thesis will present the cyclic shear resistance of laboratory prepared samples and the adopted procedures to prepare identical slurry samples.

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List of Abbreviations:

PI: Plasticity Index

PL: Plastic Limit

LL: Liquid Limit

OCR: Over-Consolidation Ratio

DSS: Direct Simple Shear

CDSS: Constant Volume Direct Simple Shear (Cyclic Direct Simple Shear)

CSR: Cyclic Stress Ratio

CRR: Cyclic Resistance Ratio

1-D: One-Dimensional

FC: Fines Content

ML: Low Plastic Silt

CL: Low Plastic Clay

CPT: Cone Penetration Test

ASTM: American Society for Testing of Materials

Hz: Hertz

USCS: Unified Soil Classification System.

W_c/LL: Ratio of Water Content to Liquid Limit

OR: Oregon

List of Symbols

e : Void ratio

e_0 : Initial void ratio of the specimen

e_c : Void ratio of the specimen after the end of consolidation phase

e_f : Final void ratio

σ'_v : Vertical effective stress

σ'_{v0} : Initial vertical effective stress

σ'_{vc} : Vertical consolidation stress

r_u : Excess pore-water pressure ratio

τ_{cyc} : Cyclic shear stress

S_u : Undrained shear strength

$S_{u, st}$: Static Undrained shear strength

$S_{u, post-cyc}$: Post cyclic undrained shear strength

γ : Shear strain

$\tau_{post-cyclic}$: Post cyclic shear strain

τ_{static} : Static shear strain

ϵ_a : Axial strain

H_0 : Initial height

H_f : Final height

K_0 : lateral earth pressure

1. INTRODUCTION

1.1 Overview of Soil Liquefaction

There are two terms commonly used in geotechnical engineering regarding ground failures under earthquake loads. These two terms are liquefaction and cyclic softening. Studies and researchers determined that the term “liquefaction” means strength loss and deformation in saturated sands and cohesionless soils where the “cyclic softening” is the strength loss and deformation in clays and plastic silts. However, the critical question is how to determine whether these fine-grained soils will be susceptible to liquefaction or cyclic softening. Some commonly used methods to determine the liquefaction susceptibility are Chinese criteria (as described in Youd et al. 2001), Bray and Sancio (2006) method, and Idriss and Boulanger (2008) method. These methods will be explained in the following section.

1.2 Liquefaction Susceptibility of Fine-Grained Soils

The Chinese criteria were the fundamental method to clarify the susceptibility of fine-grained soils. In this criteria, the clay particle fractions (finer than 0.005mm) is used to determine whether the soil sample will be susceptible to liquefaction. Chinese criteria defined liquefaction by particle size less than 15%, liquid limit (LL) less than 35%, and water content (Wc) larger than 0.9LL. Figure 1.1 presents Wang’s data (1979) that illustrates this criteria.

Figure 1.1: Liquefaction susceptibility criteria by Wang (1979)

The left chart presents the Casagrande plasticity chart. This chart classifies that all the data that their plastic limit less than 35 and their plasticity index less 15 will be considered as low plasticity clays, low plasticity silts, or low plasticity clayey silts. The data from the right chart describes that if the ratio of water content to liquid limit (W_c/LL) >0.9 , the sample will be liquefied where the data with $W_c/LL < 0.9$ will not liquefy. All the test data was done by using clay contents less than 15%.

However, recent studies and investigations (especially after 1989 Loma Prieta earthquake) noted that the liquid limit, W_c/LL , and soil particle size are not the fundamental factors to determine the liquefaction susceptibility of fine-grained soils. Thus, Boulanger and Idriss (2008) and Bray and Sancio (2006) recommended the plasticity index (PI) as a better factor in determining the liquefaction susceptibility of fine-grained soils. Idriss and Boulanger used Atterberg limits to determine the PI and define the silt sample as “clay-like” or “sand-like.” Figure 1.2 characterizes the Atterberg limits chart with different values to describe the range of plasticity for “clay-like” silts and the range of plasticity for “sand-like” silts.

Figure 1.2: Liquefaction susceptibility criteria based on Idriss and Boulanger (2008)

Atterberg limits chart clarifies that the soil sample with a PI that is less than 7 should be considered a “sand-like” silts, and the soil sample with a PI that is greater than 7 should be defined as “clay-like.” The transition between “sand-like” and “clay-like” behavior in fine-grained soils is illustrated in Figure 1.3. This transition indicates that as the plasticity index increases, the silt sample will behave as a “clay-like” behavior.

Figure 1.3: Schematic of the transition from sand-like to clay-like behavior for fine-grained soils with increasing PI, and the recommended guideline for practice (Idriss and Boulanger 2008)

After determining the PI of fine-grained soils, the effect of soil plasticity in liquefaction susceptibility should be studied. Thus, Bray and Sancio (2006) tested some specimens with 4 different PI ranging between 0 and 18 at the same isotropic confining stress and frequency of 1 Hz. The test results were grouped into three categories, soil with $PI < 12$, soil with $12 < PI < 18$, and soil with $PI > 18$. Based on these results, the soil with $PI < 12$ and $W_c/LL > 0.85$ will be susceptible to liquefaction. Also, the soil that has a $PI > 18$ will not liquefy at low effective stress where soil with $12 < PI < 18$ and $W_c/LL > 0.8$ will be moderately susceptible to liquefaction. Combined with Boulanger and Idriss (2008) method, these results are the fundamental criteria to determine the liquefaction susceptibility of fine-grained soils. Bray and Sancio (2006) method and Boulanger and Idriss (2008) are widely used in the geotechnical engineering firms. However, these two

methods are not sufficient to identify the liquefaction behavior of fine-grained soils.

Further researches are required to understand the silty soil behavior and whether it acts like clay or sand.

A constant volume Cyclic Direct Simple Shear (CDSS) is a commonly used device to determine the liquefaction susceptibility of fine-grained soils. The CDSS develop a liquefaction resistance curve that helps understand the characteristics of soils. Figure 1.4 shows the difference between sand like and clay like silt when tested in the CDSS device.

Figure 1.4: Shear Stress-Shear strain loops from CDSS device for a) Sand like b) Clay like (Idriss and Boulanger 2016)

1.3 Thesis Objective

A research at the geotechnical engineering group at Portland State University is focused on performing cyclic tests to identify the cyclic behavior and the liquefaction susceptibility of low plasticity silts. This thesis's main objective is to study the undrained cyclic shear resistance of these fine-grained soils located in Beaverton, OR. By creating identical samples and test them with different cyclic stress ratio using a constant volume cyclic direct, simple shear device (CDSS) made by GeoComp.

Chapter two represents literature reviews that focused on low plastic silts and the factors that affect the results of the laboratory tests. Chapter three introduces the tested material and the type of testes used in this project. Chapter four presents the results of these tests. Chapter five has a discussion of the final results and comparisons with other researchers' results. Finally, chapter five suggests some future works and researches that are needed to understand the behavior of fine-grained soils.

2. BACKGROUND

Researchers have utilized geotechnical laboratory tests to advance the knowledge on how natural fine-grained soils behave under cyclic loading. Many of these laboratory tests were done using a direct simple shear (DSS) machine. These tests are essential to understand the behavior of silty soils. These studies illustrate that many parameters can affect the undrained cyclic behavior of soils and help to understand silt characteristics such as plasticity index, over-consolidation ratio, initial confining stress, and soil fabric. This chapter talks briefly about past studies that focused on the effects of these factors on fine-grained soils.

2.1 Plasticity Index

As stated in chapter 1, the soil plasticity is a suitable index parameter to study the behavior of fine-grained soil and understand whether it behaves as a "sand-like" or "clay-like" silt. Many literature reviews were done using the DSS to test silt samples from one region but have different PI. Wijewickreme (2019) tested many types of soils from British Columbia, Canada, with 4 different plasticity indexes (4,5,7 and 34) to investigate these samples' mechanical behavior. The results show that as the plasticity increases, the undrained shear strength and the cyclic resistance will increase as well as increase the pore water pressure and the shear strain when tested using the same confining stress. Moreover, all the samples during the monotonic shear loading have a contractive response (tendency to decrease its volume) except the sample with very low PI (PI=4) has a slightly dilative response (tendency to increase its volume).

2.2 Over Consolidation Ratio (OCR)

Studies indicated that the increase in the over-consolidation ratio leads to an increase in the cyclic resistance of silts. This result was determined by Ishihara and (2004) as shown in Figure 2.1. Ishihara described liquefaction as pore water pressure reaches 100% and shear strain = 5%. He studied the impact of the stress history on the liquefaction behavior of clean sand based on different OCR and lateral earth pressure (K_0) using a torsional shear test. However, researchers found that it might be inaccurate to assume the sand-based OCR method for silt since the silt's compressibility is higher than the compressibility of sand. Thus, Soysa (2015) tried to explain the OCR influence on the fine-grained soils in a different way. He performed monotonic tests for silt with 3 different over consolidation ratio (OCR) ranging from 1 to 4. Results show that the normally consolidated specimen ($NC = 1$) can reach to failure with lower CSR than the over-consolidated specimen ($OCR = 4$) which required a higher CSR to reach failure. As a result, Ishihara sand-based OCR method can be applied on low plastic silts.

