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CYCLIC CONSTANT NORMAL STIFFNESS TESTS ON SAND

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CYCLIC CONSTANT NORMAL STIFFNESS TESTS ON SAND

BY

JAVIER FERNANDEZ SCARIONI

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE
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ABSTRACT

Pile-supported jacket structures were used for the Block Island Wind Farm project, the first offshore wind farm in the United States (DEEPWATERWIND 2012). Due to the significant length of the piles (60 m in some cases), the frictional resistance along the sides of these piles, termed shaft friction, is the main component of axial capacity. Shaft friction can be degraded due to cyclic loading from wind and waves, and understanding this behavior is critical for safe design. It is hypothesized that degradation of shaft friction can occur due to the contraction of a thin layer of soil, called the shear band, immediately in contact with the pile (DeJong, White & Randolph 2006).

In the laboratory, shaft friction is often modeled by performing interface shear tests with soil and the pile material. Interface shear tests, whether monotonic or cyclic, are commonly performed under constant normal load (i.e. direct shear). This boundary condition does not accurately model the shear behavior of piles as they do not account for changes in the normal stress during shearing. LeHane and White (2004) showed that, during loading, contraction of the shear band caused a reduction in the normal stress acting on a pile. In contrast, dilation of the shear band caused an increase in the normal stress. This behavior can be recreated in laboratory interface shear tests by imposing a constant normal stiffness condition (CNS) (e.g. with a spring) on the sample.

The primary objective of this thesis was to modify an existing cyclic simple shear device to be able to perform cyclic shear tests under constant normal

stiffness (CNS) conditions. A second objective was to perform a series of monotonic and cyclic tests in support of a research project funded by the United States Bureau of Safety and Environmental Enforcement (BSEE) to understand the behavior of piles under cyclic loading.

Both monotonic as well as cyclic CNS tests were performed on samples of Monterey Sand at various densities and values of normal stiffness. In the monotonic tests, dilation caused an increase and contraction caused a decrease in the shear strength of the samples compared to constant normal load (CNL) tests. Contraction of the soil along the interface occurred in all the cyclic tests, resulting in a decrease in shear resistance with each cycle of loading. The influence of initial normal stiffness and displacement amplitude was also investigated.

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CHAPTER I – INTRODUCTION

Interest in offshore wind as a renewable energy source has increased in the United States over the past decade.

In Europe, there have been many offshore wind farms constructed in the past 30 years, with most founded on large-diameter (~ 5 m) monopiles due to the relatively shallow water depths.

The Block Island Wind Farm is the first offshore wind farm in the U.S., and unlike in Europe, the relatively high-water depths (25 to 30 m) resulted in the choice of a jacket (Figure 1).

With jacket support structures, lateral loads from wind, waves, vessel impacts, and currents are transferred to the foundation as axial loads in piles. The length of the piles can exceed 50 m (Zhang, Fowai & Sun 2016), and because of this most of the load is transferred to the surrounding soil through shaft friction as opposed to tip resistance. Over the life of a structure, there will be millions of cycles of loading on the piles, and there is still uncertainty on how the cyclic loading affects the shaft resistance.

Figure 1 - Different types of offshore wind turbine foundations (European Wind Energy Association 2013)

This thesis is part of a larger study to better understand the cyclic axial behavior of piles for offshore wind jacket structures. This study is being done in collaboration with the U.S. Bureau of Safety and Environmental Enforcement (BSEE), the Norwegian Geotechnical Institute (NGI), and the University of Texas at Austin.

It is hypothesized that one of the major mechanisms responsible for a reduction in axial capacity of offshore piles under cyclic loading is contraction of the soil along interface between the pile and soil (termed the shear band). Contraction along the interface reduces the normal stress acting on the pile, which will cause the shear strength of the soil surrounding the pile to decrease because soil strength is frictional in nature (i.e. stress dependent).

The shaft resistance of piles can be estimated in the laboratory by performing interface shear tests. Commonly, these tests are performed under

constant normal load conditions even though this boundary condition does not allow for a change in normal stress due to contraction or dilation at the interface

By imposing a constant normal stiffness boundary condition on the sample instead of a constant normal load, it is possible to account for the changes in the normal stress due to contraction or dilation. These tests are called constant normal stiffness shear tests, but are never done in practice and rarely done in research labs.

The primary objective of this thesis is to modify an existing cyclic simple shear device at URI to be able to perform cyclic shear tests under constant normal stiffness (CNS) conditions. A second objective is to perform a series of monotonic and cyclic tests in support of a research project funded by the United States Bureau of Safety and Environmental Enforcement (BSEE) to understand the behavior of piles under cyclic axial loading.

CHAPTER II – LITERATURE REVIEW

This chapter presents a review of the literature relevant to this thesis. This includes a brief history of shear testing and typical results from interface tests, cyclic simple shear tests, and cyclic direct shear tests. Section 2.7 presents a review of the literature specific to constant normal stiffness shear testing.

2.1 Direct shear testing

Direct shear tests are one of the most common and oldest soil tests for determining the shear strength of soils. The objective of this test is to determine the maximum mobilized shear stress (i.e. shear strength) of a sample of soil as a function of normal stress.

Tests are performed in a container that is split horizontally in a load frame capable of applying vertical and horizontal stresses. The main components of the apparatus are shown in Figure 2 and Figure 3.

The normal (vertical) stress is distributed uniformly on the sample by means of a top cap and vertical displacement is measured by a dial gauge or a linear variable displacement transducer (LVDT). Horizontal (i.e. shear) loads and displacements are measured by additional load cells and LVDTs.

The loads are commonly divided by the cross-sectional area of the sample and reported as normal and shear stresses (Figure 4).

Figure 2 - Main elements of a direct shear apparatus.

Figure 3 – Schematic drawing of the elements of a direct shear apparatus.

The direct shear test consists of two phases. The first phase is the consolidation phase, in which the normal stress is applied by the load frame and

the sample consolidates. The normal stress usually ranges between 50 kPa and 500 kPa and is held constant throughout the test.

After the soil sample has consolidated the shearing phase occurs. Here a horizontal motion is applied, which can be either strain-controlled (preferred) or stress controlled.

The strain-controlled variant occurs at a constant rate of horizontal displacement that moves the lower half of the shear box. A load cell attached to the upper half records the shear stress on the horizontal plane. The shear stress will increase with increasing horizontal displacement until the sample fails.

Alternatively, the stress-controlled variant involves increasing the shear stress on the sample in steps and measuring the resulting horizontal displacement. This is rarely done in modern geotechnical testing.

Figure 4 – Forces in a direct shear test

2.2 Historical background

Tests that resembled direct shear tests were already being performed on different materials as early as the 18th century by the prominent French engineer and physicist Charles-Augustin de Coulomb (Lambe and Whitman 1969), and Coulomb is acknowledged as being the pioneer for these type of tests. Coulomb studied the phenomena of friction by analyzing the main factors that governed it: the normal load or stress, surface roughness, and size of the friction surface. Otto Mohr, a German civil engineer, developed the theory of the Mohr-Coulomb failure envelope in 1882, which received its name in honor of Coulomb and Mohr (Lambe and Whitman 1969).

One of the first to use direct shear tests to determine soil properties was Alexandre Collin (Skempton 1949). He studied the stability of clay slopes and published in 1846 his findings in "*Recherches Experimentales sur Quelques Principes de la Mecanique Terrestre*". A description of a shear box-type apparatus was included in the publication. The publication did not become relevant until 70 years after its publication, as the topic of slope stability regained importance within the scientific community.

The direct shear apparatus, as it is known today, was devised by Glennon Gilboy in 1936. Gilboy's apparatus was the first to feature a strain-controlled approach. The apparatus presented measurements of the shear load and led the soil sample contract or dilate freely while keeping a constant load. A picture and a drawing of the apparatus are shown in Figure 5 and Figure 6. Current devices are not significantly different in principle to Gilboy's apparatus.

Figure 5 - Drawing of shear apparatus. (Gilboy 1936)

Figure 6 - Photograph of Gilboy's shear apparatus. (Gilboy 1936)

2.3 Results of monotonic direct shear tests

Results of 4 performed direct shear tests are shown in Figure 7, Figure 8 and Figure 10. The tests were performed on samples of Monterey sand, which is a commonly tested sand for laboratory earthquake studies. Properties of the

Monterey sand were determined by Morales (2014) and are listed in Table 1. The shear displacement rate was 1 mm/min.

Table 1- Properties of Monterey sand (Morales 2014)

Grain diameter (D_{xx}) with (xx) corresponding to the percent finer by weight from a grain size analysis ¹	D ₁₀ (mm)	0.33
	D ₃₀ (mm)	0.45
	D ₅₀ (mm)	0.55
	D ₆₀ (mm)	0.58
Coefficient of Uniformity ¹	C _u (-)	1.76
Coefficient of Curvature ¹	C _c (-)	1.06
Specific Gravity ²	G _s (-)	2.66
Minimum Dry Unit Weight ³	γ_{min} (kN/m ³)	14.4
Maximum Dry Unit Weight ⁴	γ_{max} (kN/m ³)	16.4
Maximum Void Ratio ⁴	e_{max} (-)	0.808
Minimum Void Ratio ³	e_{min} (-)	0.589

¹ASTM D 422-63 (98)

²ASTM D 854-06

³ASTM D 4254-00

⁴ASTM D4253-00

The tests were performed at 4 different normal stresses: 100 kPa, 200 kPa, 300 kPa, and 400 kPa. The relative density of the soil samples ranged from 10% to 20%, and thus the samples can be considered to be in a loose condition.

Figure 7 shows the relationship between shear stress and horizontal displacement. The mobilized shear stresses increase with increasing normal stress. For the normal stresses of 100 kPa and 200 kPa there is no notable peak shear strength, whereas in the case of normal stresses of 300 kPa and 400 kPa, a peak shear strength can be identified.

Figure 7 - Horizontal displacement vs. shear stress for four samples of Monterey Sand in a direct shear test.