Figure 2.1: Cyclic stress ratio versus number of cycles to DA=7.5% Ishihara (2004)

2.3 Initial Confining Stress

In general, when applying large consolidation stress values, the soil particle will be denser, so the void ratio will decrease. Soysa (2015) tested the normally consolidated silts specimens by applying 3 different initial confining stress as shown in Figure 2.2. This study was done to determine the influence of the initial confining stress on fine-grained soils. The Pre-consolidation stress values were estimated from 1-D consolidation tests to ensure that all these specimens were prepared from a normally consolidated state. Results show that when applying higher confining stress, a decrease in shear resistance occurs as well as a low value of excess pore water pressure ratio (r_u) for low plasticity silts. However, for lower confining stress, the shear resistance increased with increasing in the excess pore water pressure ratio (r_u). The main reason for this is when applying a higher stress on a sample, the specimen gets weaker and it reaches failure easily.

Figure 2.2: Cyclic Stress Ratio versus Number of loading cycles to reach $\gamma=3.75\%$ curves Soysa (2015)

2.4 Soil Fabric

The effect of soil fabric can be seen from sampling and specimen preparation.

Wijewickreme in 2019 prepared dense reconstituted specimens with void ratio ranges between 0.8 to 0.9 using the slurry method. Then, he compared these specimens with the undisturbed specimens (Figure 2.3) from fields that have void ratio between 1 to 1.1. The goal of this study is to identify the effect of soil fabric on the CSR. Due to the changing of sample disturbance during sampling preparation, the reconstituted samples reach failure with lower CSR than the undisturbed samples as shown in Figure 2.4. The reason for that is when preparing the reconstituted specimens, the natural soil fabric destroyed which reduces the strength and stiffnesses of the soils that gained from the in-situ conditions.

Figure 2.3: e -log σ'_v relationships for undisturbed and reconstituted specimens Wijewickrem (2019)

Figure 2.4: CSR versus Number of loading cycles to reach $\gamma = 3.75\%$ Wijewickrem (2019)

The goal of this research to create identical samples from Beaverton, OR silts and test them in different cyclic stress ratio and determine the behavior of these samples. Thus, the reconstituted method was used with keeping the OCR and the initial confining stress constant.

3. MATERIAL TESTED AND TESTING PROCEDURES

This chapter have three sections. Section 3.1 presents the Atterberg limit and sample properties, where section 3.2 talks about specimen preparation and sampling processes. Finally, section 3.3 describes the types of tests that been used in the laboratory.

3.1 Material Tested

The soil tested in this study was extracted from Beaverton, Oregon. Three Shelby tubes from GRI 2016 were opened and mixed up to get a considerable amount of soil needed to be tested. The standard laboratory test methods for liquid limit (LL), plastic limit (PL), and plasticity index (PI) of soils were used to classify the USCS per ASTM D4318. Table 3.1 presents the result of these tests.

Table 3.1: Atterberg limit test results

PL	LL	PI	USCS
20	28	8	ML

3.2 Specimen Preparation and Sampling Processes

After extracting the soil from the Shelby tubes, the material was passed through sieve No.40 (425 μm) to remove any coarse-grained material. An amount of dry soil was used with adding a large amount of water to force the fine-grained soil pass through N0.40 sieve. The mixture texture should be similar to Figure 3.1.

Figure 3.1: Add water to a dry soil during washing method

As the goal of this research is to get representative samples, it is essential to obtain a large amount of soil, store it in a large Ziplock bag, and save it in a room temperature place.

Figure 3.2: storing the soil in Ziplock bag

Now, the soil is ready to be prepared and test it in the CDSS. Figure 3.3 explains what is needed to prepare the sample.

- A) Bottom plate with porous bottom stone attached, note the vertical bolts on the bottom left and top right of the base plate.
- B) 2 black O-rings.
- C) 30 Teflon stackable rings.
- D) Top cap with attached porous stone.
- E) 2 Teflon ring pins.
- F) A suction ring with vacuum hose attachment.

Figure 3.3: Preparation Equipment

The first step is adding de-aired water to a small amount of the soil that passed sieve No. 40 and carefully mix it to reach the slurry target and water content needed (around 40%-42%). The de-aired water is used to achieve a fully saturated material which helps avoiding water tension between pore air and pore water.

Figure 3.4: Adding De-aired water to create the slurry

The slurry should be placed in a small container and kept under vacuum (around 5 psi) to ensure avoiding any air absorbed while mixing the slurry. The slurry kept under vacuum until finishing the other preparation part.

Figure 3.5: Slurry kept under vacuum

The next step is to attach the membrane to the porous bottom stone. Then, add one O-ring to ensure that the membrane and the bottom stone are making contact.

Figure 3.6: Attaching the membrane into the bottom plate

Then place around 30 aligned Teflon rings around the membrane as shown in Figure 3.7.

Figure 3.7: Place around 30 aligned Teflon rings

Place the membrane around the suction ring. Ensure that the membrane is touching the inside of the stacked Teflon rings. Now, add the slurry into about 0.9in.

Figure 3.8: Adding the slurry into the sample setup

Finally, place the top cap, take off the vacuum, add one more O-ring to the top cap above the membrane. Make sure the horizontal screw on the top cap is loose, this allows the piston to slide.

Figure 3.9: Placing the top cap into the setup

3.3 CDSS Set-up and Testing Schedules

3.3.1 Consolidation Phase

After preparing the specimen and placing it in the GeoComp set-up, the sample will be submerged in a container filled with de-aired water. A 5kPa stress will be applied first on the sample using a consolidometer for 6 hours, as shown in Figure 3.10. Then, 25 kPa stress will be applied for 16 hours. The rest of the consolidation phase will be on the CDSS device to meet the required effective consolidation stress, which is 100 kPa. Thus, the sample will be transferred to the CDSS device. The step by step detailed on how to set up the CSDD explains in Appendix A. A 50 kPa stress will be applied for 6 hours, and finally, the 100 kPa will be applied for another 6 hours. The reason to do the first two consolidation phases on a consolidometer rather than the CDSS, is to take advantage of another test using the device while the specimen is consolidating.

Figure 3.10: Submerged sample on the consolidometer.

3.3.2 Monotonic Shear Loading Phase

The shear loading phase is the second stage after the consolidation. There are two types of loading that are usually preformed on CDSS. The monotonic loading shear test is used to identify the undrained shear strength of the specimen. In the monotonic shear loading test, the sample will behave until suffered 20% shear strain with a rate of 0.00127/h. The monotonic test parameters are presented is Table 3.2.

Table 3.2: Sample parameters for monotonic tests

Soil Properties						Monotonic Shear Loading Properties					
Sample ID	PL	LL	PI	USCS	GS	σ'_{vc} (kPa)	Vert. stress prior to cyclic (kPa)	OCR	Static shear, α	Rate (%/h)	Max disp. (m)
M-1	20	28	8	ML	2.7	100	100	1	0	0.00127	0.005

3.3.3 Shear Loading Phase

The second type of shear loading test is the cyclic loading test. A stress-controlled test is preforming under different CSR, while the frequency of 0.1Hz. The maximum peak to peak shear strain is applied to be 6%, maximum excess pore water pressure $ru=1$, and the maximum number of cycles to reach failure is 100 cycles. In this project, 15 cyclic shear loading tests were done with CSR varying from 0.12, 0.15, and 0.2. The sample where prepared identically to compare them repeatedly. Thus, each CSR has 4-6 identical samples tested. Tables 3.3, 3.4 and 3.5 explain the testing schedules for each cyclic stress ratio tests.

Table 3.3: Sample parameters for cyclic shear loading test for CSR =0.2

Soil Properties						Cyclic Shear Loading Properties					
Sample ID	PL	LL	PI	USCS	GS	CSR	σ'_{vc} (kPa)	Vert. stress prior to cyclic (kPa)	OCR	Cyclic loading frequency (Hz)	Static shear, α
A-1	20	28	8	ML	2.7	0.2	100	100	1	0.1	0
A-2	20	28	8	ML	2.7	0.2	100	100	1	0.1	0
A-3	20	28	8	ML	2.7	0.2	100	100	1	0.1	0
A-4	20	28	8	ML	2.7	0.2	100	100	1	0.1	0
A-5	20	28	8	ML	2.7	0.2	100	100	1	0.1	0

Table 3.4: Sample parameters for cyclic loading for CSR =0.15

Soil Properties						Cyclic Shear Loading Properties					
Sample ID	PL	LL	PI	USCS	GS	CSR	σ'_{vc} (kPa)	Vert. stress prior to cyclic (kPa)	OCR	Cyclic loading frequency (Hz)	Static shear, α
B-1	20	28	8	ML	2.7	0.15	100	100	1	0.1	0
B-2	20	28	8	ML	2.7	0.15	100	100	1	0.1	0
B-3	20	28	8	ML	2.7	0.15	100	100	1	0.1	0
B-4	20	28	8	ML	2.7	0.15	100	100	1	0.1	0

Table 3.5: Sample parameters for cyclic loading for CSR =0.12

Soil Properties						Cyclic Shear Loading Properties					
Sample ID	PL	LL	PI	USCS	GS	CSR	σ'_{vc} (kPa)	Vert. stress prior to cyclic (kPa)	OCR	Cyclic loading frequency (Hz)	Static shear, α
C-1	20	28	8	ML	2.7	0.12	100	100	1	0.1	0
C-2	20	28	8	ML	2.7	0.12	100	100	1	0.1	0
C-3	20	28	8	ML	2.7	0.12	100	100	1	0.1	0
C-4	20	28	8	ML	2.7	0.12	100	100	1	0.1	0
C-5	20	28	8	ML	2.7	0.12	100	100	1	0.1	0
C-6	20	28	8	ML	2.7	0.12	100	100	1	0.1	0

3.3.4 Post Cyclic Phase

The final phase is the post cyclic shear phase. This test usually done after the consolidation and cyclic shear loading stage. The specimen will shear till reach 20% shear strain with rate of 0.00127/h and maximum force of 10kN. Table 3.6 shows the testing plan for 11 post cyclic shear tests.