Figure 8 shows the relationship between vertical displacement and horizontal displacement during shear. When a soil is sheared it will either dilate or contract. Dilation means that the soil will increase in volume; contraction means that the volume decreases. If the soil state happens to be close to the critical state line there will be little to no change in volume (see Figure 9). In a direct shear test, the change in volume due to contraction or dilation is manifested as a change in height of the sample.

The critical state line is useful for understanding the dilative/contractive behavior of soils. For different samples with the same void ratio (or relative density), the confining stress determines if a sample dilates or contracts during shear, and the relative amount is governed by the distance from the critical state line. Samples will dilate most when confining stresses are low and the density is

high. With increasing confining stresses, the amount of dilation decreases until the sample becomes contractive (note that in direct shear testing the confining stress is the normal stress).

Dilative samples will commonly contract slightly at the beginning of shearing before dilation begins (e.g. tests at 100 kPa and 200 kPa in Figure 8)

Figure 8 - Vertical displacement vs. horizontal displacement for four samples of Monterey Sand in a direct shear test.

Figure 9 - Concept of a critical state line as a function of void ratio and confining stress for drained condition that acts as a boundary between contractive and dilative behavior.

The stresses at failure as represented by the Mohr-Coulomb failure envelope are shown in Figure 10. This envelope consists of the normal and shear stresses on the failure plane (i.e. horizontal in the direct shear test) at failure. The envelope should approximate a straight line although the true failure envelope is curved due to dilation (at low confining stresses) and contraction (at high confining stresses). For granular soils (sands, gravel) under drained loading conditions, the envelope will start at zero. Cohesive soils and soils under undrained loading conditions possess some shear strength at zero normal stress (termed “cohesion”), implying that the failure envelope will start at a value greater than zero in the shear stress axis. Cohesion is commonly represented with the letter “c”

Figure 10 - Mohr-coulomb failure envelope.

The angle of the slope of the failure envelope is called the angle of internal friction (or simply angle of friction) and is represented with the Greek letter “ ϕ ” or “ ϕ ”. The angle of friction and the cohesion are parameters used to describe the shear strength of soils. When the values of the angle of friction and cohesion are known the shear strength can be calculated at any specific normal stress with the following equation:

$$\tau_{ult} = \sigma_n * \tan \phi + c \quad (1)$$

where:

- σ_n : Effective normal stress
- τ_{ult} : Shear resistance
- ϕ : Internal angle of friction
- c : Cohesion

2.4 Monotonic interface shear tests

Shearing in direct shear tests is commonly performed along a soil-soil interface, meaning that the failure surface occurs entirely within the soil. This type of test results in the determination of the angle of friction of the soil. However, this angle of friction does not represent the friction between the surface of another material and a soil surface. Direct shear tests that measure the friction between soil and a different material (e.g. steel-sand) are called interface tests.

For these tests, the bottom half of a shear box is replaced by a base plate with the surface of a material on it (steel, aluminum, concrete, etc.). Hildebrandt (2018) constructed such a base plate with interchangeable surfaces and performed interface shear tests with numerous different surfaces as a part of a thesis at the University of Rhode Island on the behavior of helical anchors.

The interface friction angle is typically described as a percentage of the soil's friction angle. The interface friction angle for concrete, for example, ranges between 0.5ϕ and 0.66ϕ (depending on the roughness), and for steel the interface friction angle is often assumed to be 0.66ϕ (approximately 20° - 25°).

Figure 11 - Base plate built by Hildebrandt (2018) with steel surface for interface tests.

For the current study, it was possible to utilize a base plate built by Hildebrandt (2018) (Figure 11) to perform shear tests on a sand-steel surface. Four, monotonic interface shear tests were performed on samples of Monterey sand at normal stresses of 100 kPa, 200 kPa, 300 kPa, and 400 kPa.

Figure 12 shows the relationship between horizontal displacement and shear stress. When these results are compared to the results of the conventional direct shear tests (Figure 7) it is clear that the shear strength (i.e. maximum shear stress) of a steel-sand interface is considerably lower.

Another characteristic of interface tests is that the peak shear stress is reached at very low values of horizontal displacement. These results are consistent with the results presented by Hildebrandt (2018).

Figure 12 - Horizontal Displacement vs. Shear Stress for Four Interface Tests on Samples of Monterey Sand.

Figure 13 shows horizontal displacement vs. the vertical displacement for the four tests. All four samples exhibited contractive behavior; however, the amount of contraction is not consistent with the critical state line. According to the critical state line the amount of contraction becomes significantly higher when the normal load is increased, which is not the case here. One explanation for this might be that the steel interface is too smooth for the contractive/dilative behavior of soils to occur. Another cause might be that all the contraction is occurring in a thin layer adjacent to the steel surface and this does not get captured by the vertical LVDT at the top. Shear stresses performed by Hildebrandt (2018) on rougher surfaces presented a more typical dilative/contractive soil behavior.

Figure 14 presents the Mohr-Coulomb failure envelope for the four interface tests. The interface friction angle is approximately 2/3 soils' friction angle from

conventional direct shear tests. It is important to mention that the steel surface, in this case, was very smooth. If tests were to be performed on a rougher steel surface the friction between sand and steel would have been higher which would have led to higher shear stresses.

Figure 13 - Horizontal Displacement vs. Vertical Displacement for Four Interface Tests on Samples of Monterey Sand.

Figure 14. Mohr coulomb failure envelope for interface tests.

2.5 Cyclic shear tests

In many cases, such as for offshore structures or during earthquakes, cyclic loading dominates the design. Cyclic loading from wind, waves and current can degrade the shear strength of the soil and can occur under drained (no excess pore pressures develop) or undrained (no drainage during loading) conditions. Compared to offshore loading, earthquake loading is typically much higher in frequency, resulting in undrained loading that can lead to liquefaction in saturated sands, silts, and even some gravels. Cyclic loading is characterized in the laboratory by either cyclic triaxial tests or cyclic simple shear tests. The latter is comparable to direct shear tests and will be discussed further here.

In a cyclic simple shear test the soil sample is confined by a stack of rings or a wire-reinforced membrane that allows for K_0 consolidation (zero lateral strain) and simple shear (Figure 15). This allows for the development of shear strain (γ), which is the horizontal displacement divided by the height of the sample, from the application of shear stress.

Figure 15 - Confining rings in a cyclic simple shear test.

Cyclic simple shear tests can be strain- or stress-controlled, where the applied shear stress or shear strain follows a sinusoidal course. Results of typical stress-controlled cyclic simple shear test on Monterey sand are shown in Figures 16, 17, and 18. The height of the sample during shear is kept constant, which equates to an undrained loading condition (even if the samples are dry). In dry samples, the change in the normal stress during shear equals the excess pore water pressure build-up in an equivalent saturated soil. If the sample contracts (as in cyclic testing the vast majority of cases) positive pore water pressure is generated; if the sample dilates negative pore water pressure is generated.

In Figure 17, for example, near the 30th cycle, the effective stress decreased to the point that the shear strain started increasing rapidly.

$\sigma_n = 100\text{kPa}$

Figure 16 – Applied shear stress vs. number of cycles for stress-controlled cyclic simple shear test. (Bogden 2019)

Figure 17 - Excess pore water pressure for stress-controlled cyclic simple shear test. (Bogden 2019)

*Figure 18 - Shear strain vs. number of cycles for stress-controlled cyclic simple shear test.
(Bogden 2019)*

Results of a typical strain-controlled cyclic simple shear tests are shown in Figure 19. In this case, the applied cyclic shear strain stays constant. With increasing cycles, the shear stress decreases until a residual shear stress state is reached. As with stress-controlled tests, excess pore water pressures increase with each cycle.

Figure 19 - Typical results of a strain-controlled cyclic simple shear test. (Hazirbaba & Rathje 2019)

The advantage of simple shear testing over direct shear testing is that in simple shear the failure plane is not forced. In the case of direct shear, the failure plane is forced to be the interface between the two sliding halves.

A disadvantage of simple shear testing, in general, is that the stack of rings precludes the possibility of performing interface tests.

2.6 Constant Normal Stiffness Condition

As mentioned in Chapter 1, direct shear tests are conventionally performed under constant normal load (CNL) conditions. This means that the applied normal stress remains constant throughout shear and the sample is allowed to change volume (contract or dilate). This condition is representative for cases such as slope

stability or retaining wall problems where it can be assumed that the normal stress remains approximately constant during shear.

Constant normal load tests, however, are not applicable to shaft resistance along piles. When shearing occurs along a pile, volume change in the soil immediately against the pile will change the normal stresses in the surrounding soil.

Tests performed by Boulon and Foray (1986) indicated that the shear stress along piles is concentrated in a thin zone about ten times the grain diameter. This zone, referred to as the interface, is where the dilation and contraction occurs. This interface is constrained by the soil surrounding it; it can be thought of as an infinite number of uncoupled springs along the interface. These springs have an initial deflection that represents the earth pressure (normal stress) acting on the pile. If dilation occurs the spring is loaded, increasing thus the normal stress acting on the pile. If contraction occurs the spring will be unloaded, decreasing the normal stress. This is termed the constant normal stiffness (CNS) boundary condition.

Figure 20 - Constant Normal Stiffness condition for pile-soil interface. (Lehane & White 2004)

The CNS condition for pile-soil interfaces mentioned above is shown in Figure 20. “ Δt ” is the change in interface thickness due to dilation or contraction. The initial normal load acting on the pile “ σ_h ” will increase or decrease due to dilation or contraction by $\Delta\sigma_h$:

$$\Delta\sigma_h = 4G\Delta t/D = k_n\Delta t \quad (2)$$

where:

- G : Shear modulus of the soil around the pile
- D : Pile shaft diameter
- k_n : Stiffness of soil represented as spring stiffness

The above-mentioned behavior can be corroborated by measuring the change in lateral stress (or normal stress) acting on a pile during shearing. LeHane and White (2004) did so by performing pile tests in a geotechnical drum centrifuge at the University of Western Australia (UWA).

Static compression and tension tests were conducted on model piles installed in 3 different methods: Monotonic installation, jacked installation and pseudo-dynamic installation. The comparison of different pile installation methods is not the intention for this thesis, therefore, won't be compared with the 3 methods mentioned above regarding the increase in the lateral pressure. The results presented in Figure 21 show clearly that an increase in lateral pressure due to dilation occurs when the pile is sheared. In the tests performed by Lehane and White (2004) only dilation occurred; however, it can be assumed by logic that contraction would lead to a decrease in the lateral pressure.

Figure 21 - Variation in the lateral stress during static tension tests. (Lehane & White 2004)

Figure 21 shows how the normal stress (referred to as lateral stress in the graphs) increases with increasing pile displacement until reaching a maximum value. Different authors recognized that this had a significant influence on the load-

bearing behavior of piles and worked on devising a modified direct shear apparatus that could capture the changes in the normal stress due to dilation and contraction.

2.6.1 Constant Normal Stiffness Direct Shear Testing

In this chapter are presented three different CNS direct shear apparatuses. They account in different ways for the same behavior: during shearing, dilation and contraction of the interface leads to a variation in the normal stress.

Additional to schematic drawings are presented some results of tests performed with the respective apparatuses. This should give a good idea into how the CNS condition is achieved and how it reflects itself in the actual testing.

Lam and Johnston (1982)

The first authors known to devise a CNS direct shear test apparatus were Lam and Johnston in 1982. Their intention, however, was to account for a different phenomenon. When a socketed pile foundation experiences axial displacement, dilation of the socketed walls occurs due to the roughness of the interface between pile and rock. This dilation occurs against the stiffness of the rock. Figure 22 illustrates this behavior. If the pile is moved axially up or down, an increase in the lateral stress will occur due to the roughness of the rock-pile interface. The increase in the normal stress will be proportional to the amount of dilation and the stiffness of the rock.

Figure 22 - Diagrammatic displacement characteristics of piles socketed into rock (Lam & Johnston 1982)

With this in mind, Lam and Johnston (1982) devised a direct shear apparatus that imposed a constant normal stiffness on the rock-concrete sample. The principle of the shear apparatus is shown in Figure 23. The bottom rock box is only allowed to move horizontally whereas the upper concrete box, which models the surface of the pile, is only allowed to move vertically. The roughness of the surface of the concrete and rock is represented in the drawing with 3 peaks. Due to this roughness, any horizontal displacement (shear displacement) of the rock box results in a vertical displacement in the concrete box. The vertical displacement will trigger the stiffness of the spring (k_{CNS}) which represents the stiffness of the rock. The spring is in the form of a steel bar that bends; thus the bending stiffness of this bar is the stiffness of the spring.

A photograph of the CNS direct shear apparatus is shown in Figure 24.

Figure 23 - Schematic drawing of the CNS shear apparatus from Lam and Johnston (Lam & Johnston 1982)

Figure 24 - Photograph of the CNS direct shear apparatus from Lam and Johnston (Lam & Johnston 1982)

Some results are presented in Figure 25. Tests 4 and 5 were performed with a stiffness of 300 kPa/mm, and tests 6 and 7 with a stiffness of 950 kPa/mm. The stiffness is reported as the change in normal stress (i.e. force divided by the sample area) acting on the sample divided by the vertical displacement. Dilation of the sample triggered the spring which led to an increase of the normal stress.

The higher normal stress increased the shear strength of the probe. It can also be seen that tests performed with higher constant normal stiffness have denoted higher increases in the normal stress.

Figure 25- Variation of shear stress and normal stress with the shear displacement. Tests 4 & 5: $k_{CNS} = 300 \text{ kPa/mm}$; Tests 6 & 7 $k_{CNS} = 950 \text{ kPa/mm}$. (Lam & Johnston 1982)

Tabucanon (1995)

Tabucanon, Airey, and Poulos (1995) developed a CNS direct shear apparatus for granular soils. The set-up of the shear apparatus was similar to that of Lam and Johnston (1982), with the constant normal stiffness condition achieved with the bending stiffness of a plate. A schematic diagram of the CNS shear apparatus is shown in Figure 26.

Both static and cyclic direct shear tests were performed. Different bars with stiffnesses of 220, 630 and 1850 kPa/mm were used. The tests were performed on Sydney Silica Sand and Brass Strait Calcareous Sand.

Results of the static shear tests are shown in Figure 27. Medium dense samples (relative density = 60%), which have the tendency to dilate, showed a notable increase in the normal stress. The loose samples (relative density $\approx 0\%$) contracted, which led to a decrease in the normal stress. There is also a clear trend when the stiffness of the bar is increased: higher stiffness's lead to higher changes in the normal stress.

CNS tests performed on dilative samples exhibited higher shear strengths than CNL tests with the same density, CNS tests that contracted exhibited lower shear strengths than their CNL counterparts.

Figure 26 - Schematic diagram of CNS direct shear apparatus from Tabucanon et al. (1995)

Figure 27 - Static CNS shear tests performed on Bass Strait Calcareous sand. (Tabucanon et al. 1995)

Results of cyclic direct shear tests on loose (relative density $\approx 0\%$) and dense (relative density = 60%) sand samples are shown in Figure 28 and Figure 29. The tests were strain-controlled with a displacement amplitude of 1 mm.

In the loose samples (Figure 28), contraction during the first cycles due to the large displacement lowered the normal stress to 0 (i.e. unloading of the spring; right figure). This significantly decreased the mobilized shear stresses (left figure).

Figure 28 - Reductions of normal and shear stresses during loose CNS cyclic tests. (Tabucanon et al 1995)

The dense samples (Figure 29) experienced less contraction and thus a lower decrease in the normal and shear stresses.

Figure 29 - Reductions of normal and shear stresses during dense CNS cyclic tests. (Tabucanon, Airey & Poulos 1995)

Porcino et al. (2003)

Porcino, Fioravante and Pedroni (2003) devised a CNS testing apparatus that worked by back regulating the normal stress during shearing with the following equation:

$$\Delta\sigma_h = k_n \Delta t \text{ [kPa]} \quad (3)$$

Where:

- $\Delta\sigma_h$: Increment of normal stress
- Δt : Vertical displacement
- k_n : Constant normal stiffness

When dilation or contraction occurred, the normal stress is automatically increased or decreased, thus providing a constant normal stiffness condition. A schematic of the direct shear apparatus is provided in Figure 30.

Figure 30 - Schematic view of CNS direct shear apparatus. (Porcino et al. 2003)

The CNS tests were performed on an aluminum-sand interface (interface CNS test). The 3 sands used were natural silica sand with properties listed in

Table 2. Different aluminum surfaces were also used; ranging from smooth to rough. Smooth surfaces will make a sample more contractive whereas rougher surfaces will make a sample more dilative.

Table 2 - Properties of sands tested by. (Porcino et al. 2003)

Properties	Ticino Sand (TS10)	Toyoura Sand	FF Sand
$\gamma_{\max} [kN/m^3]$	16.67	16.10	14.78
$\gamma_{\min} [kN/m^3]$	13.94	13.58	11.59
$e_{\min}$	0.58	0.62	0.73
$e_{\max}$	0.89	0.92	1.21
G_s	2.68	2.64	2.61
$D_{10}[mm]$	0.35	0.19	0.05
$D_{50}[mm]$	0.56	0.24	0.09
$D_{60}[mm]$	0.62	0.26	0.10
$C = D_{60}/D_{10}$	1.78	1.42	2.09

Results of a test performed with a rough aluminum surface on a dense sample of Ticino sand are shown in Figure 31. Note that in the graphs the symbol K is the constant normal stiffness k_{CNS} . A value of $K = 0$ kPa/mm refers to a CNL test. The initial normal stress applied was $\sigma_h = 150$ kPa.

Figure 31 - CNS tests for dense sand-rough aluminum interface. (Porcino et al. 2003)

All 3 tests dilated (Figure 31, right), probably due to the rough aluminum surface. The dilation led to an increase in the normal stress in the CNS tests, where the one with the highest stiffness experienced the highest increase (Figure 31, center). Higher normal stresses led to higher shear strengths; this is reflected in Figure 31 (left).

CHAPTER III – CNS TESTING METHODOLOGY

This chapter presents the development of the Constant Normal Stiffness direct shear apparatus used in this study, including how to impose an accurate value of normal stiffness on samples. Detailed step-by-step instructions on performing monotonic and cyclic CNS tests can be found in Appendices A, B, and C.

3.1 Achieving specific values of constant normal stiffness

The CNS apparatus consists of a modified ShearTrac-II direct shear machine manufactured by the GEOCOMP Corp. (shown in Figure 2). For the initial testing, the original reaction bar provided by GEOCOMP, with a thickness of 38.5 mm, was replaced by a significantly smaller aluminum bar with a height of 6.9 mm (Figure 32).

Figure 32 - Above: Aluminum bar used for CNS testing. Below: Original aluminum bar used for CNL testing.

The theoretical bending stiffness of the plate was calculated using equations 4 and 5. These calculations were done in order to have an initial idea of the value of the stiffness that was being employed.

The actual stiffness of the system should be between the calculated stiffness values from equations 4 and 5, where the first represents the stiffness of an ideal simply supported beam and the second of an ideal completely fixed beam.

$$k = 48EJ/l^3 \quad (4)$$

$$k = 192EJ/l^3 \quad (5)$$

where

- E = Young's modulus of elasticity (69 GPa for aluminum)
- J = Moment of Inertia ($h^3b/12$)
- l = Span length (200 mm)
- k = bending stiffness in kPa/mm

Using equations 4 and 5, the 6.9 mm high aluminum bar resulted in a stiffness ranging from 127 kPa/mm to 510 kPa/mm.

The actual stiffness of the system was determined through bending tests in the direct shear apparatus. The set-up of the tests is shown in Figure 33. The shear box was replaced by a piece of steel. Loads were applied onto this steel piece. Since it is much stiffer than the system it was assumed the steel piece did not deform under the applied load, meaning that all of the measured deformations occurred in the bar.