Table 3.6: Sample parameters for post cyclic shear loading

Soil Properties						Cyclic Shear Loading Properties						Post Cyclic Properties	
Sample ID	PL	LL	PI	USCS	GS	CSR	σ'_{vc} (kPa)	Vert. stress prior to cyclic (kPa)	OCR	Cyclic loading frequency (Hz)	Static shear, α	Rate (%/h)	Max disp. (m)
A-2	20	28	8	ML	2.7	0.2	100	100	1	0.1	0	0.00127	0.005
A-3	20	28	8	ML	2.7	0.2	100	100	1	0.1	0	0.00127	0.005
A-4	20	28	8	ML	2.7	0.2	100	100	1	0.1	0	0.00127	0.005
A-5	20	28	8	ML	2.7	0.2	100	100	1	0.1	0	0.00127	0.005
B-2	20	28	8	ML	2.7	0.15	100	100	1	0.1	0	0.00127	0.005
B-3	20	28	8	ML	2.7	0.15	100	100	1	0.1	0	0.00127	0.005
B-4	20	28	8	ML	2.7	0.15	100	100	1	0.1	0	0.00127	0.005
C-3	20	28	8	ML	2.7	0.12	100	100	1	0.1	0	0.00127	0.005
C-4	20	28	8	ML	2.7	0.12	100	100	1	0.1	0	0.00127	0.005
C-5	20	28	8	ML	2.7	0.12	100	100	1	0.1	0	0.00127	0.005
C-6	20	28	8	ML	2.7	0.12	100	100	1	0.1	0	0.00127	0.005

4. TEST RESULTS

This chapter presents the results of one monotonic undrained shear test, 15 cyclic shear loading tests, and 11 post cyclic shear tests. After presenting the results, the samples' quality will be discussed to understand the parameters that can affect the results of identical samples and the experimental problems that can happen when using the CDSS device. The individual test results are presented in Appendix B.

4.1 Monotonic Undrained Loading

The monotonic undrained shear loading test is used to determine the undrained shear strength of soils (S_u) and the normalized undrained strength (S_u/σ'_{vc}). Literature reviews show that monotonic undrained shear strength can be a function of consolidation stress history. The undrained shear strength value was estimated at 10% shear strain. Figure 4.1 represents the shear stress and shear strain path, whereas Table 4.1 has the monotonic loading test results.

Figure 4.1: Stress and Strain path for monotonic test

Table 4.1: Sample results for monotonic test

ID	H _o (m)	Initial Wc (%)	Initial Void Ratio e _o	After 25kPa Wc (%)	After 25 kPa Void Ratio e _c	After 100 kPa Wc (%)	After 100kPa e	H _f (m)	Final Wc (%)	Final Void Ratio e _f	S _{u,st} (kPa)
M-1	0.022	40.5	1.061	40.5	0.805	29	0.688	0.018	29	0.898	32.56

4.2 Cyclic Shear Loading

The results of 15 cyclic shear response tests are discussed in this section. This study aims to create identical samples and test them at different CSR (0.12, 0.15, and 0.2). A slurry method was used to create identical samples with initial water content ranging from 41%-42.8% and initial void ratio ranging from 0.983 to 1.073. The water content and the void ratio were measured at four different times. The first time was initially after starting the consolidation stage. The second time was before setting the sample on the CDSS after the 25kPa consolidation stage. Then, after the test was done before taking it off the CDSS and finally after taking it from the CDSS. The water content and void ratio were calculated using these equations:

Water Content (%) =

$$[\text{weight of moist soil (g)} - \text{weight of dry soil (g)}] / \text{weight of dry soil (g)} \quad (4-1)$$

Void ratio =

$$[(\text{Density of water (g/cm}^3) * \text{Density of Solids (g/cm}^3)) / \text{Dry Density (g/cm}^3)] - 1 \quad (4-2)$$

Moreover, the failure criteria for all these tests were the same which were: maximum peak to peak shear strain 6%, maximum excess pore water pressure ratio (ru) = 1. The results of these tests are performed in tables 4.2, 4.3 and 4.4.

Table 4.2: Test results for cyclic loading for CSR =0.2

Sample ID	H ₀ (m)	Initial Wc (%)	Initial Void Ratio e ₀	After 25 kPa Wc (%)	After 25kPa Void Ratio e _c	After 100 kPa Wc (%)	After 100 kPa e	Final Wc (%)	Final Void Ratio e _f	H _f (m)	Number of cycles to Failure	Max r _u	S _u (kPa)	τ _{cyc} /S _u
A-1	0.021	42.7	0.983	42.7	0.695	29.5	0.578	29.5	0.555	0.016	3	0.6599	32.56	0.6144
A-2	0.022	41.3	1.072	41.3	0.809	31.6	0.7	31.6	0.744	0.017	2	0.6351	32.56	0.6144
A-3	0.022	41.8	1.073	41.8	0.802	27.6	0.652	27.6	0.657	0.017	3	0.7153	32.56	0.6144
A-4	0.021	41.6	1.036	41.6	0.799	26.6	0.612	26.6	0.629	0.017	5	0.6911	32.56	0.6144
A-5	0.021	42.3	1.065	42.3	0.778	28	0.551	28	0.556	0.016	5	0.7228	32.56	0.6144

Table 4.3: Sample results for cyclic loading for CSR =0.15

Sample ID	H ₀ (m)	Initial Wc (%)	Initial Void Ratio e ₀	After 25kPa Wc (%)	After 25 kPa Void Ratio e _c	After 100 kPa Wc (%)	After 100 kPa e	Final Wc (%)	Final Void Ratio e _f	H _f (m)	Number of cycles to Failure	Max r _u	S _u (kPa)	τ _{cyc} /S _u
B-1	0.022	42.8	1.071	42.8	0.802	30	0.687	30	0.682	0.017	19	0.8609	32.56	0.4608
B-2	0.022	42.6	1.065	42.6	0.797	30	0.761	30	0.753	0.018	17	0.8675	32.56	0.4608
B-3	0.022	42.1	1.066	42.1	0.794	29	0.628	29	0.648	0.017	9	0.7882	32.56	0.4608
B-4	0.022	42	1.070	42	0.803	28.7	0.720	28.7	0.730	0.018	10	0.8088	32.56	0.4608

Table 4.4: Sample results for cyclic loading for CSR =0.12

Sample ID	H _o (m)	Initial Wc (%)	Initial Void Ratio e _o	After 25 kPa Wc (%)	After 25 kPa Void Ratio e _c	After 100 kPa Wc (%)	After 100 kPa e	Final Wc (%)	Final Void Ratio e _f	H _f (m)	Number of cycles to Failure	Max r _u	S _u (kPa)	τ_{cyc}/S_u
C-1	0.022	41.7	1.033	41.7	0.781	29.2	0.701	29.2	0.701	0.018	51	0.9408	32.56	0.3686
C-2	0.022	41.6	1.06	41.6	0.755	28	0.596	28	0.606	0.016	22	0.8432	32.56	0.3686
C-3	0.022	41.6	1.055	41.6	0.756	28.7	0.678	28.7	0.690	0.017	33	0.8306	32.56	0.3686
C-4	0.022	42.6	1.059	42.6	0.799	28.8	0.623	28.8	0.672	0.017	63	0.8855	32.56	0.3686
C-5	0.022	41.1	1.057	41.1	0.805	27.7	0.656	27.7	0.690	0.018	32	0.8539	32.56	0.3686
C-6	0.022	41	1.065	41	0.757	27.4	0.626	27.4	0.626	0.017	>100	0.721	32.56	0.3686

Since the scope of this study is focusing on the identical samples, the CSR will be the factor that will affect the test results. The stress-normalized cyclic shear strengths ($CSR = \tau/\sigma'_{vo}$) versus number of uniform cycles are presented for different target stress strains including 1%, 3%, and 5% in Figures 4.3 to 4.5. Strength-normalized cyclic shear strength (τ/S_u) are plotted for the same target shear strains in Figures 4.6 to 4.8. The development of pore pressure ratios ($r_u = \text{excess pore water pressure} / \sigma'_{vo}$) versus progression of cyclic loads are plotted in Figure 4.9. By studying tables 4.2 to 4.4 and Figure 4.3 to 4.9, it appeared that there is an effect of CSR on the excess pore water pressure of fine-grained soils. As can be seen, the excess pore water pressure r_u and the number of cycles to failure increase with decreasing the CSR. Thus, the highest number of cycles was reached when $CSR = 0.12$ and r_u was almost equal to 1. The reason for that is because the lower CSR required more loading cycles to achieve the failure criteria than the higher CSR. This concept was demonstrated by De Alba et al. (1976). De Alba used a shaking table to study CSR's effect on the liquefaction trigger as shown in Figure 4.2.