The vertical deformation in the center of the aluminum bar was measured along with the load in the vertical load cell. Note that the measured vertical deformation is the combined deformation of multiple components of the system as a whole. The deformation includes the elastic extension of components such as the threaded rods and the steel block. The results of 10 3-point bending stiffness tests are shown in Figure 34, with an average stiffness of around 225 kPa/mm. This was the value of constant normal stiffness (k_{CNS}) assumed for tests performed with the 6.9 mm high aluminum plate.

Figure 33 - Set-up for determining the value of the constant normal stiffness

Figure 34 - Normal stress vs. vertical deformation for stiffness tests of 6.9 mm high aluminum bar.

A more detailed explanation on determining the value constant normal stiffness can be found in Appendix A.

3.2 Monotonic CNS Testing

The ShearTrac-II is designed for running a constant normal load (CNL) tests (Figure 35). The apparatus maintains a constant load on the sample by moving the loading frame up or down depending on what is needed. If during shearing contraction occurs the load frame will move down in order for the load to remain constant; the opposite will happen if the sample dilates. The loading frame in ShearTrac-II was designed with a bar with a very high bending stiffness so that no bending occurs.

Figure 35 - CNL test drawing.

CNS tests were performed by modifying the ShearTrac-II. As already mentioned above, the original bar was replaced by a significantly smaller one. The consolidation phase was conducted in the same way as a CNL test; the loading frame lowers until the desired normal stress is reached. Since the bar used for CNS testing is considerably less stiff it will bend noticeably in a CNS test (see Figure 36).

Figure 36 - CNS testing Drawing. Left: before consolidation phase. Right: after consolidation phase.

During the shear phase in CNS tests, dilation or contraction needs to occur against a constant normal stiffness. For this to happen, the vertical load frame must not move during shear. All the vertical displacement must occur in the cross bar

(i.e. spring) to maintain constant stiffness. This can be achieved in the ShearTrac-II by setting the P-Gain to “0” in the software. This effectively locks the cross arm during shear. Note that this must be done after the consolidation phase is concluded and before the shearing phase is started.

Step by step instructions for performing Monotonic and cyclic CNS tests can be found in Appendix B.

3.3 Cyclic CNS Testing

Cyclic CNS tests were performed in a modified Cyclic ShearTrac-II system manufactured by the GeoComp Corp. In its original form, this apparatus is made for running cyclic simple shear tests, which by definition are constant volume tests using either stacked rings or wire-reinforced membranes. As a result, the existing Geocomp system was modified in two ways to perform cyclic direct shear tests under CNS conditions. First, a steel angle was attached to the base of the restraining arm to hold the upper half of the shear box in place during the cyclic shearing.

Figure 37 – Steel angle attached to the base of the restraining arm.

The second modification was to replace the cross arm with bars of various thicknesses (i.e. various stiffnesses). After the consolidation phase, the loading frame is locked by setting the normal control to “no control” in the cyclic table of the software.

Detailed instructions for setting up the apparatus for running CNS direct shear tests can be found in Appendix C.

3.4 Sample Preparation

For comparing different tests results, it is important for samples to be prepared in a uniform fashion. In this subchapter will be explained the methods in which the different samples were prepared.

Monotonic Tests

The loose monotonic tests were prepared by filling a scoop with the sand sample and pouring it as fast as possible in the shear box. The exceeding sand was then scraped off so that the sample had an even surface. The excess sand was cleaned off the shear box and subsequently was weighed the sample. For soil on soil tests was the weight of the loose samples between 208 and 210 gr. For in interface tests was weight between 122 and 124 gr.

The dense monotonic tests were prepared by pouring sand in the shear box through a funnel until it was half full and stomping it with a pestle. When using the funnel is important to maintain a constant height. The same procedure was used for the other half of the shear box. The excess sand was cleaned off the shear box and subsequently was weighed the sample. The weight of the samples was between 220 and 223 gr for sand-sand tests and between 122 and 124 gr for the interface tests.

Cyclic Tests

The cyclic tests were prepared using an exact predetermined amount of sand. In the case of the loose tests, this amount was 115.3 gr ($D_r = 10\%$) and for the dense tests it was 122.5 gr ($D_r = 60\%$).

The loose samples were prepared by filling a scoop with the 115.3 gr of sand and then pouring it in the shear box. The surface of the sand sample was evened out the excess sand was then put back on the top of the shear box. The

surface of the sand was then evened out by pushing the metal cap on the sample until the surface was even.

The dense samples by pouring in the shear box 50 gr of sand and shaking the it per hand 5 times. Then was the metal cap pushed down on the sample and taken out. The second step was to pour 50 gr of sand and repeating the same procedure for the first 50 gr. The remaining amount of sand (22.5 gr) was poured in the shear box and the surface was evened out. The excess sand was then put back on the top of the shear box and the surface evened out by pushing down the metal cap onto the sample.

Note that the shearing occurs within the interface. This means that the density in the interface will be decisive for the overall behavior occurring in the shear tests. The void ratio or relative density that is presented is calculated for the whole volume of the shear box and could be not representative of the relative density in the interface.

CHAPTER IV – MONOTONIC CNS TESTS RESULTS

This chapter presents the results of the monotonic CNS tests. Results of CNL tests will also be provided so that the effect of the CNS condition can be compared to traditional direct shear test results. All Tests were performed with Monterey Sand (see Table 1 for index properties). Tests were performed with a constant shearing rate of 0.167 mm/min with a maximum shear displacement of 5 mm. This level of displacement is sufficient to fully mobilize the shear strength of the soil.

The normal stresses and constant normal stiffness used were chosen based on results presented by other authors. It was concluded that a normal stress range from 100 to 400 kPa would give a good range of different results and a constant normal stiffness of 225 kPa/mm was a good value for testing.

4.1 Variability in Monotonic CNS testing

First, the results of 5 CNS tests on identically prepared samples are compared to each other to assess variability in the results. One CNL test is also shown so that the general trend of CNS tests can be identified in comparison to the CNL test (Figure 42). All 6 tests were consolidated to a vertical effective stress of 100 kPa and the stiffness, k_{CNS} , of the CNS tests was 225 kPa/mm.

The relative density of the tests was between 20 and 30%. All the CNS tests showed considerably higher shear strength than the CNL test. This is in accordance with the results issued by other authors shown herein.

Figure 38 shows 5 CNS tests performed with almost equal conditions (same starting normal stress and similar relative density), yet there is some variability. This variability is likely due to several factors: differences in the relative density, differences in the sample preparation, inhomogeneity of the sand, inaccuracies in the shear apparatus, etc.

Figure 38 - Shear stress vs. horizontal displacement of 5 loose CNS tests and a loose CNL test.

These dissimilarities are all inherent to direct shear testing, CNL as well as CNS. In the case of CNS testing any dissimilarity will cause the normal stress to vary differently in each test; this does not happen in CNL testing. Because of this will be the variability in CNS testing higher than in CNL testing.

Figure 39 and shows the results of 5 CNL tests. These tests were performed under a normal stress of 50 kPa. The variability in the results of these tests is much smaller than in the CNS tests shown in Figure 38. This is the case even though there was a considerable spread in the vertical deformation in vertical displacement (Figure 40).

Figure 39 - Shear stress vs. horizontal displacement of 5 CNL tests.

Figure 40 – Vertical displacement vs. horizontal displacement of 5 CNL tests.

For the CNS tests, the variation of the normal stress during shearing is shown in Figure 41. There is a considerable spread in the change in the normal stress. It can also be noted that at the beginning of shearing the normal stress decreases; this is due to the initial contraction of the sand sample during shearing.

Figure 41 - Normal stress vs. horizontal displacement of 5 loose CNS tests and a loose CNL test (Monterey Sand).

In Figure 42 is presented the vertical displacement with the horizontal displacement. All tests dilated.

Figure 42 - Vertical Displacement vs. Horizontal Displacement of 5 loose CNS tests and a loose CNL test (Monterey Sand).

4.2 Monotonic CNS testing on a sand-sand interface with Monterey sand

Figure 43 to Figure 54 present results of CNS and CNL with different starting normal stress for loose and dense samples. The tests were performed with a k_{CNS} of 225 kPa/mm. The shear rate was 0.166 mm/min with a maximum shear displacement value of 5 mm. In Table 3 can be seen the test matrix of the tests from this chapter

All dense samples presented dilative behavior. At a normal stress of 100 kPa, all the loose samples dilated. At 200 kPa the loose CNL test dilated while the loose CNS tests contracted. At 300 and 400 kPa all loose samples contracted. As will be explained below, whether a sample dilates, or contracts plays is an important role in the behavior of CNS testing.

The CNS tests on the dense samples all had higher values of shear strength, which is due to an increase in the normal stress due to dilation. As expected, dense CNL tests yielded higher shear strength than tests on loose samples.

The loose CNS tests exhibited the most variability. At a normal stress of 100 kPa, the samples were dilative and the shear strength was higher than the loose CNL tests.

At a normal stress of 200 kPa, the loose CNS samples become contractive. The contraction in the sample leads to a decrease in the normal stress thus the lowest shear stresses.

At normal stresses of 300 and 400 kPa, the loose CNS samples contracted and subsequently stayed constant; probably because the sample was close to the critical state line. This caused the loose CNS and CNL samples to have similar shear strengths (normal stress remained close to constant until failure in the CNS test).

A general pattern can be seen when looking at Figure 43 to Figure 54: In CNS testing, dilative samples will have higher shear strengths, whereas contractive samples will have lower shear strengths when compared to CNL tests under same conditions.

Dilative CNS tests dilated consistently less than their CNL counterparts. This is due to the increase in the normal stress during shearing (higher normal stress leads to lower dilations). The opposite happens to contractive CNS tests: lower normal stresses lead to higher contractions.

Dilative CNS tests reached their peak shear stress at larger shear displacement than the CNL tests. The shear displacement at which the contractive CNS tests reached the peak shear stress did not differ much from the CNL tests.