Figure 4.2: The CSR required to reach initial liquefaction ($r_u = 100\%$), from shaking table tests by De Alba et al. (1976)

- Beaverton/OR, A-1 ■ Beaverton/OR, A-2 ■ Beaverton/OR, A-3
- Beaverton/OR, A-4 ■ Beaverton/OR, A-5 ▲ Beaverton/OR, B-1
- ▲ Beaverton/OR, B-2 ▲ Beaverton/OR, B-3 ▲ Beaverton/OR, B-4
- × Beaverton/OR, C-1 × Beaverton/OR, C-2 × Beaverton/OR, C-3
- × Beaverton/OR, C-4 × Beaverton/OR, C-5 × Beaverton/OR, C-6

Figure 4.3: 1% Single Amplitude Shear Strain τ_{cyc} increased 9% for equivalent 0.1Hz loading

Figure 4.4: 3% Single Amplitude Shear Strain τ_{cyc} increased 9% for equivalent 0.1Hz loading

- Beaverton/OR, A-1 ■ Beaverton/OR, A-3 ■ Beaverton/OR, A-4
- Beaverton/OR, A-5 ▲ Beaverton/OR, B-1 ▲ Beaverton/OR, B-3
- ▲ Beaverton/OR, B-4 × Beaverton/OR, C-4 × Beaverton/OR, C-5

Figure 4.5: 5% Single Amplitude Shear Strain τ_{cyc} increased 9% for equivalent 0.1Hz loading

Figure 4.6: 1% Single Amplitude Shear Strain τ_{cyc} increased 9% for equivalent 0.1Hz loading S_u measured from static DSS at 10% shear strain.

Figure 4.7: 3% Single Amplitude Shear Strain τ_{cyc} increased 9% for equivalent 0.1Hz loading S_u measured from static DSS at 10% shear strain.

- Beaverton/OR, A-1 ◆ Beaverton/OR, A-3 ■ Beaverton/OR, A-4
■ Beaverton/OR, A-5 ▲ Beaverton/OR, B-1 ▲ Beaverton/OR, B-3
▲ Beaverton/OR, B-4 × Beaverton/OR, C-4 × Beaverton/OR, C-5

Figure 4.8: 5% Single Amplitude Shear Strain τ_{cyc} increased 9% for equivalent 0.1Hz loading. S_u measured from static DSS at 10% shear strain.

Figure 4.9: Pore water pressure buildup

Also, the results from the cyclic shear loading tests show that the Beaverton silts were transitional silts between sand-like to clay-like behavior as shown in Figure 4.10.

Figure 4.10: Beaverton silts' stress-strain loops

4.3 Post Cyclic Shear Loading

The results of the post cyclic shear loading tests are shown in Figures 4.10 to 4.14. The post cyclic shear test helps determining the strength of sample after shaking. As stated in chapter 3, the post cyclic shear loading tests were performed on 11 samples shear strain with a 0.05/h and maximum force of 10kN. At 10% shear strain, the shear resistance was determined for each test. By comparing the results of shear resistance values at 10% shear strain for post cyclic shear loading and the monotonic shear loading tests, all the post cyclic shear resistance values are less than the monotonic shear resistance value which was 32.56 kPa at 10% shear strain. The main reason is that the soil samples loss strength during cyclic stage. For sample C-6, since it did not reach failure and the cyclic

strain is only 1.2%, the soil strength has not loosed much strength as other specimens.

The results are listed in Table 4.5.

Table 4.5: Post cyclic shear loading values.

Sample ID	CSR	$S_{u,post-cyc}$ (kPa)	$S_{u,post-cyc} / \sigma'_{vc}$
A-2	0.2	17.65	0.1765
A-3	0.2	18.74	0.1874
A-4	0.2	21.26	0.2126
A-5	0.2	16	0.16
B-2	0.15	19.18	0.1918
B-3	0.15	13.34	0.1334
B-4	0.15	15.62	0.1562
C-3	0.12	14.36	0.1436
C-4	0.12	17.54	0.1754
C-5	0.12	12	0.12
C-6	0.12	28.8	0.288

Figure 4.11: Stress normalized resistance vs shear strain.

Figure 4.12: Static S_u measured from static DSS at 10% shear strain vs shear strain.

- A-2, (Max cyclic strain 4.3%, Max Ru 0.635)
- A-3, (Max cyclic strain 6.3%, Max Ru 0.715)
- A-4, (Max cyclic strain 5.5%, Max Ru 0.691)
- A-5, (Max cyclic strain 6.5%, Max Ru 0.723)
- ▲— B-2, (Max cyclic strain 3.75%, Max Ru 0.867)
- ▲— B-3, (Max cyclic strain 6%, Max Ru 0.788)
- ▲— B-4, (Max cyclic strain 5.2%, Max Ru 0.809)
- x— C-3, (Max cyclic strain 4%, Max Ru 0.831)
- x— C-4, (Max cyclic strain 5.2%, Max Ru 0.886)
- x— C-5, (Max cyclic strain 6%, Max Ru 0.854)
- x— C-6, (Max cyclic strain 1.2%, Max Ru 0.721)

Figure 4.13: $\tau_{\text{post-cyclic at } x\% \text{ strain}} / \tau_{\text{static at } x\% \text{ strain}}$ vs shear strain.

Figure 4.14: Su_{st} measured from static DSS at 10% shear strain $Su_{post-cyc}$ measured from post-cyclic DSS at 10% shear strain vs max shear strain.

Figure 4.15: $S_{u,st}$ measured from static DSS at 10% shear strain $S_{u,post-cyc}$ measured from post-cyclic DSS at 10% shear strain vs max ru.

5. DISCUSSION

5.1 Void Ratio Variation

As mentioned before, the specimens' preparation goal was to prepare identical samples to get identical results. However, the results in some of these tests were not identical to other results even though the test parameters were the same, so what was the problem? It seems that there was a variation in void ratio data during consolidation as shown in Figure 5.1. This figure represents the minimum, maximum, 25 percentile, median, and 75 percentile values for the initial void ratio, after 25kPa consolidation, after 50kPa consolidation (using the CDSS software), after 100 kPa, final void ratio and CDSS reported void ratio. The variation in the void ratio increases as the consolidation levels increase, which means the samples were successfully performed identically, but some factors affected the results. Table 5.1 presents the statistical values for these consolidation levels.

Table 5.1: Statistical values for different consolidation levels

	Initial Void Ratio e_0	After 25kPa Consolidation	After 50kPa Consolidation	After 100kPa Consolidation	Final Void Ratio e_f	CDSS Reported Void Ratio
Minimum	0.983	0.695	0.553	0.551	0.555	0.519
25% tile	1.056	0.768	0.644	0.618	0.628	0.601
Median	1.065	0.797	0.700	0.653	0.672	0.634
75% tile	1.068	0.802	0.727	0.694	0.696	0.685
Maximum	1.074	0.810	0.820	0.761	0.753	0.792
Mean	1.056	0.782	0.687	0.651	0.663	0.640
Standard deviation	0.023	0.031	0.069	0.057	0.061	0.067
COV	0.022	0.039	0.100	0.088	0.091	0.105

Figure 5.1: Void ratio variation per consolidation levels

As can be seen from the figure, the variation in the void ratio started to increase when transferring the samples from the consolidometer (after 25kPa) to the CDSS device (before 50 kPa). This increased in variety stays constant to the end of the test. Thus, the problem occurred when the sample moved from one lab to another as the slurry sample is super sensitive. Moving the slurry sample many times can affect the sample quality, which leads to change the results. There is some updating on the sampling preparation procedures that can be done to try avoiding the variation in the void ratio. First, the

sample should be prepared in the lab that has the consolidometer to prevent moving the sample between two labs. Also, the void ratio should be determined before and after every consolidation level. The void ratio can be measured before and after the 5 kPa and 25 kPa manually. It can also be done manually before applying the 50 kPa consolidation stress and after the test. However, after 50 kPa and after 100 kPa before the cyclic stage, the void ratio can be determined using the CDSS software. Updating these procedures helps avoid having the variation in void ratio and keeping the sample's quality elevated.

5.2 Sample's Quality

According to Green and Marek (2019) there are several factors that can affect the quality of the CDSS tests especially because there is no standard ASTM that can be checked. Thus, they tried to identify these factors and mark the test as a “high” quality test and a “questionable” test. Some factors that can affect the sample quality are shear stress or shear strain ramp-up during consolidation phase. This means that the specimen can be subjected to shear strain (γ) during consolidation before cyclic phase which affects the liquefaction resistance of the specimen. The other factors can be negative pore water pressure and axial strain that is larger than 0.05%. Per ASTM D6528 the acceptable axial strain during monotonic shear loading test cannot exceed 0.05% to avoid having problems when measured vertical effective stress (σ'_v) and shear stress (τ) at failure. In table 5.2, Green and Marek factors were used to assign (H) for high quality tests and (Q) for questionable tests.

Table 5.2: Sample's quality

Sample ID	CSR	Number of Cycles to Reach 6% Peak to Peak	Void Ratio After Test (e _r)	Sample's Quality	Criteria Used for Determining Sample's Quality
A-1	0.2	2.14	0.5777	Q	Axial strain > 0.05%
A-2	0.2	1.5	0.7013	Q	Negative PWP & Axial strain > 0.05%
A-3	0.2	2	0.652	Q	Axial strain > 0.05%
A-4	0.2	3.6	0.6122	H	
A-5	0.2	3.41	0.5513	H	
B-1	0.15	17	0.687	Q	Negative PWP & Axial strain > 0.05%
B-2	0.15	16.3	0.76	Q	Negative PWP & Axial strain > 0.05%
B-3	0.15	7.46	0.628	Q	Axial strain > 0.05%
B-4	0.15	8.52	0.72	Q	PWP reduced before start increasing
C-1	0.12	49	0.7012	H	
C-2	0.12	20.3	0.5962	Q	Axial strain > 0.05%
C-3	0.12	31	0.6776	Q	Negative PWP & Axial strain > 0.05%
C-4	0.12	61	0.623	Q	Negative PWP & Axial strain > 0.05%
C-5	0.12	30	0.655	Q	Negative PWP & Axial strain > 0.05%
C-6	0.12	100	0.6252	H	Reaches 100 cycles before reaching peak to peak

After splitting the tests to high quality and questionable test, the method described in Idriss and Boulanger (2008) was used to identify the projected CRR at N= 15 cycles using equation (5-1).

$$CRR = a \cdot N^{-b} \quad (5-1)$$

The projected CRR is the CSR required to reach liquefaction in a specified number of loading cycles, where N number of cycles =15 and “a” and “b” are fitting parameters which describe the slope of the power fit relationships using excel (these relationships are also shown in Figure 4.3 to 4.8). The projected CRR, a and b parameters for high-quality tests, and questionable tests are shown in Table 5.3 and d 5.4 and Figures 5.2 to 5.5.

Table 5.3: $CRR_{N=15}$ for high quality tests

Sample ID	CS R	Number of Cycles to Reach 6% Peak to Peak	Void Ratio After Test (e_f)	b	a	Projected CRR (N=15)
A-4	0.2	3.6	0.6122	0.194	0.2549	0.152
A-5	0.2	3.41	0.5513	0.194	0.2549	0.150
C-1	0.1 2	49	0.7012	0.194	0.2549	0.151
					Average :	0.151

Figure 5.2: High quality tests' CSR vs N to 6% peak-peak shear strain

Figure 5.3: High quality tests' $CRR_{N=15}$ vs final void ratio

Table 5.4: $CRR_{N=15}$ for questionable tests

Sample ID	CS R	Number of Cycles to Reach 6% Peak to Peak	Void Ratio After Test (e_f)	b	a	Projected CRR (N=15)
A-1	0.2	2.14	0.5777	0.147	0.2126	0.150
A-2	0.2	1.5	0.7013	0.147	0.2126	0.143
A-3	0.2	2	0.652	0.147	0.2126	0.149
B-1	0.15	17	0.687	0.147	0.2126	0.153
B-2	0.15	16.3	0.76	0.147	0.2126	0.152
B-3	0.15	7.46	0.628	0.147	0.2126	0.135
B-4	0.15	8.52	0.72	0.147	0.2126	0.138
C-2	0.12	20.3	0.5962	0.147	0.2126	0.125
C-3	0.12	31	0.6776	0.147	0.2126	0.134
C-4	0.12	61	0.623	0.147	0.2126	0.147
C-5	0.12	30	0.655	0.147	0.2126	0.133
C-6	0.12	100	0.6262	0.147	0.2126	0.159
					Average	0.143

Figure 5.4: Questionable tests' CSR vs N to 6% peak to peak shear strain

Figure 5.5: Questionable tests' $CRR_{N=15}$ vs final void ratio

The comparison of the results of projected $CRR_{N=15}$ is shown in Figure 5.6. it is approximated that the projected $CRR_{N=15}$ for high quality test are higher than the questionable tests $CRR_{N=15}$. It also shown that the a and b parameters for the questionable tests are lower than those parameters for the high-quality tests. This method is used to compare behaviors of cyclic loading applied during the test. According to Boulanger and Idriss (2014), parameter b is increasing with increasing the soil disturbance and density. This means that as the void ratio increase, the b parameter decreases. The $CRR_{N=15}$ vs final void ratio relationship is submitted in Figure 5.7. The average void ratio for high quality tests is 0.62 where the final void ratio of the questionable tests is 0.66. Thus, it makes sense to have a lower b value for questionable tests. However, the difference between the projected CRR for high quality tests and

Projected CRR for questionable tests is only 5% which means that the questionable tests results are reliable.

Figure 5.6: Compared projected $CRR_{N=15}$

Figure 5.7: Projected $CRR_{N=15}$ vs final void ratio.

5.3 Comparison of Projected CRR for 15 Cycles

The results of this study were compared with 2 other researches. The first study was done by Adam Price (2017). He used a loose slurry of non-plastic silt specimens to evaluate and study cyclic strength behavior with multiple cyclic shearing and reconsolidation stages using a direct, simple shear test. The other study was performed by Melissa Preciado, who is a master student at Portland State University. Preciado used the same soil that has been used in this project, but she tested it differently. She tested her samples using the same method as Price, which was a “consecutive testing method.” A comparison of projected CRR for 15 cycles versus the final void ratio is shown in Figure 5.8. The $CRR_{N=15}$ is dependent on the slope of the relationship between CSR and the number of cycles to failure.

Figure 5.8: Comparison of projected $CRR_{N=15}$ vs the final void ratio

Figure 5.8 clearly shows that for Price and Preciado specimens, the projected CRR in each repeatable sample increased as the void ratio decrease. However, for high quality and questionable tests, the projected CRR values are almost constant for all the

specimens. These projected CRR that came from individual test results are similar to Price's and Preciado's initial project CRR values for each test (virgin samples). However, for Preciado's and Price's tests, the CRR is increasing when they retested their sample again, and their void ratio is reducing with this increase. Further research can be done to understand why there is no effect of the void ratio initially for the virgin point with projected CRR. Though, when retested the soil sample, the $CRR_{N=15}$ increases with reduces the void ratio.

5.4 Comparison of b-values

As mentioned before, “b” is a parameter that describes the slope of the power fit relationship. This parameter is being applied to compute the project CRR values for 15 cycles. Figure 5.9 illustrates the relationship between Projected CRR and parameter b for this project data and Price data and compares the results with Boulanger and Idriss (2014). Idriss and Boulanger curves represent the variation of b with CRR based on a wide range of fine content and plasticity index.

Figure 5.9: $CRR_{N=15}$ vs parameter b

This comparison helps to identify that the results of high-quality tests are reliable since these results within the range of Adam Price (2017) and Boulanger and Idris (2014). On the other hand, the b values for the questionable tests are lower than other results because of the increased of the sample disturbance.

6. CONCLUSION & RECOMMENDATION FOR FUTURE WORKS

This research's scope was preparing identical samples and test them using different CSR.

The results showed that as the CSR increases, the number of loading cycles to failure decrease. Also, it represented that performing identical samples is not sufficient to study the behavior of fine-grained soils as there are many factors that can affect the results.

A magnitude 9 earthquake is expected to trigger in the pacific northwest because of the Cascadia subduction zone. Therefore, further researches should be done to try limiting the consequences of this earthquake. Some of the recommendation of future researches are listed:

- Understanding the fundamental behavior of fine-grained soils needs further research. One potential future study is to investigate the effects of shear loading frequency on an undrained cyclic shear response of low plasticity silts to tests silts sample at 1Hz, 0.33Hz, and 0.1HZ at isotopically initial confining stress.
- Another potential future study is to determine the relationship between undrained cyclic shear resistance (CRR) and over consolidation ratio (OCR) for silt specimens obtained from different sites in the Pacific Northwest. The approach of this project will be performed monotonic and cyclic shear tests to characterize undrained shear strength and shear resistance of each site using constant volume (DSS) device by Geocomp.
- In addition, future research can focus to investigate the liquefaction susceptibility in interlayered soil and the effect on CPT data interpretation.