Table 3. Test matrix for subchapter 4.2

Test Number	Test Type	Normal Stress (kPa)	Initial Void Ratio (-)	Initial Relative Density (%)	Normal Stiffness (kPa/mm)
1	CNL	100	0.77	16.7	-
2	CNS	100	0.78	12.6	225
3	CNL	100	0.69	58.3	-
4	CNS	100	0.67	65.5	225
5	CNL	200	0.77	18.2	-
6	CNS	200	0.78	14.3	225
7	CNL	200	0.68	59.8	-
8	CNS	200	0.68	60.3	225
9	CNL	300	0.76	23.2	-
10	CNS	300	0.78	14.9	225
11	CNL	300	0.68	63	-
12	CNS	300	0.67	69.9	225
13	CNL	400	0.78	13.3	-
14	CNS	400	0.77	19.9	225
15	CNL	400	0.68	64.1	-
16	CNS	400	0.66	70.4	225

Figure 43 – Shear stress vs. horizontal displacement of dense and loose CNS and CNL tests on Monterey Sand for $\sigma_N = 100$ kPa and $k_{CNS} = 225$ kPa/mm

Figure 44 - Normal stress vs. horizontal displacement of dense and loose CNS and CNL tests on Monterey Sand for $\sigma_N = 100$ kPa and $k_{CNS} = 225$ kPa/mm

Figure 45 – Vertical displacement vs. horizontal displacement of dense and loose CNS and CNL tests on Monterey Sand for $\sigma_N = 100 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 46 – Shear stress vs. horizontal displacement of dense and loose CNS and CNL tests on Monterey Sand for $\sigma_N = 200$ kPa and $k_{CNS} = 225$ kPa/mm

Figure 47 – Normal stress vs. horizontal displacement of dense and loose CNS and CNL tests on Monterey Sand for $\sigma_N = 200$ kPa and $k_{CNS} = 225$ kPa/mm

Figure 48 - Vertical displacement vs. horizontal displacement of dense and loose CNS and CNL tests on Monterey Sand for $\sigma_N = 200 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 49 – Shear stress vs. horizontal displacement of dense and loose CNS and CNL tests for on Monterey Sand $\sigma_N = 300 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 50 – Normal stress vs. horizontal displacement of dense and loose CNS and CNL tests for on Monterey Sand $\sigma_N = 300 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 51 - Vertical displacement vs. horizontal displacement of dense and loose CNS and CNL tests on Monterey Sand for $\sigma_N = 300 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 52 – Shear stress vs. horizontal displacement of dense and loose CNS and CNL tests on Monterey Sand for $\sigma_n = 400 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 53 – Normal stress vs. horizontal displacement of dense and loose CNS and CNL tests on Monterey Sand for $\sigma_n = 400 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 54 – Vertical displacement vs. horizontal displacement of dense and loose CNS and CNL tests on Monterey Sand for $\sigma_n = 400 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

4.3 Mohr-Coulomb failure envelopes for Sand-Sand Tests

The results of the 16 monotonic CNL and CNS shear tests are summarized in this subchapter. Figure 55 shows the Mohr-Coulomb failure envelopes for those tests and the results are summarized in Table 4. The medium dense CNS tests present the highest values of shear stress whereas the loose CNS tests the lowest. The reason for this is that the medium dense samples exhibited dilation which led to an increase of the normal stress, and so increasing the shear strength of the sample. The values of the correlated angles of friction are listed in Table 4.

Note that the peaks of the CNS tests are not at the starting normal stresses of 100, 200, 300 and 400 kPa. This is because the normal stress changed due to the CNS condition and thus the peaks occurred at a different normal stress.

Figure 55 – Mohr-Coulomb failure envelope for sand-sand interface of Monterey Sand.

Table 4 - Angles of internal friction for monotonic CNS and CNL tests on Monterey Sand.

Test Type	Angle of Friction for Sand-Sand Interface
Dense CNS	39.22°
Dense CNL	38.4°
Loose CNS	32.1°
Loose CNL	32.9°

4.4 Monotonic CNS testing on steel (smooth)-sand interface with Monterey sand

Figure 56 to Figure 67 show the results of CNS and CNL interface tests performed at normal stresses of 100, 200, 300 and 400 kPa. The constant normal stiffness had a value of 225 kPa/mm. The shear rate was 0.166 mm/s with a maximum shear displacement value of 5 mm. The test results are also summarized in Table 5.

All CNS tests contracted; the reason for this is the smooth steel surface and relatively high normal stresses. This led to a decrease in the normal stress in all the CNS tests. Dense CNS tests had a lower contraction and thus lower decreases in the normal stress.

Dense CNS tests consistently presented higher peak shear strengths than the dense CNL tests. The value of the residual (i.e. large displacement) shear strengths of the dense CNS and CNL tests were close to each other.

The loose samples contracted significantly. This led to a large decrease in the normal stress in the CNS tests, and these tests had the lowest shear strengths.

The values of peak shear strength were reached at low horizontal displacements of approximately 0.5 mm. Increasing the confining pressure did not cause a significant change in the horizontal displacement at which the peak is reached.

Table 5. Test Matrix for Subchapter 4.4

Test Number	Test Type	Normal Stress (kPa)	Initial Void Ratio (-)	Initial Relative Density (%)	Normal Stiffness (kPa/mm)
1	CNL	100 kPa	~0.787	~10	-
2	CNS	100 kPa	~0.787	~10	225
3	CNL	100 kPa	0.682-0.661	60-70	-
4	CNS	100 kPa	0.67	64.3	225
5	CNL	200 kPa	~0.787	~10	-
6	CNS	200 kPa	~0.787	~10	225
7	CNL	200 kPa	0.682-0.661	60-70	-
8	CNS	200 kPa	0.67	64.72	225
9	CNL	300 kPa	~0.787	~10	-
10	CNS	300 kPa	~0.787	~10	225
11	CNL	300 kPa	0.682-0.661	60-70	-
12	CNS	300 kPa	0.67	64.48	225
13	CNL	400 kPa	~0.787	~10	-
14	CNS	400 kPa	~0.787	~10	225
15	CNL	400 kPa	0.682-0.661	60-70	-
16	CNS	400 kPa	0.67	64.91	225

Figure 56 – Shear stress vs. horizontal displacement of dense and loose CNS and CNL interface tests on Monterey Sand for $\sigma_n = 100 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 57 - Normal stress vs. horizontal displacement of dense and loose CNS and CNL interface tests on Monterey Sand for $\sigma_n = 100 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 58 – Vertical displacement vs. horizontal displacement of dense and loose CNS and CNL interface on Monterey Sand tests for $\sigma_n = 100 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 59- Shear stress vs. horizontal displacement of dense and loose CNS and CNL interface tests on Monterey Sand for $\sigma_N = 200$ kPa and $k_{CNS} = 225$ kPa/mm

Figure 60 - Normal stress vs. horizontal displacement of dense and loose CNS and CNL interface tests on Monterey Sand for $\sigma_N = 200$ kPa and $k_{CNS} = 225$ kPa/mm

Figure 61 – Vertical displacement vs. horizontal displacement of dense and loose CNS and CNL interface tests on Monterey Sand for $\sigma_N = 200 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 62 - Shear stress vs. horizontal displacement of dense and loose CNS and CNL interface tests on Monterey Sand for $\sigma_n = 300 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 63 - Normal stress vs. horizontal displacement of dense and loose CNS and CNL interface tests on Monterey Sand for $\sigma_n = 300 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 64 – Vertical displacement vs. horizontal displacement of dense and loose CNS and CNL interface tests on Monterey Sand for $\sigma_N = 300 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 65- Shear stress vs. horizontal displacement of dense and loose CNS and CNL interface tests on Monterey Sand for $\sigma_n = 400 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 66 - Normal stress vs. horizontal displacement of dense and loose CNS and CNL interface tests on Monterey Sand for $\sigma_n = 400 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

Figure 67 – Vertical displacement vs. horizontal displacement of dense and loose CNS and CNL interface tests on Monterey Sand for $\sigma_N = 400 \text{ kPa}$ and $k_{CNS} = 225 \text{ kPa/mm}$

4.5 Mohr-Coulomb failure envelopes for Interface Test

Table 6 and Figure 68 presents the Mohr-Coulomb failure envelope for the Mohr-Coulomb failure envelope and resulting friction angle for the interface tests. The medium dense CNS tests presented the highest shear stresses whereas the loose CNS tests the lowest. This is consistent with the Sand-Sand tests.

Note that the peaks of the CNS tests are not at the starting normal stresses of 100, 200, 300 and 400 kPa. This is because the normal stress changed due to the CNS condition and thus the peaks occurred at a different normal stress.

Table 6 - Angles of internal friction for monotonic CNS and CNL interface tests on Monterey Sand.

Test Type	Angle of Friction for Steel-Sand Interface
Dense CNS	23.6°
Dense CNL	22.2°
Loose CNS	16.7°
Loose CNL	17.9°

Figure 68 - Mohr-Coulomb failure envelopes of peak shear strength for a steel-sand interface for Monterey sand.

CHAPTER V – CYCLIC CNS TESTS RESULTS

In this chapter the results of the cyclic CNS direct shear tests are presented. All the tests were performed with a steel-sand interface. Multiple cross arms were used to achieve different constant normal stiffnesses. The influence of the density of the sample was also studied by preparing loose and dense samples. The cyclic shear apparatus was setup as described in Appendix C.

5.1 Tests on Loose and Medium Dense Samples

Tests were performed with Monterey Sand (see Table 1) on a smooth steel interface with a medium dense ($D_r = 60\%$) and a loose relative density ($D_r = 10\%$). The cyclic period was 10 s. All tests presented in this subchapter were run with a constant normal stiffness of 225 kPa/mm. Different horizontal displacement amplitudes were analyzed: 0.25 mm, 0.5 mm, 0.75 mm, 1 mm and 2 mm. This was done to have an idea about how many cycles to expect until the normal stress reached zero depending on the horizontal displacement amplitude.

The starting normal stress was 100 kPa. All the tests exhibited contractive behavior and thus the normal stress decreased throughout shear. The tests were ended when the normal stress reached a value of 0 kPa.

Table 7 shows the test matrix for these tests, including levels of cyclic displacement amplitude, initial normal stress, initial void ratio, initial relative density and the value of constant normal stiffness. Figure 69 to Figure 78 show the test results in terms of shear stress vs. horizontal displacement, shear stress vs.

normal stress (i.e. stress path), vertical displacement vs. shear displacement, and degradation of shear stress and normal stress vs. cycles of loading.

Table 7. Test Matrix for Subchapter 5.1

Test Number	Cyclic Displacement Amplitude (mm)	Initial Normal Stress (kPa)	Initial Void Ratio (-)	Initial Relative Density (%)	Normal Stiffness (kPa/mm)
1	0.25	100	0.68	~60	225
2	0.5	100	0.68	~60	225
3	0.75	100	0.68	~60	225
4	1.0	100	0.68	~60	225
5	2.0	100	0.68	~60	225
6	0.25	100	0.68	~10	225
7	0.5	100	0.68	~10	225
8	0.75	100	0.68	~10	225
9	1.0	100	0.68	~10	225
10	2.0	100	0.68	~10	225

Figure 69 – Cyclic medium dense CNS test results on Monterey Sand with displacement amplitude = 0.25 mm

Figure 70 – Cyclic medium dense CNS test results on Monterey Sand with displacement amplitude = 0.5 mm

Figure 71 – Cyclic medium dense CNS test results on Monterey Sand with displacement amplitude = 0.75 mm

Figure 72 – Cyclic medium dense CNS test results on Monterey Sand with displacement amplitude = 1 mm

Figure 73 – Cyclic medium dense CNS test results on Monterey Sand with displacement amplitude = 2 mm

Figure 74 – Cyclic loose CNS test results on Monterey Sand with displacement amplitude = 0.25 mm

Figure 75 – Cyclic loose CNS test results on Monterey Sand with displacement amplitude = 0.5 mm

Figure 76 – Cyclic loose CNS test results on Monterey Sand with displacement amplitude = 0.75 mm

Figure 77 – Cyclic loose CNS test results on Monterey Sand with displacement amplitude = 1 mm

Figure 78 – Cyclic loose CNS test results on Monterey Sand with displacement amplitude = 2 mm

5.1.1 Summary of Results of Cyclic CNS tests for Dense and loose samples

The results of the medium dense and loose tests show that increasing the shear displacement amplitude increases substantially the rate of contraction, and a corresponding decrease in the normal stress.

The relative density of the sample plays a major role in the general behavior of the test: The loose samples exhibited more rapid contraction thus needing fewer cycles to reach a normal stress of 0 kPa. For a shear displacement amplitude of 0.25 mm, the medium dense sample reached 35 cycles whereas the loose sample reached only 7.

The results are summarized by the loss in the peak-to-peak shear stress vs. the number of cycles in Figure 79 and Figure 80.

Figure 79 – Peak to peak shear stress vs. number of cycles summary for loose samples.

Figure 80 - Peak to peak shear stress vs. number of cycles summary for dense samples.

Figure 81 shows shear stress vs. the normal stress for all the displacement amplitudes. Additionally, can be seen that the angles of friction are similar to the angles of friction from the monotonic interface tests.

Figure 81 – Shear stress vs. normal stress summary for dense samples.

Figure 82 – Shear stress vs. normal stress summary for loose samples.

5.2 Influence of value of the constant normal stiffness

The amount of change in the normal stress during shear depends very strongly on the value of the normal stiffness and the amount of contraction or dilation of the sample. Higher values of normal stiffness will cause a more rapid change in the normal stress. This is corroborated by the results displayed in Figure 83.

Figure 83 – Peak to Peak Shear Stress vs Number of Cycles of CNS tests with different stiffness.

The 3 tests shown in Figure 83 were performed with the same sand with the same relative density ($D_r = 50\%$). Horizontal displacement amplitudes (0.25 mm) and a cyclic period (10 sec) were kept the same. The only difference is the value of normal stiffness: 225 kPa/mm, 680 kPa/mm, and 1900 kPa/mm.

The trend is clear: a higher constant normal stiffness value causes a significantly faster reduction in the normal stress.

Figure 84 – Normal Stress vs Number of Cycles of CNS tests with different stiffness.

It can also be seen in Figure 83 that the starting peak-to-peak shear stress of the 3 tests is different. This is due to the reduction of normal stress within the first cycle, with the amount differing depending on the stiffness that was used. This is visualized in Figure 85.

Figure 85 – Reduction of normal stress during the first cycles for different stiffness.

5.3 Testing for NGI

Although the overall objective of this thesis was to develop a CNS apparatus, this study is part of a larger research project to analyze the degradation of the shaft friction in piles due to cyclic loading. One part of the project consists of performing in-situ tests on a 4.5 m long model pile. The site is in Allen Harbor, Quonset, Rhode Island. The characterization of the site was done by means of 4 SCPT tests (Seismic cone penetration test), 1 CPT test (Cone Penetration Test), and sample collections at 6 points (see Figure 86, URI 1 to 6).

Figure 86 – Site Plan for in-situ Testing. (Keefe 2019)

CNS tests were performed to develop a better understanding of the shaft resistance of the test piles. The results of the CNS tests will be used by the Norwegian Geotechnical Institute for adjusting parameters from the designing procedure of offshore piles. The tests should be performed under the conditions requested by NGI, which are covered in chapter 4.3.1.

5.3.1 CNS Testing Conditions for NGI

NGI (Norwegian Geotechnical Institute), the sponsor of the project, requested CNS shear tests to be performed within the conditions enlisted in Table 8.

Table 8 – NGI Test Matrix

Field Depth (m)	Normal Stress (kPa)	Shear Stiffness (kPa/mm)	Density (%)	Type	Amplitude
0.3*Pile length	Field	Field (Gmax)	Field	Monotonic	30
0.3*Pile length	Field	Field (Gmax)	Field	2-way, displ.	+/- 1 mm (Confirmed from monotonic)
0.3*Pile length	Field	Field (Gmax)	Field	2-way, displ.	+/- 5 mm (Confirmed from monotonic)
0.3*Pile length	Field	Field (Gmax)	Field	2-way, stress.	Decided from monotonic
Repeat for 0.5 L _{field}					
Repeat for 0.8 L _{field}					

The lab testing conditions are correlated with the soil properties at the site in Figure 86. With the results of the 4 SCPT and the 1 CPT tests done in the

different locations, correlations were used by Keefe (2019) in order to determine the soil properties. The length of the model pile is 4.5 m (15 ft.). The properties for the NGI test matrix are listed in Table 9. The values of normal stress, constant normal stiffness, and density are averages of the different SCPT and CPT locations.

Table 9 - Correlated Site Soil Properties (Keefe 2019)

NGI Test Depth	Depth (m)	Normal Stress (kPa)	Shear modulus (MPa)	Constant Normal Stiffness (kPa/mm)	Relative Density (%)
0.3*Pile length	1.4	27.2	29	1015.1	73.01
0.5*Pile length	2.3	51.14	36.9	1290.4	81.12
0.8*Pile length	3.7	117.44	45.5	1591.8	84.56

After consulting with NGI, it was determined that the displacement amplitudes employed in the cyclic tests should be determined from the results of the monotonic tests. A displacement amplitude was to be chosen that represented a shear stress prior to reaching the peak shear stress, and another displacement amplitude just past the peak shear stress.

All the samples were prepared to the same density with the intention of making dense specimens. No information on the minimum and maximum densities was available for this study.

The test matrix that was used for the actual testing is shown in Table 10. The values of the constant normal stiffness employed were close to the values correlated by Keefe (2019) from the in-situ soil tests.

5.3.2 Results of Testing for NGI

In this chapter are presented the results of the tests for NGI. Table 10 shows the test matrix of the tests that were performed. Tests were performed with Monterrey sand (see Table 1). The cyclic period was set to 10 s. The interface consisted of a sand – smooth steel surface.

Table 10. Test Matrix of Test performed for NGI.

Depth (m)	Normal Stress (kPa)	Constant Normal Stiffness (kPa/mm)	Relative Density (%)	Displacement Amplitudes (mm)	Shear Stress Amplitudes (kPa)
1.4	27	970	dense	+/-0.3 +/- 0.6	+/- 5
2.3	52	1235	dense	+/-0.15 +/-0.4	+/- 7.5
3.7	117	1570	dense	+/-0.1 +/-0.25	+/- 15

Monotonic CNS tests and stress controlled cyclic CNS tests were also performed.

5.3.2.1 Depth = 1.4 m

Figure 87 – Results of Monotonic CNS Test for Depth = 1.4 m to determine the displacement amplitude for the cyclic tests. Allen harbor sand sample URI-1 S2.

Shear tests performed at low normal stresses dilate the most. In the results presented in Figure 87, the dilation led to an increase in the normal stress and thus in the shear stress. Due to this it is difficult to choose a displacement amplitude for the cyclic shear testing. Hence it was decided to run a monotonic CNL test. The results are shown in Figure 88.

Based on these results displacement amplitudes of $\pm 0.3 \text{ mm}$ and $\pm 0.6 \text{ mm}$ were chosen for the strain-controlled cyclic testing. It was of interest for these tests choosing displacement amplitudes that were, based on the monotonic tests, before and after reaching the peak shear stresses. For the stress-controlled cyclic testing was chosen a stress amplitude of 5 kPa.

The results of the cyclic tests at stresses corresponding to 0.3 L are shown in Figure 89 through Figure 91.

Figure 88 – Results of Monotonic CNL Test for Depth = 1.4 m to determine the displacement amplitude for the cyclic tests. Allen harbor sand sample URI-1 S2.

Figure 89 – Results of Cyclic Test with displacement amplitude = 0.3 mm for Depth = 1.4 m. Allen harbor sand sample URI-1 S2.

Figure 90 – Results of Cyclic Test with displacement amplitude = 0.6 mm for Depth = 1.4 m. Allen harbor sand sample URI-1 S2.

Figure 91 – Results of cyclic test with shear stress amplitude = 5 kPa mm for depth = 1.4 m. Allen harbor sand sample URI-1 S2.