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APPENDIX A

Set-up of Cyclic Direct Simple Shear Machine (CDSS) - Geocomp

Prepared by Melissa Preciado, Max Miller, and Rawan Almoumen

A.1 Before the Test

1. Prepare the sample as required by the test (remolded vs. undisturbed).

Remolded sample: Ensure membrane and the bottom O-rings are making good contact; there are grooves on the bottom top cap for the Bottom O-rings to fit snugly. Place aligned Teflon rings around the membrane, then place the membrane around the suction ring, ensuring the membrane is touching the inside of the stacked Teflon rings. Make sure the horizontal screw on the top cap is loose, this allows the piston to slide into the top cap.

Notes: For remolded samples, it is advised to always use two o-rings on the bottom plate and two rings on the top cap. If consolidation pressures are too high, it can push the soft soil out of a one o-ring set up.

Undisturbed sample: Place undisturbed sample centered on top of the bottom cap. Wrap membrane around the suction ring and ensure it is against the inner wall of the suction ring. Place the membrane around the undisturbed sample making sure not to touch the sample during the process. Then place the bottom O-ring around one of the grooves on the bottom cap, using the suction ring.

A.1.1 Important Measurements

These need to be taken *prior* to testing since they are the input parameters for the CDSS software.

Total mass of the soil:

- Undisturbed samples can be done directly on the scale.
- For remolded samples it is best to weigh the mass of the set-up (without the remolded soil), then add the remolded soil into the set-up, measure again, and then subtract the mass of the set-up.

Water content: Measure water content of the soil that will be tested from sample trimmings or slurry.

Water Content (%) = [*weight of moist soil (g) – weight of dry soil (g)*]/ *weight of dry soil (g)*

Sample height: Take the initial soil height. As of July 2020, there are three sample setups (AMP, RTA, KRS) with different top cap heights and bottom plate heights. Take the height of the set up (Bottom plate, soil sample, and top cap) and subtract the corresponding set-up height. The height needs to be measured *after* the device has been calibrated and the sample placed in the machine. Calibration instructions are provided in the following sections

Figure A1. Shear box components. Top (left to right): bottom plate with bottom porous stone attached, note the vertical bolts on the bottom left and top right of the bottom plate. Black O-rings. Bottom (left to right): Teflon stackable rings, top cap with attached porous stone, Teflon ring pins, suction ring with vacuum hose attachment.

Figure A2. Geocomp's ShearTrack-II.

- A. LVDT
- B. Four prong nuts (4-star knobs)
- C. Piston Cap screws
- D. Bolts for lowering and lifting arm
- E. Large Piston Screw
- F. Shear box T-bolts
- G. Water bath box
- H. Horizontal control keypad

A.2 Setting Up the Sample in the CDSS

1. Open the CDSS software on the computer. Then make sure the System Monitor and Calibration windows are side by side. At this point, with the bath box centered and the crossbar hanging with no load, your System monitor should read '0' load in the Vertical Load and Horizontal load outputs. If that is not the case, you can tell the device there are no loads at this point by typing in the "counts," a five-digit number shown in the System Monitor next to the load, to the Calibration window. Press [Okay], then [Apply].
2. If the test requires the sample to be submerged, add water to the bath halfway so as not to overflow when adding the shear box assembly.
3. Ensure the sample's top cap horizontal bolt (green in figure 3) is loose before placing a sample in the CDSS device. When placing the sample into the bath box, make sure the bottom plate is pushed to the right and fit snug, it should not wiggle out of place. The large vertical bottom plate bolts (Figure 2) should always be on the *top left* and *bottom right* sides of the bath box.
4. Tighten bottom plate to the shear box (bath box) using T-bolts [F]. Ensure the bath box is centered before continuing onto the next step.

5. Raise the vertical threads for the load cell crossbar to the highest position before each test. This will allow your sample to consolidate without hitting bottom vertical limits, therefore ending your test too early. You can raise the crossbar by pressing 2-Position, then 4-Jog on the vertical control panel of the CDSS.
6. Lower arm with the piston. Tighten arm bolts [D]. If the top cap hole does not line up with the piston significantly (more than a couple of millimeters), turn the shear box 180 degrees (making sure the large shear box screws are on the *top left* and *bottom right* sides of the bath box. Use the Horizontal control panel to align the shear box with the piston (2-Position, then press 4-Jog) while ensuring the bath box remains within the CENTER boundaries that have been labeled in white labels on the PSU CDSS device. The *Position* and *Jog* functions should be used to make final adjustments and to make sure the sample is fully centered and aligned with the piston.
7. Lower piston onto the top cap, then secure piston by tightening the adjacent screw [E]. This will keep any unwanted load from transferring to your sample while you are finishing up the device setup. Keep this screw tightened until right before you hit RUN on your test.
8. At this point, screws/bolts F, D, and E should have been tightened in that order. You will now make sure the top cap is around the piston tightly. You need to:
 - Tighten the red screws, as seen below. Tighten top cap bolt (green, see below)
 - Tighten piston cap bolts. These screws connect the top and bottom portions of the top cap together.

Figure A3. Schematic for top cap set up.

9. Now fill in all necessary information about your project, sample inputs, loading conditions, and other necessary information onto the software:

- PROJECT tab: Fill in any relevant project information such as the project name/number, location of soil tested, preparation, test date, etc.
- SPECIMEN tab: Measurements taken in STAGE 1 are inserted here. The Initial height will be the last item to fill in since it is necessary to take the H2 measurement after calibration.
- WATER CONTENT tab: Fill in the necessary information obtained from A.1 (Left side only)
- READ TABLE tab: you **do not** need to change anything in this tab.
- TEST PARAMETERS: Choose which test you will begin with from the options given.
- CONSOLIDATION TABLE tab: fill out the consolidation parameters you wish to apply to the soil sample.

- CYCLIC TABLE: fill out the cyclic parameters you wish to apply to the soil sample (make sure to choose stress or strain control test).
- SHEAR TABLE: fill out any shear parameters you wish to apply to the soil sample.
- (if you have the CYCLIC TABLE filled out, this step will be post-cyclic shear. You can perform a monotonic DSS test (MDSS) by making sure the CYCLIC table does not have any rows filled.

10. Save! Save Save!

11. Open the **Calibration** window and observe how your loads change as you tighten the screws you will be directed to tighten below. Be sure to follow the tightening sequence provided in this document as doing so can decrease the chance of you applying incorrect loads!

- At this point, the piston bolt should be tightened to keep loads being transferred onto the sample.
- Adjust the lower crossbar hex nuts to make sure your crossbar is making full contact with the piston.
- Turn the lower hex nuts two rotations, leaving < 1mm gap between the load cell and the piston. You can check you have a gap by sliding a piece of paper between the load cell and the piston, and it should move freely.
- Place the piston cap and crew it onto the load cell. These should be tight and not wiggle. Look at your System Monitor window as you do this. You do not want to over-tighten the cap or have loads larger than |8lbs| showing on the monitor.

- At this point, lower the 4-star knobs [B] to secure the crossbar down. Make sure your sample is level. Again, look at the System monitor to make sure you do not over-tighten the 4 Star knobs. Your load should not read higher than |8lbs|.
- Place the LVDT on the rod and secure it in place with the 4-Star knob.
- Release the arm screw [E]. Your load in the System monitor should be lower than 1lb before starting your test. With experience, you will learn how hard to tighten the screws to prevent your loads higher than 1lb when releasing screw [E]. It is suggested you practice this method or dummy samples until you feel confident with the procedures, and you are consistently achieving the <1lb load threshold before starting each test.

A.3 Notes from Geocomp

Before every test make sure to check the following settings:

1. Hardware Setup:

Go to: Options > Hardware and the following window shows up:

Hardware Setup

Actuator Port: COM1

Vertical Motor

Node ID: 101

Step Multiplier: 3.89e-07 in/step

Horizontal Motor

Node ID: 102

Step Multiplier: 3.25520833333333 in/step

☒ Enable FB

Control Loop Period: 250 msec

Ok Cancel Apply Help

Make sure the Enable FB box is checked. Click Apply.

2. Drive Settings:

Go to Options > Drive. The following window shows up:

The Drive Settings dialog box contains the following fields and controls:

- Encoder: 4096 (dropdown menu) counts/turn
- Pulley Ratio: 1.5 (text box) motor/screw
- Screw Pitch: 5 (text box) turn/in
- P: (text box) I: (text box) D: (text box)
- Drive: (text box) Disable Enable
- Fault: (text box) Clear
- Buttons: Close Scan Apply Help

Click Scan, then Apply, then Close.

***If the following error message pops up two things might be the problem:

1. *The cable connecting the CDSS to the computer might be loose*
2. *A test is in process*

3. Check Constant Volume Gain- CVG- (based on soil type and trial/error): This value will need to be adjusted if the test results show an axial strain outside -0.005 to 0.005%.