5.3.2.2 Depth = 2.3 m

Based on the results of the CNS monotonic tests shown in Figure 92, displacement amplitudes of $\pm 0.15 \text{ mm}$ and $\pm 0.4 \text{ mm}$ were chosen. For the stress-controlled cyclic tests, a shear stress amplitude of $\pm 7.5 \text{ kPa}$ was chosen. It was of interest for these tests choosing displacement amplitudes that were, based on the monotonic tests, before and after reaching the peak shear stresses. The cyclic results are shown in Figure 93 through Figure 95.

Figure 92 – Results of Monotonic CNS Test for Depth = 2.3 m to determine the displacement amplitude for the cyclic tests. Allen harbor sand sample URI-1 S2.

Figure 93 – Results of Cyclic Test with displacement amplitude = 0.15 mm for Depth = 2.3 m. Allen harbor sand sample URI-1 S2.

Figure 94 – Results of Cyclic Test with displacement amplitude = 0.4 mm for Depth = 2.3 m. Allen harbor sand sample URI-1 S2.

Figure 95 – Results of Cyclic Test with displacement amplitude = 0.4 mm for Depth = 2.3 m. Allen harbor sand sample URI-1 S2.

5.3.2.3 Depth = 3.7 m

Based on the results of the CNS monotonic tests shown in Figure 96, displacement amplitudes of ± 0.1 mm and ± 0.25 mm were chosen. For the stress-controlled cyclic tests, a shear stress amplitude of ± 15 kPa was chosen. It was of interest for these tests choosing displacement amplitudes that were, based on the monotonic tests, before and after reaching the peak shear stresses.

Figure 96 - Results of Monotonic CNS Test for Depth = 3.7 m to determine the displacement amplitude for the cyclic tests. Allen harbor sand sample URI-2 S3.

Figure 97 – Results of Cyclic Test with displacement amplitude = 0.1 mm for Depth = 3.7 m. Allen harbor sand sample URI-2 S3.

Figure 98 - Results of Cyclic Test with displacement amplitude = 0.25 mm for Depth = 3.7 m. Allen harbor sand sample URI-2 S3.

Figure 99 - Results of Cyclic Test with displacement amplitude = 0.25 mm for Depth = 3.7 m.
Allen harbor sand sample URI-2 S3.

CHAPTER VI - SUMMARY AND CONCLUSIONS

Offshore piles may experience degradation in the shaft friction due to cyclic axial loading. The mechanism responsible for this is the contraction of the shear band due to cyclic shear displacements. The contraction causes a reduction of the normal stress acting on the pile and thus a reduction in the shear strength. This can be modeled in lab tests and numerically by using a constant normal stiffness boundary condition. This boundary condition is almost never considered in laboratory testing.

The main objective of this thesis was to develop a cyclic constant normal stiffness (CNS) direct shear apparatus. The second objective was to perform cyclic CNS tests in support of a research project on axial pile behavior funded by the Norwegian Geotechnical Institute and the U.S. Bureau of Safety and Environmental Enforcement (BSEE).

The first objective was accomplished by modifying a cyclic simple shear device manufactured by the GeoComp Corp. The CNS boundary condition was achieved by replacing the cross arm with thinner beams of varying thickness that acted as a beam. A series of strain-controlled, monotonic and cyclic CNS interface tests were performed on samples of Monterey sand to investigate the effects of relative density, initial normal stress, displacement amplitude, and normal stiffness on the results.

Based on the results of these tests, the following conclusions can be made:

- Dilation causes an increase in normal stress and contraction causes a reduction in the normal stress.
- In both monotonic and cyclic tests, the value of the constant normal stiffness plays a significant role in the overall behavior. Higher stiffness causes the changes in the normal stress to be faster. This is especially significant for contractive sands, where after a few cycles a major reduction in the normal stress occurs.
- In cyclic testing, the displacement amplitude has a significant effect in the reduction of the normal stress. High displacement amplitudes may fully mobilize the full shear strength of the sample in one cycle, whereas lower displacement amplitudes don't fully mobilize the shear strength until after a number of cycles of loading.

The results presented herein will be used to model pile performance for an ongoing research project. For future work, it is of interest to evaluate the influence of the roughness of the interface. Additionally, it is of interest to study the effect of soil type on these results.

Some improvements to the equipment could be made to minimize sand loss during cycling, especially in CNS tests performed with high displacement amplitudes.

APPENDICES

Appendix A: Determining values of normal using a thin beam

The value of the constant normal stiffness (k_{CNS}) of a soil can be calculated with the following equation:

$$k_{CNS} = \frac{4G}{D} \quad (3)$$

where

- k_{CNS} : Constant normal stiffness
- G : Shear modulus in kPa
- D : Diameter of the pile in mm

When the target value of k_{CNS} is known the next step is to choose a bar with a similar value of stiffness. As part of this thesis, a series of different cross arms of varying bending stiffness were produced.

If the stiffness of a bar is unknown, it can be determined using the direct shear apparatus.

The first step is to take out the shear box water bath and replace it with a solid piece of metal that will not deform.

Place on top of the metal piece the metal cap with the concavity in the middle that fits the steel ball. The set-up should look like in Figure 100.

Figure 100 – Metal piece used as a boundary.

Make sure to check that the new cross arm is level and tightened with a torque wrench to 10 lb. ft or 13.55 Nm. Consistency in tightening the nuts holding the cross arm is critical for consistent stiffness measurements.

NOTE: During the consolidation phase the cross arm will move down with the loading frame, thus applying the load to the bar. The LVDT attached to the middle of the bar will measure the bending of the bar as well as lowering of the loading frame. Because of this, an additional LVDT must be mounted on top of one of the threaded rods and the difference between the two LVDT readings is the deformation of the beam.

Set the horizontal LVDT on the top of one of the threaded rods of the loading frame. You can use for this a connector like the one shown in Figure 101.

Figure 101 – Connector for attaching LVDT

After doing so the direct shear apparatus should look like in Figure 102.

The next step is to run consolidation tests. There should be run for multiple steps with increasing normal stress. Between each step should be let the normal stress stabilize before starting the next one.

The amount of bending in the bar will be the value of the horizontal strain gauge minus the value of the vertical strain gauge (Use only the absolute values). An example of this in an excel worksheet is shown in Figure 103.

The last step is to plot the normal stress or load with the bending of the bar (Figure 104). The trend of the curve is the bending stiffness of the bar. The bending stiffness of the bar is the constant normal stiffness when using this specific bar.

Figure 102 - Set-up for determining the bending stiffness of a plate.

Load [kPa]	H Def [mm]	V Def [mm]	Abs H Def [mm]	Abs V Def [mm]	H-V [mm]
0	-18.236	0.89309	0	0	0
10	-18.368	0.82836	0.132	0.06473	0.06727
20	-18.458	0.78812	0.222	0.10497	0.11703
30	-18.545	0.76625	0.309	0.12684	0.18216
40	-18.64	0.75663	0.404	0.13646	0.26754
50	-18.745	0.75313	0.509	0.13996	0.36904
100	-19.136	0.73476	0.9	0.15833	0.74167
150	-19.493	0.7094	1.257	0.18369	1.07331
200	-19.836	0.67616	1.6	0.21693	1.38307
250	-20.158	0.66129	1.922	0.2318	1.6902
300	-20.487	0.64642	2.251	0.24667	2.00433
350	-20.786	0.63767	2.55	0.25542	2.29458
400	-21.021	0.62018	2.785	0.27291	2.51209
450	-21.266	0.60443	3.03	0.28866	2.74134
500	-21.515	0.58344	3.279	0.30965	2.96935
550	-21.731	0.56682	3.495	0.32627	3.16873

Figure 103 – Example of an Excel table for determining the bending stiffness of a plate.

Figure 104 – Plot of normal stress vs. vertical deformation with a trend line.

If a specific k_{CNS} is desired but the required height of the bar is unknown, Figure 105 can be used. The 2 lines represent the bending stiffness of a bar with increasing height for a simply supported and completely fixed system. The green point represents the stiffness of a plate 6.9 mm thick. The yellow point represents the stiffness of a plate 10 mm thick.

Figure 105 - Theoretical bending stiffness of an ideally fixed and ideally simply supported plate and determined bending stiffness of different plates.

Appendix B: Running a monotonic CNS direct shear test

Running CNS tests is very similar to running a CNL test. The first thing to do is to replace the original black bar used for CNL testing by the cross arm with the k_{CNS} of interest. The original screw that connects the black bar to the vertical load cell needs to be replaced by a smaller screw.

The nuts that fix the new bar to the threaded rod of the loading frame should be, if possible, tightened to 10 lb. ft or 13.55 Nm.

In the software of ShearTrac-II can now be set the desired value of initial normal stress for the consolidation phase.

Begin the consolidation phase. When the normal stress stabilizes the 4 lifting screws must be put in contact the lower half of the box and be turned 180 degrees. This will create a gap between the two halves of the shear box in order to prevent any additional friction. Now the 2 plastic bolts that hold the direct shear box together can be removed and subsequently the 4 lifting screws need to be unscrewed.

Before the shearing phase is started, the vertical load frame must be locked by setting the velocity limit to 0. This can be done under Options/PID/Vertical Load (see Figure 106). Set the value of the velocity limit to 0, click “apply” and “Ok”.

Figure 106 – Settings for locking the loading frame vertically.

Appendix C: Running a Cyclic CNS direct shear tests

Cyclic CNS tests must be performed in the cyclic simple shear apparatus and the equipment must be set up as shown in Figure 107.

Figure 107 – Cyclic CNS direct shear apparatus setup.

In its original form, was this apparatus conceived for cyclic simple shear tests. For running cyclic direct shear tests, modifications had to be done. The first modification was attaching an angle on the top of the base of the steel arm. At the end of this angle is attached a metal rod (Figure 108). The function of this metal rod is to prevent the upper half of the shear box from moving horizontally.

Figure 108 – Angle attached to the top of the base of the metal arm.

If the apparatus looks like that in Figure 107, can be followed the subsequent step by step instructions for performing a CNS cyclic direct shear test.

Step by Step Instructions

Replace the original black plate provided by GEOCOMP with a plate with the bending stiffness of interest. Raise the metal arm. Make sure that the angle is attached to the base of the metal arm and that the metal rod is set up like in Figure 109.

Figure 109 – Metal rod set-up

Prepare the sample and place the shear box in the shear apparatus. Make sure that there is no gap between the nut and the screw attached to the upper half of the shear box (see Figure 110).

Figure 110 - Connection between the metal rod and Shear box. Left is incorrect, Right is correct.

In cyclic testing, it is important that the lower half of the shear box is tightly fixed. For doing so, use the spacers as shown in Figure 111, and firmly tighten the lower half shear box.

Figure 111 – Set-up of the shear box in shear apparatus with spacers.

Now the steel restraining arm can be lowered. While doing so, insert the metal rod from the loading frame in the steel arm. Move the plate from the loading frame in position and tighten it to 10 lb. ft or 13.55 Nm. The set-up should look like in Figure 107. Begin the test.

Open the Shear-Trac software CDSS. If a warning indicates that there is no connection between the equipment and the computer attached to it, restart the computer while the shear apparatus is turned on (restart; do not turn off and on).

For cyclic testing, it is very important to set in the software an accurate value of the sample height. This due to that the amount of shear displacement is dependent on the value of the sample height. This is done in the specimen table, shown in Figure 112. It is also important to input the value of the diameter of the shear box.

CDSS - C:\Users\Geotech\Desktop\Cyclic testing SS 2019\07.06\CNS, Strain Amplit...

File View Run Calibrate Control Report Options Help

Project Specimen Water Content Read Table Test Parameters Consolidation Table Cyclic Table Shear Table

Initial Diameter: 63.5 mm

Initial Height: 24.5 mm

Initial Sample Mass: 0.12474 kg

Specific Gravity: Measured 2.65

Plasticity: ☐ Plastic ☐ Non-Plastic ☒ Unknown

Liquid Limit: 0

Plastic Limit: 0

Figure 112 - Specimen table

In the cyclic table (Figure 113), one can choose between strain and stress-controlled tests. If strain-controlled (under cyclic control) is chosen, a value of cyclic strain amplitude can be set. This value will determine how much the shear box will move.

$$\text{Shear Displacement Amplitude} = \frac{\text{Cyclic Strain Amplitude} * \text{Sample Height}}{100} \quad (4)$$

The cyclic period of the loading can also be input.

CDSS - C:\Users\Geotech\Desktop\Cyclic testing SS 2019\07.06\CNS, Strain Amplitude = 1%. not fixed.dat

File View Run Calibrate Control Report Options Help

Project Specimen Water Content Read Table Test Parameters Consolidation Table Cyclic Table Shear Table

	Strain Amplitude %	Maximum Pressure Ratio	Cycle Period s	Maximum Number of Cycles	Number of Readings per Cycle
1	1	0.995	10	1000	128
2	0	0	0	0	0
3	0	0	0	0	0
4	0	0	0	0	0
5	0	0	0	0	0
6	0	0	0	0	0
7	0	0	0	0	0
8	0	0	0	0	0
9	0	0	0	0	0
10	0	0	0	0	0

Cyclic Control: Desired Response Gain: Max. Mean Horz. Strain: %

Normal Control: Gain Update Period: Cycle Max. Total Horz. Strain: %

Constant Volume Gain: Gain Update Window: Cycle Filter Norm. Cutoff Freq.:

Figure 113 – Cyclic table

It is recommended to input the rest of the settings in the cyclic table as they are in Figure 113. If tests are run with low strain amplitudes, the desired response gain can be increased to 6 or 7. This will make the strain amplitude curve during testing more accurate.

In the consolidation table, set the value of the normal stress. Note that the rod of the loading frame is not attached to the load cell, thus is this weight acting on the soil probe. This amounts to roughly 2 kPa.

For running CNS tests, the cyclic table, the “Normal Control” must be set to no control. This will prevent the loading frame from moving during shearing.

Make sure that all screws are tight before starting. The test can now be started. After the consolidation phase is concluded the two bolts holding the two halves of the shear box together must be removed.

Begin the cyclic shear phase. After the test is concluded the results can be reviewed under the report tab (Figure 114). If you want to extract the results of an excel table, you can click while viewing the results on view/export to export the data to an excel file.

Figure 114 – Results of a CNS direct shear test.

Appendix D: Cyclic Tests on Rubber Samples

The following figures show the results of tests performed on a rubber sample (Figure 115 and Figure 116). The tests were performed to analyze the disturbances (jumps in the shear stress) noted in subchapter 4.2. When tests are performed on rubber samples, similar jumps can be seen in the shear stress.

Figure 115 - Results of cyclic tests performed on rubber sample.

Since the tests are strain-controlled, the jumps are in the shear stress-dependent of the horizontal displacement. In the graphs for the horizontal displacement can be seen that at the indicated points, for a small period of time, there is a decrease in the shearing velocity. This decrease in the velocity causes a jump in the shear stress. This jump is more notable for low strain amplitudes because the apparatus is less accurate for low values.

Figure 116 - Results of cyclic tests performed on rubber sample.

The rubber tests also lead to the conclusion that the jumps are caused by the velocity regulating mechanism of the cyclic shear apparatus and not the shear box.

Appendix E: Improvements for avoiding sand loss

Shambhu Sharma, Principle Engineer of NGI in the Perth office, recommended a method for avoiding sand loss during CNS cyclic testing: Wrapping the sand sample in a rubber band similar to the triaxial test. This reduces overall the sand loss and makes it possible to measure it if it occurs.

Appendix F: Cyclic CNS Tests on Alan Harbor Sand

Figure 117 shows the grain size distributions of samples obtained from URI's Allen Harbor Educational Facility at boring location URI-1

Figure 117 – Sieve Line for Alan Harbor Sand in Location URI-1. (Keefe 2019)

Table 11 shows the Test Matrix for the tests performed on the Alan Harbor Sand. The sand samples were chosen to be the closest to the targeted depths.

Table 11 – Test Matrix for Cyclic CNS Tests on Alan Harbor Sand

Sample Used	Targeted Depth (m)	Normal Stress (kPa)	Constant Normal Stiffness (kPa/mm)	Mass of Sample (gr)	Displacement Amplitudes (mm)
URI -1 S2 (1.524 -2.1336 m)	1.4	27	970	125	+/- 0.1 +/- 0.25 +/- 0.5
URI -1 S2 (1.524 -2.1336 m)	2.3	52	1235	125	+/- 0.1 +/- 0.25 +/- 1
URI - 2 S4 (4.572 – 5.1816 m)	3.7	117	1570	125	+/- 0.1 +/- 0.25 +/- 1

1.4 m target depth

Figure 118 - Results of Monotonic CNS Test on Alan Harbor Sand (URI -1 S2) for targeted Depth = 1.4 m

Figure 119 - Results of Cyclic CNS Test on Alan Harbor Sand (URI -1 S2) with displacement amplitude = 0.1 mm for targeted Depth = 1.4 m

Figure 120 - Results of Cyclic CNS Test on Alan Harbor Sand (URI -1 S2) with displacement amplitude = 0.25 mm for targeted Depth = 1.4 m

Figure 121 - Results of Cyclic CNS Test on Alan Harbor Sand (URI -1 S2) with displacement amplitude = 0.5 mm for targeted Depth = 1.4 m

2.3 m target depth

Figure 122 - Results of Monotonic CNS Test on Alan Harbor Sand (URI -1 S2) for targeted Depth = 2.3 m

Figure 123 - Results of Cyclic CNS Test on Alan Harbor Sand (URI -1 S2) with displacement amplitude = 0.1 mm for targeted Depth = 2.3 m

Figure 124 - Results of Cyclic CNS Test on Alan Harbor Sand (URI -1 S2) with displacement amplitude = 0.25 mm for targeted Depth = 2.3 m

Figure 125 - Results of Cyclic CNS Test on Alan Harbor Sand (URI -1 S2) with displacement amplitude = 1 mm for targeted Depth = 2.3 m

3.7 m target depth

Figure 126 - Results of Monotonic CNS Test on Alan Harbor Sand (URI -1 S4) for targeted Depth = 3.7 m

Figure 127 - Results of Cyclic CNS Test on Alan Harbor Sand (URI -1 S4) with displacement amplitude = 0.1 mm for targeted Depth = 3.7 m

Figure 128 - Results of Cyclic CNS Test on Alan Harbor Sand (URI -1 S4) with displacement amplitude = 0.5 mm for targeted Depth = 3.7 m

Figure 129 - Results of Cyclic CNS Test on Alan Harbor Sand (URI -1 S4) with displacement amplitude = 1 mm for targeted Depth = 3.7 m

Additional Tests

Figure 130 - Results of dense cyclic CNS test on Allen Harbor sand (URI -1 S2) with displacement amplitude = 0.25 mm for targeted depth = 1.4 m

Figure 131 - Results of loose cyclic CNS test on Allen Harbor sand (URI -1 S2) with displacement amplitude = 0.25 mm for targeted depth = 1.4 m

Figure 132 - Results of dense cyclic CNS test on Allen Harbor sand (URI -1 S4) with displacement amplitude = 0.25 mm for targeted depth = 2.3 m

Figure 133 - Results of loose cyclic CNS test on Allen Harbor sand (URI -1 S4) with displacement amplitude = 0.25 mm for targeted depth = 2.3 m

Figure 134 - Results of dense cyclic CNS test on Allen Harbor sand (URI -1 S4) with displacement amplitude = 0.25 mm for targeted depth = 3.7 m

Figure 135 - Results of loose cyclic CNS test on Allen Harbor sand (URI -1 S4) with displacement amplitude = 0.25 mm for targeted depth = 3.7 m

Appendix G: Graphs of all cyclic CNS tests performed

Tests on Monterey Sand with Stiffness = 225 kPa/mm

Medium Dense samples ($D_r = 60\%$)

Loose samples (Dr= 10%)

Comparison of tests with different constant normal stiffness

Note: The vertical displacement in the test shown above was not measured correctly.

Tests on Allen Harbor Sand

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