Cyclic Control:	Strain	Desired Response Gain:	3	Max. Mean Horz. Strain:	5	%	
Normal Control:	Constant Volume	Gain Update Period:	0.032	Cycles	Max. Total Horz. Strain:	5	%
Constant Volume Gain:	1	Gain Update Window:	0.25	Cycles	Filter Norm. Cutoff Freq.:	0.25	

Below is an example of what a good value chosen for CVG looks like in the report:

4. Desired Response Gain- DRG- (Based on soil stiffness, stiff soils typ. 2, soft soils 6-8):

A good value for DRG is determined by observing the sinusoidal waves during the test (View> test graph). A good value would show a sinusoidal wave like the ones below.

Cyclic Control:	Strain	Desired Response Gain:	3	Max. Mean Horz. Strain:	5	%	
Normal Control:	Constant Volume	Gain Update Period:	0.032	Cycles	Max. Total Horz. Strain:	5	%
Constant Volume Gain:	1	Gain Update Window:	0.25	Cycles	Filter Norm. Cutoff Freq.:	0.25

5. As to why we might see the silt tests have a preference to the left:

Artur mentioned our samples may not be consolidated enough, he suggests increasing the time of consolidation, AND making sure step 1 (above) of the Hardware Setup is completed.

6. Additional Comments:

- Minimum strains the device can capture are 1/2000"
- Maximum strain 5%, sometimes even 10%
- Maximum axial strain should be less than 0.5%.
- Readings per cycle were reduced from 512 readings to 128. He says 512 is excessive.

APPENDIX B

Individual Tests Results

B.1 Cyclic Shear Loading Tests Results (CSR=0.2A-1)

Figure B1: Cyclic debug for sample A-1

Figure B2: Cyclic data for sample A-1

Figure B3: Cyclic modulus/damping results for sample A-1

	Project Name: Liquefaction Sills Boring Number: Sample Number: CSR 0.2A-1 Test Number: Description: Remolded silt de-aired water used Remarks:	Location: Beaverton, OR Tester: Rawan Test Date: 02/19/2020 Preparation: Remolded	Project Number: Checker: Depth: Elevation:
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Figure B4: Cyclic stress strain results for sample A-1

Figure B5: Cyclic strain results for sample A-1

B.2 Cyclic Shear Loading Tests Results (CSR=0.2A-2)

Figure B6: Cyclic debug for sample A-2

Figure B7: Cyclic data for sample A-2

Cyclic Simple Shear Test

Cyclic Modulus/Damping Results

Step 1 of 1

	Project Name: Liquefaction Silt	Location: Beaverton, OR	Project Number:
	Boring Number:	Tester: Rawan	Checker:
	Sample Number: CSR=0.2A-2	Test Date: 02/28/2020	Depth:
	Test Number:	Preparation: Remolded	Elevation:
	Description: Remolded silt, de-aired water used		
	Remarks:		

2020-07-28 16:37:40

3.0.18.220 / 3.0.18.220

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Figure B8: Cyclic modulus/ Damping results for A-2

Figure B9: Cyclic stress strain results for sample A-2

Figure B10: Cyclic strain results for sample A-2

B.3 Cyclic Shear Loading Tests Results (CSR=0.2A-3)

Figure B11: Cyclic debug for sample A-3

Cyclic Simple Shear Test

Cyclic Data
Step 1 of 1

	Project Name: Liquefaction Silt	Location: Beaverton, OR	Project Number:
	Boring Number:	Tester: Rawan	Checker:
	Sample Number: CSR=0.2A-3	Test Date: 03/6/2020	Depth:
	Test Number:	Preparation: Remolded	Elevation:
	Description: Remolded silt, de-aired water used		
	Remarks:		

2020-07-28 16:53:03

3.0.18.220 / 3.0.18.220

4

Figure B12: Cyclic data for sample A-3

Cyclic Simple Shear Test

Cyclic Modulus/Damping Results

Step 1 of 1

	Project Name: Liquefaction Silt	Location: Beaverton, OR	Project Number:
	Boring Number:	Tester: Rawan	Checker:
	Sample Number: CSR=0.2A-3	Test Date: 03/6/2020	Depth:
	Test Number:	Preparation: Remolded	Elevation:
	Description: Remolded silt, de-aired water used		
	Remarks:		

2020-07-28 16:53:43

3.0.18.220 / 3.0.18.220

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Figure B13: Cyclic modulus/damping results for sample A-3

Figure B14: Cyclic stress strain results for sample A-3

Cyclic Simple Shear Test

Cyclic Strain Results

Step 1 of 1

Cycle 0.0 to 50.0

Project Name: Liquefaction Silt	Location: Beaverton, OR	Project Number:
Boring Number:	Tester: Rawan	Checker:
Sample Number: CSR=0.2A-3	Test Date: 03/6/2020	Depth:
Test Number:	Preparation: Remolded	Elevation:
Description: Remolded silt, de-aired water used		
Remarks:		

2020-07-28 16:59:29

3.0.18.220 / 3.0.18.220

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Figure B15: Cyclic strain results for sample A-3

B.4 Cyclic Shear Loading Tests Results (CSR=0.2A-4)

Figure B16: Cyclic debug for sample A-4

Figure B17: Cyclic data for sample A-4

Figure B18: Cyclic modulus/damping results for sample A-4

Figure B20: Cyclic strain results for sample A-4

B.5 Cyclic Shear Loading Tests Results (CSR=0.2A-5)

Figure B21: Cyclic debug for sample A-5

Figure B22: Cyclic data for sample A-5

Figure B23: Cyclic modulus/damping results for sample A-5

Figure B24: Cyclic stress strain results for sample A-5

	Project Name: Liquefaction Silt	Location: Beaverton, OR	Project Number:
	Boring Number:	Tester: Rawan	Checker:
	Sample Number: CSR=0.2A-5	Test Date: 03/17/2020	Depth:
	Test Number:	Preparation: Remolded	Elevation:
	Description: Remolded silt, de-aired water used		
	Remarks:		

Figure B25: Cyclic strain results for sample A-5

B.6 Cyclic Shear Loading Tests Results (CSR=0.15B-1)

Figure B26: Cyclic debug for sample B-1

Figure B27: Cyclic data for sample B-1

Figure B28: Cyclic modulus/damping results for sample B-1

B.7 Cyclic Shear Loading Tests Results (CSR=0.15B-2)

Figure B31: Cyclic debug for sample B-2

Figure B32: Cyclic data for sample B-2

Figure B33: Cyclic modulus/damping results for sample B-2

B.8 Cyclic Shear Loading Tests Results (CSR=0.15B-3)

Figure B36: Cyclic debug for sample B-3

Figure B37: Cyclic data for sample B-3

Figure B38: Cyclic modulus/damping results for sample B-3

Figure B39: Cyclic stress strain results for sample B-3

B.9 Cyclic Shear Loading Tests Results (CSR=0.15B-4)

Figure B41: Cyclic debug for sample B-4

Cyclic Simple Shear Test

Cyclic Data

Step 1 of 1

Project Name: Liquefaction Silt	Location: Beaverton, OR	Project Number:
Boring Number:	Tester: Rawan	Checker:
Sample Number: CSR=0.15B-4	Test Date: 03/13/2020	Depth:
Test Number:	Preparation: Remolded	Elevation:
Description: Remolded silt, de-aired water used		
Remarks:		

2020-07-28 18:05:33

3.0.18.220 / 3.0.18.220

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Figure B42: Cyclic data for sample B-4

Cyclic Simple Shear Test

Cyclic Modulus/Damping Results

Step 1 of 1

Project Name: Liquefaction Silt	Location: Beaverton, OR	Project Number:
Boring Number:	Tester: Rawan	Checker:
Sample Number: CSR=0.15B-4	Test Date: 03/13/2020	Depth:
Test Number:	Preparation: Remolded	Elevation:
Description: Remolded silt, de-aired water used		
Remarks:		

2020-07-28 18:06:00

3.0.18.220 / 3.0.18.220

5

Figure B43: Cyclic modulus/damping results for sample B-4

B.10 Cyclic Shear Loading Tests Results (CSR=0.12C-1)

Figure B46: Cyclic debug for sample C-1

Figure B47: Cyclic data for sample C-1

Figure B48: Cyclic modulus/damping results for sample C-1

B.11 Cyclic Shear Loading Tests Results (CSR=0.12C-2)

Figure B51: Cyclic debug for sample C-2

Figure B52: Cyclic data for sample C-2

Figure B53: Cyclic modulus/damping results for sample C-2

Cyclic Simple Shear Test
Cyclic Stress Strain Results
Step 1 of 1
Cycle 0.0 to 50.0

Project Name: Liquefaction Silt	Location: Beaverton, OR	Project Number:
Boring Number:	Tester: Rawan	Checker:
Sample Number: CSR=0.12C-2	Test Date: 02/24/2020	Depth:
Test Number:	Preparation: Remolded	Elevation:
Description: Remolded silt, de-aired water used		
Remarks:		

2020-07-29 02:51:10

3.0.18.220 / 3.0.18.220

6

Figure B54: Cyclic stress strain results for sample C-2

Figure B55: Cyclic strain results for sample C-2

B.12 Cyclic Shear Loading Tests Results (CSR=0.12C-3)

Figure B56: Cyclic debug for sample C-3

Figure B57: Cyclic data for sample C-3

Figure B58: Cyclic modulus/damping results for sample C-3

Cycle 0.0 to 50.0

	Project Name: Liquefaction Silt Boring Number: Sample Number: CSR-0 12C-3 Test Number: Description: Remolded silt, de-aired water used Remarks:	Location: Beaverton, OR Tester: Rawan Test Date: 02/26/2020 Preparation: Remolded	Project Number: Checker: Depth: Elevation:
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2020-07-29 03:00:27

3.0.18.220/3.0.18.220

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Figure B60: Cyclic strain results for sample C-3

B.13 Cyclic Shear Loading Tests Results (CSR=0.12C-4)

Figure B61: Cyclic debug for sample C-4

Figure B62: Cont. cyclic debug for sample C-4

Figure B63: Cyclic data for sample C-4

Figure B64: Cont. cyclic data for sample C-4

Figure B65: Cyclic modulus/damping results for sample C-4

Figure B65: Cont. cyclic modulus/damping results for sample C-4

The figure consists of two side-by-side plots. The left plot shows Shear Stress (kPa) on the y-axis (ranging from -20 to 20) versus Shear Strain (%) on the x-axis (ranging from -10 to 10). The data points form a dense, elongated, and slightly curved band, indicating a shear band. The right plot shows Normal Stress (kPa) on the y-axis (ranging from -20 to 20) versus Normal Stress (kPa) on the x-axis (ranging from 0 to 200). The data points form a dense, elongated, and slightly curved band, indicating a shear band.

	Project Name: Liquefaction Silt	Location: Beaverton, OR	Project Number:
	Boring Number:	Tester: Rawan	Checker:
	Sample Number: CSR-0.12C-4	Test Date: 03/4/2020	Depth:
	Test Number:	Preparation: Remolded	Elevation:
	Description: Remolded silt, de-aired water used		
	Remarks:		

3.0.18.220/3.0.18.220

Figure B67: Cyclic stress strain results for sample C-4

Figure 10 consists of two plots. The top plot shows Shear Stress (kPa) on the y-axis (ranging from -20 to 20) versus Shear Strain (%) on the x-axis (ranging from -10 to 10). The plot displays a hysteresis loop, with the loading curve (upper curve) and the unloading curve (lower curve) showing a non-linear relationship. The bottom plot shows Normal Stress (kPa) on the y-axis (ranging from -20 to 20) versus Normal Strain (%) on the x-axis (ranging from 0 to 200). The plot displays a hysteresis loop, with the loading curve (upper curve) and the unloading curve (lower curve) showing a non-linear relationship.

	Project Name: Liquefaction Silt Boring Number: Sample Number: CSR-0, 12C-4 Test Number: Description: Remolded silt, de-aired water used Remarks:	Location: Beaverton, OR Tester: Rawan Test Date: 03/4/2020 Preparation: Remolded	Project Number: Checker: Depth: Elevation:
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3.0.18.220 / 3.0.18.220

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Cyclic Simple Shear Test

Cyclic Strain Results

Step 1 of 1

Cycle 0.0 to 50.0

Project Name: Liquefaction Silt	Location: Beaverton, OR	Project Number:
Boring Number:	Tester: Rawan	Checker:
Sample Number: CSR=0.12C-4	Test Date: 03/4/2020	Depth:
Test Number:	Preparation: Remolded	Elevation:
Description: Remolded silt, de-aired water used		
Remarks:		

2020-07-29 03:17:34

3.0.18.220 / 3.0.18.220

7

Figure B69: Cyclic strain results for sample C-4

Cyclic Simple Shear Test

Cyclic Strain Results

Step 1 of 1

Cycle 13.0 to 63.0

Project Name: Liquefaction Silt	Location: Beaverfon, OR	Project Number:
Boring Number:	Tester: Rawan	Checker:
Sample Number: CSR=0.12C-4	Test Date: 03/4/2020	Depth:
Test Number:	Preparation: Remolded	Elevation:
Description: Remolded silt, de-aired water used		
Remarks:		

2020-07-29 03:19:15

3.0.18.220 / 3.0.18.220

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Figure B70: Cont. cyclic strain results for sample C-4

B.14 Cyclic Shear Loading Tests Results (CSR=0.12C-5)

Figure B71: Cyclic debug for sample C-5

Figure B72: Cyclic data for sample C-5

Figure B73: Cyclic modulus/damping results for sample C-5

Figure 10 consists of two plots. The left plot shows Shear Stress (kPa) on the y-axis (ranging from -20 to 20) versus Shear Strain (%) on the x-axis (ranging from -10 to 10). The right plot shows Normal Stress (kPa) on the y-axis (ranging from -20 to 20) versus Normal Strain (%) on the x-axis (ranging from 0 to 200). Both plots show multiple red curves representing different stages of the experiment.

	Project Name: Liquefaction Silts	Location: Beaverton, OR	Project Number:
	Boring Number:	Tester: Rawan	Checker:
	Sample Number: CSR-0, 12C-5	Test Date: 03/10/2020	Depth:
	Test Number:	Preparation: Remolded	Elevation:
	Description: Remolded silt, de-aired water used		
	Remarks:		

3.0.18.220/3.0.18.220

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Figure B74: Cyclic stress strain results for sample C-5

7

B.15 Cyclic Shear Loading Tests Results (CSR=0.12C-6)

Figure B76: Cyclic debug for sample C-6

Figure B77: Cont. cyclic debug for sample C-6

Figure B78: Cyclic data for sample C-6

Figure B79: Cont. cyclic data for sample C-6

Figure B80: Cyclic modulus/damping results for sample C-6

Figure B81: Cont. cyclic modulus/damping results for sample C-6

Figure B85: Cont. cyclic strain results for sample C-6

B.16 Post Cyclic Shear Loading Results (CSR=0.2A-2)

Figure B86: Shear strain results for sample A-2

Figure B87: Shear stress results for sample A-2

Figure B88: Shear modulus results for sample A-2

B.17 Post Cyclic Shear Loading Results (CSR=0.2A-3)

Figure B89: Shear strain results for sample A-3

Figure B90: Shear stress results for sample A-3

Figure B91: Shear modulus results for sample A-3

B.18 Post Cyclic Shear Loading Results (CSR=0.2A-4)

Figure B92: Shear strain results for sample A-4

Figure B93: Shear stress results for sample A-4

Figure B94: Shear modulus results for sample A-4

B.19 Post Cyclic Shear Loading Results (CSR=0.2A-5)

Figure B95: Shear strain results for sample A-5

Figure B96: Shear stress results for sample A-5

Figure B97: Shear modulus results for sample A-5

B.20 Post Cyclic Shear Loading Results (CSR=0.15B-2)

Figure B98: Shear strain results for sample B-2

Figure B99: Shear stress results for sample B-2

Figure B100: Shear modulus results for sample B-2

B.21 Post Cyclic Shear Loading Results (CSR=0.15B-3)

Figure B101: Shear strain results for sample B-3

Figure B102: Shear stress results for sample B-3

Figure B103: Shear modulus results for sample B-3

B.22 Post Cyclic Shear Loading Results (CSR=0.15B-4)

Figure B104: Shear strain results for sample B-4

Figure B105: Shear stress results for sample B-4

Figure B106: Shear modulus results for sample B-4

B.23 Post Cyclic Shear Loading Results (CSR=0.12C-3)

Figure B107: Shear strain results for sample C-3

Cyclic Simple Shear Test

	Project Name: Liquefaction Silt	Location: Beaverton, OR	Project Number:
	Boring Number:	Tester: Rawan	Checker:
	Sample Number: CSR=0.12C-3	Test Date: 02/26/2020	Depth:
	Test Number:	Preparation: Remolded	Elevation:
	Description: Remolded silt, de-aired water used		
	Remarks:		

2020-07-29 03:03:00

3.0.18.220 / 3.0.18.220

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Figure B108: Shear stress results for sample C-3

Figure B109: Shear modulus results for sample C-3

B.24 Post Cyclic Shear Loading Results (CSR=0.12C-4)

Figure B110: Shear strain results for sample C-4

Figure B111: Shear stress results for sample C-4

Figure B112: Shear modulus results for sample C-4

B.25 Post Cyclic Shear Loading Results (CSR=0.12C-5)

Figure B113: Shear strain results for sample C-5

Figure B114: Shear stress results for sample C-5

Figure B115: Shear modulus results for sample C-5

B.26 Post Cyclic Shear Loading Results (CSR=0.12C-6)

Figure B116: Shear strain results for sample C-6

Figure B117: Shear stress results for sample C-6

Figure B118: Shear modulus for sample C-6

B.27 Monotonic Shear Loading Results

Figure B119: Shear strain for the monotonic loading test.

Figure B120: Shear stress for the monotonic loading test.

Figure B121: Shear modulus for the monotonic loading test.